# Section 4

# Wastewater Treatment Plants – Existing Conditions

## 4.1 Introduction

This section describes the age and condition of the Water Pollution Control Authority's (WPCA's) two wastewater treatment plants (WWTPs) and associated outfalls. In addition, a description of the performance of the two facilities, including operations under high flow events, is discussed, as well as presenting a summary of the environmental site assessment. This information was gained through the review of existing plans, specifications, operations and maintenance (O&M) manuals, and past reports, along with site visits, discussions with plant operators, a review of Connecticut Department of Energy and Environmental Protection's (DEEP's) environmental records, a hazardous building material survey and a preliminary soil sampling program. All the information presented herein feeds into the decision-making process related to retaining or rebuilding infrastructure and the cost to implement.

## 4.2 West Side Wastewater Treatment Plant

The existing West Side WWTP, located at 205 Bostwick Avenue, currently consists of influent screens, influent pumping, rectangular primary settling tanks, and an activated sludge process modified to operate in a Modified Ludzack-Ettinger (MLE) configuration for nitrogen removal, with rectangular secondary clarifiers, sodium hypochlorite for disinfection and sodium bisulfite for dechlorination with discharge via a 72-inch effluent outfall into Cedar Creek (Black Rock Harbor). Primary sludge is directed to a sludge degritter before being conveyed to the gravity thickeners (GTs) and combined with thickened waste activated sludge (WAS). Thickened sludge is held in a thickened sludge storage tank and trucked off-site for disposal. The process flow diagram is presented in **Figure 4.2-1.**

Wastewater infrastructure has existed on this site since at least the early 1900s. In 1969 the facility was upgraded from a primary treatment plant to a secondary treatment plant under the "Additions and Alterations to Wastewater Treatment Facilities" project. The treatment facility had significant mechanical system upgrades under the 1992 "Rehabilitation of West Side Wastewater Treatment Plant" project. Under the 2001 "Interim BNR Retrofit" project the activated sludge system was converted to an MLE process to impart some level of nitrification and denitrification in accordance with the DEEP's General Permit for Nitrogen Discharges. In addition, dechlorination was incorporated in 2004 under the "Dechlorination Facilities Project No. 3100". An aerial of the West Side WWTP site is presented in **Figure 4.2-2**.

The West Side WWTP was originally designed to achieve secondary effluent quality at annual average design flow capacity of 30 million gallons per day (mgd) and a peak secondary treatment capacity of 58 mgd. All wet weather flows in excess of the secondary treatment capacity, up to 90 mgd, receives primary treatment before recombining with secondary effluent prior to effluent disinfection.

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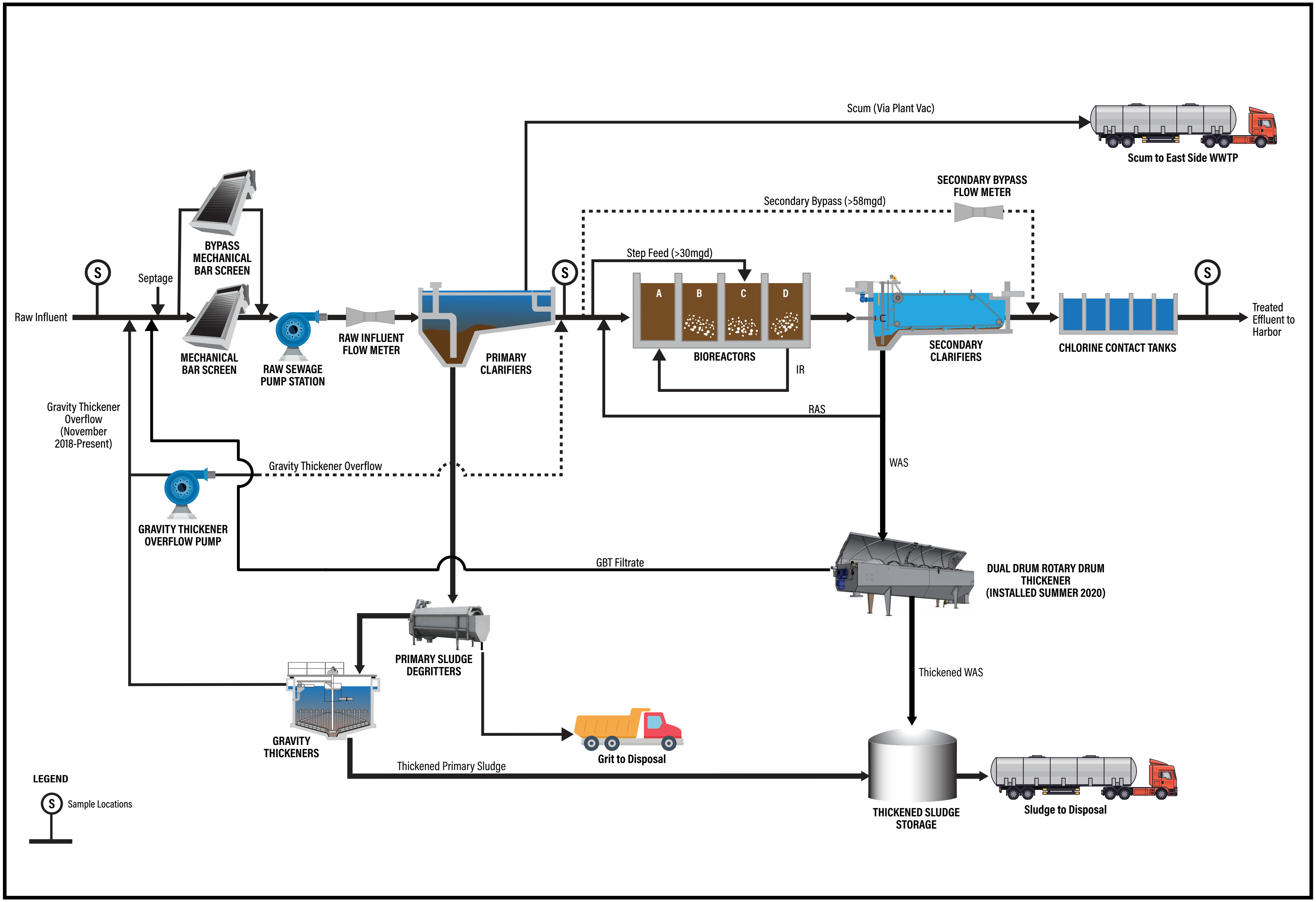

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**Figure 4.2-1**
**West Side Process Flow Diagram**

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Figure 4.2-2
Existing Conditions - West Side WWTP

Water Pollution Control Authority, City of Bridgeport
Facilities Plan

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No major upgrades have been completed at this facility since the dechlorination project in 2004. In general, the entire facility has exceeded its useful life expectancy. Many areas of the plant are not in compliance with current codes and standards and much of the facility is below the existing 100-year flood plain, and therefore at additional risk with rising sea level.

### 4.2.1 Influent Structures and Influent Screening

The majority of the flow to the West Side WWTP is conveyed to the plant through one 72-inch and one 78-inch gravity sewers, which connect in a concrete junction chamber just north of the existing fence line at the corner of Bostwick Avenue and Morris Street. About 10-feet upstream of the junction chamber, the 78-inch line is redirected to the junction chamber as shown on **Figure 4.2-3.** The interiors of these structures were not inspected as a part of this planning effort, but it was reported by operators that any internal gates or equipment are not functional. No information on these structures was found on historical drawings.

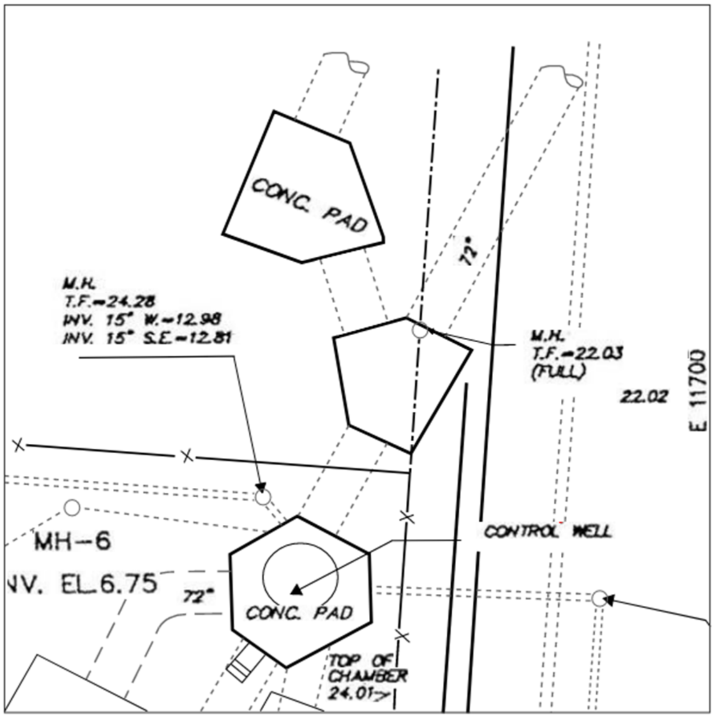

**Figure 4.2-3**
**Influent Chambers**

Following the influent junction chamber is the Influent Control Well, located in the northeastern most corner of the fenced site. This structure receives flow from the junction chamber as well as two smaller sewer lines serving local areas. The influent sample for permitting is taken from this influent control well; however, it does not include the flow and load from local lines. The combined flow exits the influent control well through two ports – a 72-inch pipe to the Screen Building and a 72-inch pipe to the Bypass Screen in the Raw Sewage Pump Station. Flow through these pipes is controlled via sluice gates located in the two buildings at the discharge location.

The West Side WWTP accepts septage from domestic and commercial sources six days per week. Septage is introduced to the liquid treatment system just upstream of the main influent screen, located upstream of the raw wastewater pumping station. There are no special facilities for receiving (nor processing) septage waste.

The plant has two influent screens: the Main Screen and the Bypass Screen, to remove screenings ahead of the influent pumps. The Main Screen, located in the Screening Building, is a climber style bar screen by Infilco Degremont with a bar spacing of ½-inch. A sluice gate upstream of the screen controls the influent flow to the plant, limited to 90 mgd. The screen operates automatically based on a timer setting with an override based on level. A 24-inch gravity thickener overflow/drain line discharges into the main screen channel just upstream of the screen, and downstream of the sluice gate. The Bypass Screen, located in the Raw Sewage Pump Station, is also an Infilco Degremont climber style screen but with ¾-inch bar spacing. The operation of the bypass screen is similar to the main screen, operation on a timer with a level differential override. A sluice gate is also located ahead of this screen to limit flow to the plant.

Each screen can pass the peak influent flow of 90 mgd. Normal operation is for the Main Screen to be in service, with the Bypass Screen put into service when the Main Screen is taken out of operation for maintenance or repairs.

Each screen is equipped with a dedicated screenings washer/compactor located directly below the screen discharge point to wash, dewater, and compact the screenings prior to discharge to a disposal bin.

Prior to the 1992 improvement project, a channel downstream of the Main Screen served as a horizontal grit channel with longitudinal flight collection system and vertical elevator system to collect and lift grit out of the channel. The grit removal system was removed in the 1992 project. To help prevent grit and other solids from setting out in this channel, it was equipped with coarse bubbler diffusers. The air for the diffusers in the channel is provided by two blowers located in dedicated blower room in the Screen Building.

### 4.2.1.1 Assessment and Deficiencies – Influent Structures and Screening

The Monthly Operating Reports (MORs) indicate that annual average septage quantity from 2017-2019 was 42,500 gallons per day (gpd) (daily septage receipt ranged from 3,000 gpd to 105,000 gpd). This is greater than the reported average 2008 septage received (30,000 to 33,000) reported in the 2012 Sludge Processing Report. Plant staff routinely collect a small aliquot of each septage load and record a pH measurement of the sample. Aside from the pH measurements, there is no known septage characterization data available to assess the impacts septage could be having on the West Side WWTP's performance.

As evidenced by the screenings that accumulate in downstream processes, the current screening system is ineffective, due to the age, configuration, and size of the screening system. The Screen Building and Control Well are more than 50 years old and the screens and associated mechanical equipment are nearly 30 years old. The overall system lacks grit removal, necessitating de-gritting of the primary sludge. In addition, area classification issues exist between the Screening Building and the Main Pump Station per the latest National Fire Protection Association (NFPA) 820 fire code and there is electrical equipment located in the Screen Building that are not properly rated for the hazardous space. Lastly, the operating floor and the entrance door elevation to the Screen Building are vulnerable to flooding under current and future flood conditions.

### 4.2.1.2 Recommendations – Influent Structures and Screening

Given the age and condition of the two screening structures and associated equipment, the lack of a grit removal system, and the potential for flooding, it is recommended that these facilities be replaced.

Without a properly engineered septage receiving system, it is possible that septage receival could cause shock loads to the WWTP or have negative impacts on plant processes and effluent quality. TR-16 guidelines report that septage received at municipal wastewater treatment facilities should be treated before being incorporated into either liquid or solid stream unit processes. Additional monitoring is necessary to gather information about the characteristics of hauled waste that is discharged to the WWTP, and to determine if waste should be rejected because of potential negative impacts to the WWTP.

## 4.2.2 Influent Pumping

Following the influent screening process, all flow to the West Side WWTP is conveyed to the Main Lift Pump Station (Raw Sewage Pump Station). This station lifts all flow up to the primary settling tanks to allow gravity flow through all downstream treatment processes and out the effluent outfall.

The original pump station structure (wet wells, dry well, building enclosure) was constructed as part of the original 1916 construction contract. The current pumps were installed as part of the 1992 Modifications and Improvements project and are now nearly 30 years old.

The station consists of four vertical, non-clog centrifugal, single stage, end suction pumps, three duty and one standby. Each pump is sized for 23,500 gallons per minute (gpm) (approximately 34 mgd) at 50.7-feet total dynamic head (TDH), for a total station firm capacity of about 90 mgd to accommodate the peak influent flow and other plant recycle flows. The 400 horsepower (hp) pump motors are equipped with variable frequency drives (VFDs).

The collection system has the capacity to convey more than 90 mgd to the pump station. When the pump station discharge is between 80 and 90 mgd and the wet well level continues to rise above the high level setpoint, the main influent gate in the Control Well upstream of screening building is manually throttled to prevent flooding of the pump station. This results in an increasing hydraulic grade line in the collection system which causes combined sewer overflows (CSOs) in the system.

### 4.2.2.1 Assessment and Deficiencies – Influent Pumping

Given the age and condition of the existing influent pump station and difficulties associated with upgrading an aging building to meet current codes and standards it is recommended that this pump station be replaced.

## 4.2.3 Primary Treatment

The West Side WWTP currently has three 136-ft by 36-ft rectangular primary settling tanks with concrete support center walls with an operating sidewater depth of 10.7 ft.

Screened influent is pumped through a 54-inch force main into the influent distribution channel. The influent flow is measured by a Parshall flume located in the influent distribution channel. Flow isolation to each primary settling tank is provided through three slide gates along the channel. After the isolation gate the flow enters a second dedicated influent channel which distributes flow to each tank through six influent ports. The influent channels are aerated with coarse bubble diffusers to keep solids (including grit) in suspension.

As presented in **Table 4.2-1**, the primary clarifiers were designed with overflow rates far above conventional design standards. Primary clarifiers are typically designed for an average day overflow rate of 1,200 gpd/ft$^2$, and a peak overflow rate of 3,000 gpd/ft$^2$. With all three primary clarifiers in operation at the West Side plant the overflow rate at design average daily flow of 30 mgd is 2,042 gpd/ft$^2$ and at the peak flow of 90 mgd, the overflow rate is 6,127 gpd/ft$^2$. This significantly impacts the performance of the primary clarifiers.

The sludge and scum collection mechanisms were replaced as part of the 1992 upgrades and are nearly 30 years old. Each tank has two sets of non-metallic chain and flight scum and sludge collectors. Cross collectors exist in the sludge trough to direct sludge to the sludge hopper. Primary sludge is drawn from the sludge hopper with a dedicated, recessed impeller vortex pump located in the primary sludge pump gallery at the head end of the primary settling tanks.

Primary effluent flows into six double-sided weir troughs. From the effluent troughs, primary effluent flows to the effluent channel. Flow up to 58 mgd is directed to the secondary treatment system, flow in excess of 58 mgd is directed to a secondary bypass channel which conveys flow to the chlorine contact tanks.

**Table 4.2-1 West Side WWTP Primary Settling Tank Information**

| Primary Settling Tank Information | |
|---|---|
| Number of Tanks | 3 |
| Tank Dimensions | 136 ft x 36 ft x 10.7 ft |
| Surface Area, each | 4,896 ft$^2$ |
| Surface Area: total | 14,688 ft$^2$ |
| Overflow Rate (all tanks operational) | |
| Average | 2,042 gpd/ ft$^2$ |
| Peak | 6,127 gpd/ ft$^2$ |

There are five primary sludge pumps designed for three operating (one per tank), with two standby. These pumps are located in the pump gallery below grade adjacent to the head end of the primary settling tanks. Four of primary sludge pumps are Fairbanks Morse and the final pump is a Chesterton. The characteristics for the primary sludge pumps are shown in **Table 4.2-2**, below.

Table 4.2-2 West Side WWTP Primary Pump Information

| Primary Sludge Pump Information – Fairbanks Morse | |
|---|---|
| Manufacturer | Fairbanks Morse (4 Pumps) |
| Model | Model 4222 SN – K4B1-061754 |
| Type | Nonclog, Recessed Impeller, Vortex Type |
| Number of Pumps | Four |
| Approximate Age | Over 20 years old, some pumps have been rebuilt. |
| rpm | 1000 RPM |
| Capacity | 350 GPM @ 38 FT TDH |
| Motor: | 15 hp Motor |
| **Primary Sludge Pump Information - Chesterton** | |
| Manufacturer: | Chesterton (1 Pump) |
| Model: | 4 x 4 x 12 |
| Number of Pumps: | One |
| Approximate Age: | Over 12 years old. |
| RPM: | 1800 RPM |
| Capacity: | 100 GPM @ 20 psi |
| Motor: | Unknown |

The primary sludge pumps convey sludge collected to the sludge degritters in the Degritter Building. As part of the 1992 project, two grit washers and one grit cyclone/classifier were installed to remove grit from the primary sludge. Since 2012, the grit washers and cyclone/ classifier have been replaced with rotating drum screens. Degritted primary sludge flows by gravity to the gravity thickeners and grit is retained within the drum screen and conveyed to dumpsters for disposal. Dumpster contents are brought to the grit and screenings tent at the East Side WWTP where grit is further drained before hauling and disposal by Synagro Technologies.

Primary scum was designed to be conveyed from the manually operated rotating scum trough to a primary scum pit located adjacent to the primary tanks. According to WWTP staff, the scum removal system never operated well and the system has since been abandoned, resulting in scum accumulation at the primary effluent weirs as shown in **Figure 4.2-4**.

Figure 4.2-4
Scum Accumulation on Primary Clarifier Weirs

Plant operators currently remove scum periodically from the surface of the primary tanks using a vactor truck. Despite this periodic scum removal, a significant amount of scum passes to downstream processes. The scum collected in the vactor truck is transported to the East Side WWTP where the liquid is decanted and discharged at the headworks and the solids are discharged in the solids handling tent at the East Side WWTP until they can be disposed of. Operators have noted that this decanting process of the scum at the East Side WWTP often results in West Side WWTP scum accumulation into the East Side WWTP process.

#### 4.2.3.1 Assessment and Deficiencies – Primary Treatment

The existing primary settling tanks are undersized for all design and observed flow conditions which negatively impact the overall primary treatment performance. Most of the equipment within the primary clarifiers have exceeded its useful life and equipment associated with these tanks is at or beyond its useful life.

No grit removal system currently exists. Grit settles in the sewers, wet well, and primary influent channels under low flows. During peak flows, settled grit is flushed into the primary tanks, increasing the risk of collector failures, over-loading the sludge hopper and clogging primary sludge pumps and discharge piping. The scum collection system is defunct, therefore scum, rags and other floating debris accumulates along the weirs and get flushed to downstream processes during peak flow events.

Although the primary clarifier top of concrete is at elevation 39.0, well above the design flood elevation, the primary sludge pumps are located in the pump gallery at an elevation of approximately 23.0. Entrance into the pump gallery is at elevation 24.0, which could result in flooding of the gallery if dry-proofing is not incorporated.

#### 4.2.3.2 Recommendations – Primary Treatment

The existing conventional primary settling tanks are grossly undersized for design average day and peak flow conditions, and most equipment and piping is past its useful life. It is recommended that the system be rehabilitated and expanded or replaced in its entirety. Tankage could be reused if feasible.

### 4.2.4 Secondary Treatment

The secondary treatment system consists of the bioreactors (aeration tanks with anoxic zones), blowers, secondary clarifiers, return and waste activated sludge pumping (RAS and WAS), and a secondary scum removal system. The original activated sludge process was designed to treat an average daily flow of 30 mgd and a maximum daily flow of 58 mgd to achieve conventional secondary treatment standards. The tanks were constructed as part of the 1969 upgrade. As mentioned previously, a secondary treatment bypass is used to direct influent flow in excess of 58 mgd, up to 90 mgd, from the primary effluent channel directly to the chlorine contact tanks.

The secondary treatment process is currently operated as three individual treatment trains, each consisting of two bioreactors and one rectangular secondary clarifier. Typically, all three treatment trains are operational year-round. Each treatment train has a dedicated RAS pumping system from its designated secondary clarifier and WAS is drawn off the RAS line. Each bioreactor is divided into four, evenly sized compartmentalized cells referred to as Zones “A” through “D”. Fine bubble diffused aeration was incorporated in the early 1990s and upgraded with new diffusers in 2018.

### 4.2.4.1 Bioreactors

The bioreactors were modified as part of the 2001 Interim BNR Retrofit Project to operate in an MLE process configuration to achieve some level of nitrogen removal in accordance with the General Permit for Nitrogen Discharges. At that time, Zone A was converted from an aerobic zone to an anoxic zone for denitrification with mechanical mixing and internal recycle (IR) pumping to return nitrate from the end of the aerobic zone (Zone D) to the anoxic zone (Zone A) as shown in **Figure 4.2-5**. Each anoxic zone has two submersible mechanical mixers to mix primary effluent, RAS and IR flows and keep solids in suspension. IR pumps are designed to provide a recycle up to three times the average design flow. The anoxic zone is followed by three-stage aerobic zone (B, C, and D).

The bioreactors and influent channels are configured such that it is possible to operate in step feed mode for wet weather management by introducing a portion of the primary effluent to Zone B, C, or D. When the influent flow exceeds 30 mgd, the plant is operated in step-feed mode by directing 50% of the primary effluent to Zone C, as shown on Figure 4.2-5, below**.** This mode of operation serves to reduce the solids loading to the secondary clarifiers. **Table 4.2-2** summarizes the biological nutrient removal (BNR) system design criteria.

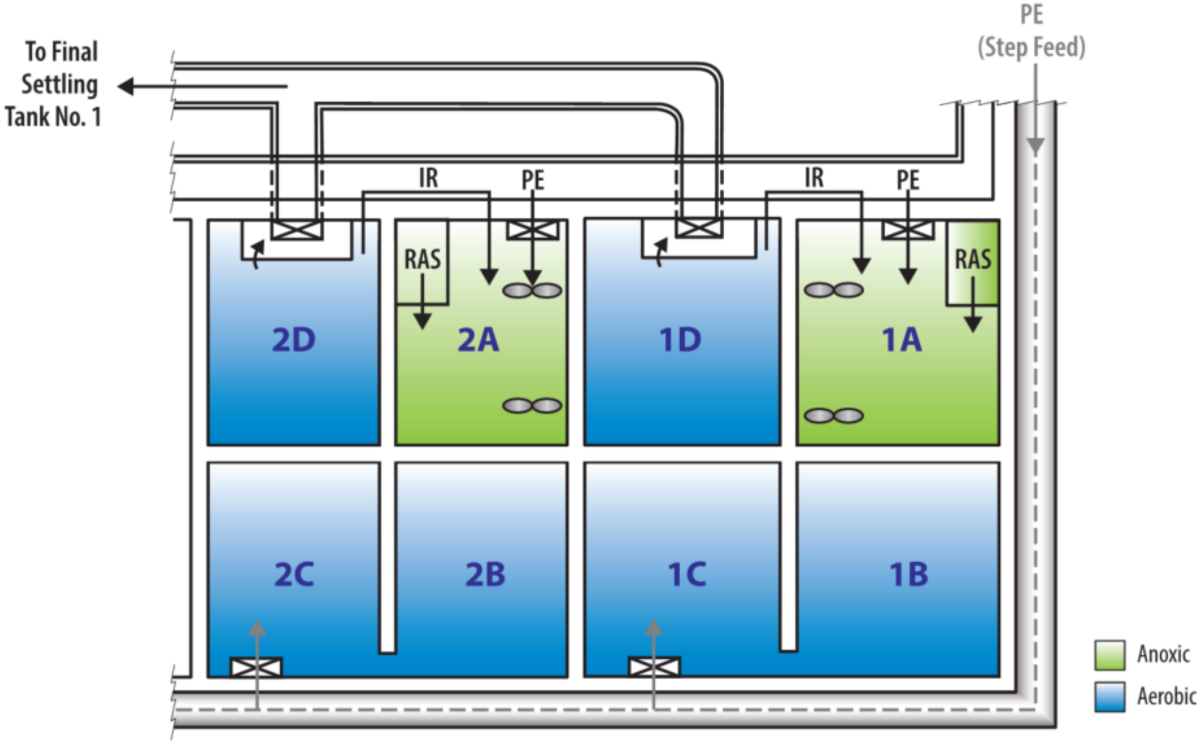

**Figure 4.2-5**
**Schematic of Existing MLE Process: Bioreactors No. 1 and No. 2**

As shown in Table 4.2-2, the existing bioreactors provide a total of 4.92 million gallons (MG) of aeration volume. Based on 2017-2019 primary effluent five-day biochemical oxygen demand ($BOD_5$) loads and required aerobic solids retention time (SRT) required for nitrification, this aerobic volume is inadequate.

Based on the secondary clarifier sizing, to avoid overloading the secondary clarifiers, the maximum allowable mixed liquor suspended solids (MLSS) concentration is 2,600 milligrams per Liter (mg/L) with all secondary clarifiers operational, given current flows and loads. The WWTP currently operates at a much higher MLSS concentration, to maximize nitrogen removal within the existing tankage, thereby eroding the safety factor included in the clarifier design. When influent flow approaches 30 mgd operators shift to step feed mode in order to reduce clarifier loading, however, this mode of operation is still unable to allow the facility to adequately treat flows up to the secondary systems capacity (58 mgd) with an appropriate operating safety factor.

The existing aeration tanks were initially designed for $BOD_5$ removal alone (not nitrogen removal). When the facility was upgraded in 2002 under the “Interim BNR Retrofit” Contract, one of the four zones within each aeration tank was converted to anoxic volume. This modification reduced the aerobic volume by 25%. It was likely understood that nitrogen removal could not be maintained at higher flows nor during the coldest months. But given that the average annual flow was well below 30 mgd at the time, this plant modification made sense to achieve some level of nitrogen removal year-round on an ***interim*** basis.

It has been noted by plant staff that the IR pumps and submersible mixers have experienced severe maintenance challenges, resulting in much of this equipment being out of service for extended periods of time. Rags and other large solids that are not captured at the influent bar screens damage the mixers and recycle pumps in the BNR basins.

**Table 4.2-3 Bioreactor Process Design Data[1]**

| Design Element | Value |
|---|---|
| **Secondary Flows** | |
| Design Average | 30 mgd |
| Peak Flow | 58 mgd |
| Number of Trains | 6 |
| Total Process Volume | 6.55 MG |
| Process Volume, each train | 1.09 MG |
| Average Aerobic SRT (2017-2019) | 15 days |
| Average MLSS (2017-2019) | 4,400 mg/L |
| **Pre-Anoxic Zone** | |
| Total Anoxic Volume | 1.64 MG |
| Anoxic Volume, each train | 0.27 MG |
| Number of Stages, each train | One (A) |
| Stage (zone) Dimensions (L x W x SWD) | A: 74.25 ft x 30 ft x 16.4 ft |
| Diffusers<br>Type<br>Number of Installed Diffusers, per stage<br>Number of Diffuser Holders, per stage | <br>Fine bubble 8.75-inch ceramic disk diffusers<br>A: ---<br>A: 684 |
| Mixer<br>Type<br>Number of units, per train<br>Motor Size | <br>Submersible Mechanical<br>2<br>4.9 hp |
| **Aerobic Zone** | |
| Total Aerobic Volume | 4.92 MG |
| Aerobic Volume, each train | 0.82 MG |
| Number of Stages, each train | Three (B, C, and D) |
| Stage (zone) Dimensions (L x W x SWD) | B: 74.25 ft x 30 ft x 16.4 ft<br>C: 74.25 ft x 30 ft x 16.4 ft<br>D: 74.25 ft x 30 ft x 16.4 ft |
| Internal Recycle Pumps<br>Type<br>Number of Units, total<br>Capacity<br>Motor Size, each<br>Diffusers<br>Type<br>Number of Installed Diffusers, per stage<br><br><br>Number of Diffuser Holders, per stage | <br>Submersible propeller<br>6<br>10,417 gpm (300% forward flow)<br>12.2 hp<br><br>Fine bubble 9-inch ceramic disk diffusers<br>B: 690<br>C: 378<br>D: 234<br>B: 852<br>C: 684<br>D: 684 |

[1]Sidewater depths (SWD) are based on average water surface elevation in the 1992 hydraulic profile.

### 4.2.4.2 Blowers

Air is supplied to the fine-bubble diffusers in the aerobic zones by three, 600 hp, single stage centrifugal blowers manufactured by Roots Dresser. The blowers are housed the Control Building adjacent to the bioreactors. Design data for the aeration blowers are shown in **Table 4.2-3**.

The existing blowers were installed during Contract No. 1 of the 1992 modifications and improvements project. Hach LDO dissolved oxygen (DO) probes were installed in each oxic zone in 2010, but these probes have not been maintained and are no longer used to monitor or control the aeration system.

Currently, the air supply is adjusted manually based on DO readings that are taken during each shift from each of the aeration basins using a portable meter. Based on the readings, the inlet guide vanes on the blowers are positioned to regulate the amount of air supplied. Data collected indicate that the tanks are often over-aerated, resulting in wasted energy and reduced denitrification performance from pumping high DO back to the anoxic zones with the internal recycle.

Typically, only one of the blowers is utilized at a time, and the inlet guide valve opening is set to approximately 75% (open). The setting on the blowers is typically only changed seasonally. During the summer, the guide vane position is set to 90 to 95% to meet the increased air requirements. The aeration blowers also provide air to the aeration influent channels to keep solids in suspension.

**Table 4.2-4 Aeration Blower Design Data**

| Design Element | Value |
|---|---|
| Blower Type | Single stage centrifugal (integrally geared) |
| Number of Units | 3 |
| Capacity, each | 10,900 cfm |
| Motor Size, each | 600 hp |
| Differential Pressures | 9.8 psig |
| Maximum Air temp. At Relative Humidity | 100 deg. F; 90% |
| Minimum Turndown Capability | 100 to 46% |

These blowers have been considered reliable by operations staff, however it is difficult to assess blower performance based on the lack of operating data (e.g. airflow measurements and pressure readings). There is no automated control of the air supply to the aeration tanks. The air flow to the basins is maintained constant and there are no functioning air flow meters on any of the aeration piping. DO control is critical to the performance of an activated sludge process designed for BNR, so it is very uncommon for a BNR plant of this size to rely solely on manual operations.

### 4.2.4.3 Secondary Clarifiers

The three rectangular, dual channeled secondary settling tanks are each 325-ft long and 50-ft wide with a center sludge collection trough. The existing tanks were built as part of the 1969 upgrade making them over 50 years old. Three influent distribution channels convey mixed liquor from a pair of aeration tanks to a dedicated secondary clarifier and RAS is returned to the same pair of aeration tanks thus creating three independent parallel trains. When one clarifier is out of service, aeration basin effluent may be redirected by slide gates located along the distribution channels. The clarifiers operate with a sidewater depth of 11 feet, which is below the recommended design minimum of 12 feet from TR-16 and have a combined surface area of 48,750 square feet[2].

At the secondary system's capacity (or maximum daily flow) of 58 mgd, the maximum allowable MLSS to be maintained within the secondary process is limited to 2,500 mg/L (with a safety factor of 1.3 applied).

The sludge and scum collection mechanisms were replaced as part of the 1992 project, so this equipment is nearly 30 years old. Non-metallic chain and flight systems scrape settled sludge into the center sludge trough and cross collectors direct sludge to the sludge hopper. Each tank has two 4-shaft longitudinal collectors at the front end of the tank and two 3-shaft longitudinal collectors at the back end, and one 3-shaft cross collector in the center trough. The 4-shaft front end collectors' return path moves floating scum from the center of the tanks to a skimmings weir at the influent end. Periodically, scum gates are manually opened to convey scum into a collection trough and out to the secondary scum pit. There is typically significant surface scum accumulation along the effluent troughs. This buildup of solids contributes to high $BOD_5$ and TSS in the chlorine contact tanks and plant effluent. From the sludge hopper, the sludge is returned to the aeration tanks or wasted to the mixed sludge wet well. Treated secondary effluent is discharged to the chlorine contract tanks.

### 4.2.4.4 Return and Waste Activated Sludge Pumps

The RAS pumps were installed during the 1992 treatment plant upgrade, are nearly 30 years old, and are nearing the end of their useful lives. These pumps convey settled sludge from the final settling tanks to the head end of the aeration tanks to maintain the desired MLSS concentration for biological treatment. Until recently, WAS was wasted directly from the RAS line into GT tank #1 using the head of the RAS pumps. A new rotary drum thickener was put into operation in 2020 and is now used to thicken WAS.

There are six 75 hp returned activated sludge pumps – two RAS pumps are provided for each final settling tank. One operates as a duty pump and the other acts as a standby pump. These pumps are located in the secondary gallery below grade adjacent to eastern side of the three final settling tanks.

### 4.2.4.5 Secondary Scum

The secondary scum equipment and conveyance piping was upgraded during the 1992 facility upgrade.  Skimming flights push the scum towards the influent side of the rectangular final settling tanks. The scum is pushed over a fiberglass weir, into a scum weir trough. From this point, six scum gates (one for each final tank bay) open to allow the scum pass into a scum collection trough and then to the secondary scum pit. The collected scum can be decanted and removed from the site by a vacuum truck or can be recirculated to the primary influent channel.

### 4.2.4.6 Assessment and Deficiencies – Secondary Treatment

The West Side WWTP does not have adequate aeration basin volume to reliably maintain nitrification through winter months at present-day primary effluent flows and loads, much less, future conditions. The conversion to an MLE process was designed as an interim solution to impart BNR with limited investment.

The pre-anoxic reactor is not ideally configured, as one single pre-anoxic zone per tank. There are no baffles to create smaller, compartmentalized zones which would create a F:M gradient that would inhibit the growth of filamentous organisms within the downstream aeration reactors.

In addition, the mixers and IR pumps have experienced problems due to chronic ragging, based on inefficient upstream systems, which results in equipment out of service for extended periods of time. The BNR mechanical equipment is nearly 20 years old and has reached the end of its useful life.

The aeration blowers are currently more than 20-years old and should be replaced with more appropriately sized, efficient, modern blowers. Aeration piping has reached the end of its useful life. No operable automated blower control strategy exists resulting in over-aerating, return of DO to the anoxic zone in the IR piping, and wasted energy.

The secondary system operates at a much higher MLSS concentration to facilitate BNR within the bioreactors, leaving the secondary clarifiers susceptible to solids washout at higher flows through secondary system.

The secondary clarifier sludge and scum collection equipment is also past its useful life. The RAS pumps and associated piping and valving is in poor condition and should also be replaced. There is no accurate method of quantifying the amount of sludge that is wasted, which is a critical factor for the operation of any BNR WWTP. Significant surface scum accumulation exists along the effluent troughs – in part due to poor scum removal in the primary tanks and in part due to the practice of recycling secondary scum to the primary tanks. This buildup of scum contributes to high $BOD_5$ and TSS in the plant effluent during high flow events.

From a resiliency standpoint, aeration blowers and associated motor control centers (MCCs), located in the control building, are installed more than one foot below the elevation required for flood protection. The secondary gallery, housing the RAS pumps, is also currently susceptible to flooding. The access elevation to the secondary gallery is approximately 5.5 feet below the design flood conditions.

The roof of the secondary gallery has recently experienced leaking. The seals have failed on the access hatches, located directly above the RAS pumps resulting in leaks during rain events.

#### 4.2.4.7 Recommendations – Secondary Treatment

The secondary system in its current state is undersized and not appropriately designed to consistently achieve low level nitrogen removal year-round, and most equipment and controls are beyond their useful life. The system must be upgraded and expanded to enable year-round nitrogen removal and improve operational efficiency or be replaced with advanced treatment system.

### 4.2.5 Disinfection

The West Side WWTP uses sodium hypochlorite for disinfection and sodium bisulfite for dechlorination. The plant has three identical chlorine contact tanks, each with two passes. The tanks were originally constructed in 1951 as primary clarifiers. In 1969, they were converted to chlorine contact tanks when new primary clarifiers were constructed. Since the tanks were not originally constructed as chlorine contact tanks, the tank configuration is not optimized for disinfection, which can result in short circuiting and excessive chemical use. Each pass is 210-ft long by 20-ft wide and the side water depth is approximately 12-ft, resulting in a length to width ratio of 21 to 1. This is about half of the TR-16 recommendation of a minimum length to width ratio of 40 to 1. Each tank has a total volume of approximately 740,000 gallons resulting in detention time in excess of what is required. Effluent from the chlorine contact tanks discharges into an effluent channel which connects to the 72-inch plant outfall pipe.

The chlorine contact tanks receive all plant flow including secondary effluent and bypassed primary effluent (during wet weather flows). At the design average daily flow of 30 mgd, the chlorine contact tanks provide 35 minutes of detention time with only one tank in service. At the plant capacity of 90 mgd, the chlorine contact tanks provide 35 minutes of detention time with all three tanks in service. Typically, all three chlorine contact tanks are in operation, resulting in excessive detention time under average daily flows, which can result in settling solids settling, inadequate mixing which can impart dead zones in the tanks and cause regrowth.

Despite providing the required contact time for disinfection, the plant has had trouble meeting their National Pollutant Discharge Elimination System (NPDES) permit limits for disinfection, since the tanks are not properly configured to avoid short-circuiting. The enterococci monthly geometric mean has often exceeded the permit limit including an eight month stretch in 2018 of violations with two of those months exceeding 200 per 100 milliliter (mL). Fecal coliform and total residual chlorine limits have not been violated over the past three years.

All chlorine contact tanks are aerated with a single 30 hp multistage centrifugal blower located in a fiberglass enclosure at the east end of the chlorine contact tanks. The aeration system is used to help with chlorine mixing and to keep any remaining solids in suspension. Aerating chlorine contact tanks is not typical in wastewater treatment.

### 4.2.5.1 Sodium Hypochlorite Storage and Feed System

The chlorination system was originally designed for use of chlorine gas; however, the gaseous system was converted to sodium hypochlorite in 1999 under the "West and East Side Wastewater Treatment Plants Hypochlorination System" project.

The system has five, 3,000-gallon crosslinked polyethylene tanks for a total storage capacity of 15,000 gallons providing approximately 15 days of storage. The tanks are equipped with a leak detection system and are located in an addition to the influent pump station building with a block containment wall to contain spills. The plant currently receives deliveries of 12.5 to 15.6% sodium hypochlorite, by weight.

Three, JCS vacuum injectors distribute the sodium hypochlorite to multiple application points throughout the plant. Sodium hypochlorite can be injected into the following locations (1) into the common secondary clarifier effluent, (2) into the bypassed primary effluent channel upstream of the chlorine contact tanks, (3) into the raw sewage influent for odor control and (4) into the RAS piping for bulking control.

### 4.2.5.2 Dechlorination Storage and Feed System

The plant has an effluent chlorine residual maximum daily limit of 0.1 mg/L and an average monthly limit of 0.05 mg/L. To achieve these limits, the plant dechlorinates using sodium bisulfite. A storage and feed system installed in 2004. The chemical is stored in two, 2,000-gallon storage tanks and is conveyed to the effluent via four variable rate dosers operating in two duty/two standby configuration. The rate of dosing is controlled by the influent flow rate. Sodium bisulfite is discharged into the downstream end of the chlorine contact effluent channel, after the chlorine contact tank weirs, with two induction mixers. While the reaction between chlorine and sodium bisulfite is considered instantaneous, the mixers help to distribute the chemical throughout the effluent flow. A chlorine residual analyzer is located downstream of the sodium bisulfite application point to confirm compliance with the permit.

Assuming a maximum of 2 mg/L of chlorine residual, the existing storage tanks provide 21 days of storage. This is not sufficient per the TR-16 requirement of 30 days minimum storage.

### 4.2.5.3 Plant Water System

Chlorinated effluent from the chlorine contact tanks is pumped to a piping loop to deliver non-potable water for various plant services. Non-potable water is used for the tank and equipment washdown, the chlorine injectors, and the odor control system as scrubbing water.

The plant water system consists of three, centrifugal pumps, located in the Degritter Building, each have a maximum capacity of 2,800 gpm at 80 ft TDH and are powered by 75 hp motors through a VFD. Three high-pressure effluent water pumps, installed in the secondary pipe gallery, raise the effluent water pressure to deliver to the chlorine injectors.

### 4.2.5.4 Assessment and Deficiencies - Disinfection

The fatal flaw with the existing chlorine contact tanks is the elevation of the tanks and their susceptibility to flooding. The top of the chlorine contact tank walls is at elevation 27.00 which is just above the current 100-year flood plain elevation of 26.60, and the weir elevation is 22.75 which is below even the 25-year flood elevation. As a critical process TR-16 and CT DEEP require that the unit process remain fully operational under a 100-year flood event and be protected against 100-year plus three feet (elevation 29.60). The existing facilities cannot meet this design criteria. The sodium hypochlorite storage and feed system including tanks and dosing units are also below this design flood elevation. Because of this the existing system cannot be re-used in its current configuration.

As with other mechanical systems on-site it is assumed that the plant water system is well past useful life and should be replaced as part of the plant upgrade.

### 4.2.5.5 Recommendations – Disinfection

Because of the elevation of the chlorine contact tanks, and the age of the existing chemical storage and feed systems, and the inappropriate sizing of the existing tanks, the current disinfection system must be replaced with a new system at a higher elevation. If disinfection with sodium hypochlorite is maintained, the dechlorination system may be salvageable.

## 4.2.6 Effluent Outfall

A 72-inch reinforced concrete pipe (RCP) conveys disinfected plant effluent from the chlorine contact tanks to the discharge point in Cedar Creek (Black Rock Harbor). The outfall pipe is a straight alignment, approximately 800 feet in length from the contact tanks to the discharge point. There are two major junction chambers along the outfall route, as well as several manholes for access to the pipe.

Record plans indicate that the entire length of the outfall pipe was constructed in 1948, making it over 70 years old. The outfall pipe, with an invert elevation of 3.0, is completely submerged under most tidal conditions. The outfall's invert at its terminus is approximately 7.6 feet below Mean Lower Low Water (MLLW) zero.

Approximate velocity through the effluent outfall is 1.6 feet per second (fps) under design average flow conditions and 4.9 fps at current peak plant capacity of 90 mgd.

### 4.2.6.1 Outfall Inspection

On January 30, 2020, the outfall pipe was inspected via underwater inspection by ASI Marine using a small remotely operated vehicle (ROV). The ROV was equipped with a high-definition camera, imaging sonar, and profiling sonar. A complete report of the inspection work is included as **Appendix E**. The overall observations and findings are presented below:

- Joints appeared to be intact without signs of separation or misalignment.
- Debris and sediment piles were observed throughout the pipe. Larger pieces of rocks and debris were noted throughout the pipe. Trash was visible at different points along the outfall pipe, including tangled rubber tubing. A maximum debris depth of 16.5 inches was noted on the sonar profiles included in the report.
- Concrete aggregate was visible within the video footage at some points. There were some noticeable unknown protrusions that could be seen in the concrete.
- Two openings in the crown of the pipe were observed, these are believed to be "break-in" access manholes along the outfall pipe.
- A metal band for a flow meter was observed in the outfall pipe downstream of the chlorine contact tanks. This flow meter is not operational.

### 4.2.6.2 Assessment and Deficiencies – Effluent Outfall

Overall, the West Side WWTP outfall pipe is aging, but appears intact. Although no structural testing was conducted, no major defects were observed in the outfall inspection. Debris and sediment accumulation was observed throughout the outfall pipe, as well as larger pieces of rock, concrete, and trash.

### 4.2.6.3 Recommendations – Effluent Outfall

If maintained, it is recommended that the existing outfall be cleaned and lined to extend the life of the outfall pipe.

## 4.2.7 Sludge Processing and Disposal

Over the years, the West Side plant has implemented a number of solids handling technologies. In 2018 the building that housed the former gravity belt thickener (GBT) was condemned due to structural concerns. Having lost the ability to mechanically thicken WAS, the contract operator made temporary piping modifications so that WAS and primary sludge could be co-thickened in GT No. 1 while primary sludge only continued to be thickened in GT No. 2. The combined thickened sludge was hauled directly from the GTs at an average solids concentration of 5%.

In 2020, the contract operator installed a new dual drum rotary drum thickener (RDT) system and truck loading pumps in the lower level of the Sludge Disposal Building as an interim means to improve sludge processing capabilities.

### 4.2.7.1 Gravity Thickeners

The two GTs are located between the primary settling tanks and the chlorine contact tanks. The GTs were constructed during the 1969 upgrade and the Gravity Thickener Building was partially rehabilitated during the 1992 upgrade. Design details are presented in **Table 4.2-4.**

**Table 4.2-5 Gravity Thickener Design Information, West**

| Gravity Thickener Information | |
|---|---|
| Number | 2 |
| Diameter (ft) | 50 |
| Side Water Depth (ft) | 12.5 |
| Surface Area ($ft^2$), each | 1,960 |
| Volume (gal), each | 183,500 |
| Design Overflow Rate (gpd/sf)[1] | 473 gpd/sf |
| Sludge Received | Primary Sludge and WAS |
| Solids Loading Rate (lb/sf/day)[1] | 23-26 lb/sf/day |

[1]Assumes Both Thickener Tanks in Service

Prior to the new RDT WAS thickening system put into service in 2020, WAS was directed to GT No. 1 and primary sludge was directed to GT No. 2.

The GTs were designed to receive degritted primary sludge from the mixed sludge wet well in the Degritter Building. The mixed sludge wet well was intended to provide a means to convey WAS to the gravity thickeners if the GBT was out of service. Reportedly, the mixed sludge pumps frequently clogged. A gravity mixed sludge line was subsequently installed (approximately 10 years ago) directly from the primary sludge degritters to the influent channel of the GTs. This gravity pipe has reportedly worked well. This gravity line can be seen in **Figure 4.2-6**. The temporarily WAS pipe to GT tank #1 for the period between 2018 and 2020 for co-thickening with the primary sludge can be seen in **Figure 4.2-7.**

The GT tanks each have a center drive mechanism that moves a rake arm and scraper around the tank. The rake moves the sludge towards the center of the tank for withdrawal and releases gases from the sludge to improve thickening. The thickened sludge at the bottom of each tank is pushed to a center hopper by the squeegee blade scrapers.

A vault with a supernatant pump once conveyed supernatant flow to the primary effluent channel, but the pump was problematic and was taken out of service in November of 2018. Since November 2018, supernatant flow has been conveyed by gravity to the plant headworks ahead of influent flow metering but after influent flow sampling. This recirculated flow is estimated to be an average of 1.5 to 2.0 mgd based on the MOR data from 2017 through 2019.

**Figure 4.2-6**
**Gravity Thickener Tank #1 (temporary WAS pipe on walkway)**

**Figure 4.2-7**
**Gravity Thickener Tank #2 and Mixed Sludge Gravity Line**

The GTs at the West Side Plant are uncovered with no associated odor control system. There have been no known reported issues of excessive odors from the thickeners. The mechanisms on both of the GTs have reached their useful life. The concrete, over 50 years old, appears to be in fair condition but is showing some cracking and wear.

The GTs currently have no operable scum collection or removal system. According to the original design drawings the thickeners were designed with scum beaches and scum drains that discharged by gravity to the scum handling pit. This scum pit has been abandoned. It does not appear that any of the conveyance piping from the GT tanks to the scum pit is still viable. It appears that scum accumulates on top of the tanks and leaves the tanks with the supernatant flow.

### 4.2.7.2 Rotary Drum Thickener (RDT)

The contract operator has recently (summer 2020) rehabilitated the Sludge Storage Tanks and installed thickening/pumping equipment in the lower level of the partially demolished Sludge Disposal Building. The temporary thickening of WAS in GT No.1 is no longer practiced and the West Side WWTP has returned to mechanical WAS thickening and co-storing thickened sludge (WAS and Primary) in the Sludge Tanks. Combined thickened sludge is then pumped and hauled for disposal. The packaged thickening system includes a dual drum RDT, WAS feed pump, washwater booster pump, polymer system and associated electrical, instrumentation and control panels. The RDT is manufactured by BDP Industries and sized to thicken up to 250 gpm of WAS with 1.0 to 1.5% solids content to a final solids content of up to 7%. The sludge feed pump is a Borger rotary lobe pump capable of pumping 415 gpm at 30 pounds per square inch (psi). This pump, equipped with a magnetic flow meter, is used to control the WAS sludge feed rate and controls polymer dosing. The thickened sludge is then pumped into the Sludge Storage Tanks by a 6-inch double disc Penn Valley Pump. The skid mounted equipment, prior to startup/connection is shown in **Figure 4.2-8**.

**Figure 4.2-8**
**Rotary Drum Thickener Skid**

### 4.2.7.3 Sludge Hauling

Prior to the recent installation of the RDT and rehabilitation of the sludge tanks and hauling systems, the thickened combined sludge was pumped from the GTs into tanker trucks to be hauled off-site for disposal. Inframark has a contract with Synagro Technologies for sludge hauling and disposal for thickened sludge with solids concentrations greater than 2%. Synagro will accept thinner sludge at a different rate and reserves the right to impose additional demurrage charges when the time to fill a truck exceeds one hour.

The West Side WWTP fills 275 truckloads per month averaging 1.2 tons (dry solids) per load. During 2019, the plant almost never produced thickened sludge that contained less than 2% solids. Demurrage charges at the West Side are quite common totaling an average of $7,500 per month.

Prior to the installation of the RDT, the sludge tanker fill station was located adjacent to the GTs. The fill station consisted of a 6-in heavy duty hose with two, manual knife gate valves and a 6-in flexible fill hose. This location did not have a contained drainage area. There is tank drain manhole in the vicinity that is currently used to catch spills. The hose and valves are not heat traced and operators note that continuous flow is required to prevent freezing in the winter months. Truck operators pull up to the station, connect the fill hose to the truck and open the valve. A plant operator then starts the Truck Loading Pump and monitors the filling process throughout. After the truck is full the operator records the volume received.

Subsequent to the recent improvements, primary sludge is thickened in the GTs, and WAS is now mechanically thickened with the new RDT. Both thickened primary sludge and thickened WAS are conveyed to the thickened sludge storage tanks in the sludge processing building. Trucks are now filled from the sludge storage tanks rather than directly from the GTs. Current truck loading is provided by a newly installed 8-inch duplex model double disc Penn Valley Pump in the lower level of the Sludge Building. A demurrage charge is applied every 30 minutes over the one-hour loading limit. The pump is manually controlled through a remote control panel located adjacent to the truck fill station. There is no local control at the pump.

#### 4.2.7.4 Assessment and Deficiencies – Sludge Processing and Disposal

In recent years, the ability to appropriately manage solids generated on-site has impacted the overall performance of the WWTP. The inability to adequately thicken sludge and then quickly load trucks has resulted in solids being recirculating throughout the system. At this point, the interim solution of installing an RDT for WAS thickening and rehabilitating the thickened sludge storage tanks and truck loading pumps could improve the situation, however no redundant thickening or pumping equipment has been provided.

The Sludge Building, constructed prior to 1950, has been partially demolished due to structural concerns and largely abandoned. Nearly all of the supporting building mechanical equipment (HVAC, electrical, structural, etc.) has been removed along with the superstructure. The sludge storage tanks located here have been returned to service. The new RDT skid, thickened sludge pump and truck loading pump are also located in the partially demolished Sludge Building.

The GTs are over 50 years old but appear to be in fair condition structurally, exhibiting some cracking and concrete deterioration. The GT equipment in GT No. 1 was recently replaced but the remainder of the equipment is nearly 30 years old and has reached the end of its useful life. Furthermore, the GTs are not sized adequately sized for co-settling primary and WAS.

The entrance to the GT building is located at 25.5 feet in elevation, and the floor of the GT gallery is located at approximately 13 feet in elevation making this area vulnerable in a flood event. The partially demolished Sludge Building is also within the 100-year floodplain.

### 4.2.7.5 Recommendations – Sludge Processing and Disposal

The existing sludge processing and disposal system is inadequate and is in need of replacement, to upgrade existing equipment and to provide permanent, reliable solution to sludge management. The newly installed RDT should be relocated and reused as part of the upgrade.

## 4.2.8 Odor Control

The existing odor control system consists of four, vertical countercurrent flow scrubbers to treat hydrogen sulfide ($H_2S$) and other gases in the air ventilated from several locations of the treatment plant. The screening room in the Screening Building and the sludge Degritter Building each have one dedicated scrubber, while the bypass screen room and wet well serving the influent pump station have two dedicated scrubbers.

Each scrubber was designed for 10 feet of packing creating a gas/liquid interphase between the influent air and the scrubbing water. The scrubbing liquid was created by mixing service water with solutions of 15% sodium hypochlorite and 50% sodium hydroxide, which was sprayed from a nozzle at the top of the device to absorb $H_2S$ and other gases from the odorous air stream. Each scrubber location has two recirculating pumps to cycle scrubbing liquid that accumulates in the sump of the scrubber back to the spray nozzle. Plant water was added to the cycling scrubbing liquid at a rate not to exceed 10% of the recycle rate through the scrubber, while excess waste scrubbing liquid was directed through an overflow connection in the sump and flows to the plant drain. The addition of hypochlorite and hydroxide was automated based on the oxidation-reduction potential (ORP) and pH, respectively, of the cycling scrubbing liquid, monitored by probes within the scrubber.

In addition to the air scrubbing system, the West Side WWTP has one, 55-gallon Air Phase Carbon drum to treat odorous air from the septage receiving area. The odorous air is drawn to the unit by an exhaust fan, where it passes through the carbon bed and then exits the drum. The removal efficiency of the drum will decline over time, and spent drums are disposed and replaced by maintenance personnel.

### 4.2.8.1 Assessment and Deficiencies – Odor Control

The plant no longer uses sodium hypochlorite and sodium hydroxide to treat effluent air. Presently, the WWTP pumps chlorinated process water from the chlorine contact tanks to the scrubbers. This system has been relatively effective at removing odors from the airstream and eliminates the need for pH adjustment. Since discontinuation of the chemical feed system for the odor control system, the chemical feed pumps and storage tanks have been abandoned in place.

On numerous occasions, during plant site visits, the $H_2S$ monitor was activated which indicates that un-safe levels of $H_2S$ are present in the area and the ventilation in these process areas in inadequate.

#### 4.2.8.2 Recommendations – Odor Control

Due to the age and condition of the existing odor control systems, it is recommended that these systems be removed and replaced with systems designed to manage odorous air from newly designed unit processes, namely preliminary treatment (headworks areas), primary treatment, and sludge processing areas.

### 4.2.9 Structural

The oldest structures on site are the main pump station, the existing chlorine contact tanks, (the original primary settling tanks), and the Sludge Building. The existing primary settling tanks, bioreactors, secondary clarifiers, and GTs were constructed in the 1969 upgrade. Since then only a few minor structures have been added to the site for chemical storage.

The general deficiencies with the structures observed to date include: deteriorated concrete with exposed aggregate in some location; concrete spalling and exposed, rusted rebar in some locations; and leaky roof hatches in the gallery.

### 4.2.10 Architectural

From an architectural standpoint, in general, many of the buildings are in poor condition and exhibit deficiencies, vulnerabilities and code compliance issues. Architectural issues include:

- Damaged, missing and non-compliant railings throughout the plant site presenting a life safety issue;
- Many stairs missing non-slip embedded nosing; emergency exit door hardware non-compliant;
- Many doors and door frames corroded; signs of water leakage, moisture buildup and condensation on ceiling tiles; some bathrooms inoperable or non-compliant;
- The roof areas that were accessible were in poor condition and would require replacement;
- Lab equipment and casework in need of repair.

### 4.2.11 HVAC, Plumbing, and Fire Protection

In general, most heating, ventilation, and air conditioning (HVAC) equipment at the West Side WWTP is past its useful life, in poor condition, and/or inoperable. This includes exhaust fans, air handling units, unit heaters and duct work. Roof mounted units are located too close to the roof edge without proper fall protection and electrical components located in hazardous areas are not explosion proof. The exceptions to this are the boiler systems which appear to be in good condition.

With respect to plumbing and fire protection, the following deficiencies were noted:

- The lack of proper fire protection systems is a life safety issue and needs to be addressed. Storage of chemicals, lubricants, waste oils and yard equipment without proper fire protection is a violation of code.

- Cross connections on the water system were noted throughout the buildings on site.
- Many emergency eyewash and showers were not functional.
- Other deficiencies include inadequate or not existent floor drains, no hose bibs for washdown, inadequate number or bathroom facilities and/or water supplies shut off to service sinks and bathrooms.

As part of the overall WWTP upgrades, the entire HVAC and plumbing systems should be upgraded to meet current codes and standards.

## 4.2.12 Electrical

The majority of the electrical distribution equipment was installed as part of the plant upgrade in the 1990s and was noted during the site visit to be in fair to poor physical condition. The average useful life of electrical equipment, including cables, is considered to be approximately 25 to 30 years. Therefore, many of the components (i.e. circuit breakers, contactors, transformers, etc.) of this equipment may be worn beyond the manufacturers' recommended limits. It is not known whether the main switchgear and motor control centers (MCCs) are tested and maintained annually as there were no maintenance stickers on any of the equipment. It is not unreasonable to expect an increase of failures above normal levels of maintenance as this equipment is at, or has exceeded, its anticipated useful life cycle.

### 4.2.12.1 Electrical Service

Electrical service is provided via an overhead primary line at 13.8 kilovolt (kV) to a riser pole located across the street on Bostwick Avenue near the marina. From the riser pole, the primary 13.8 kV service is routed, via an underground duct bank system, to a 15 kV Outdoor Load Interrupter Switchgear, located in the grass area southeast of the bioreactors. The utility revenue metering is provided at the 15 kV Outdoor Load Interrupter Switchgear. The switchgear was installed in the 1990s and is nearing the end of its useful life.

From the 15 kV Outdoor Load Interrupter Switchgear, 13.8 kV is distributed via underground duct banks and manholes to two, outdoor 13.8kV-480/277-volt (V) units substations. The double ended unit substations are outdoor walk-in type with 4,000 Amp main-tie-main 480-volt switchgear located in the walk-in enclosure between the two outdoor transformers. The mains and tie circuit breakers are key interlocked to allow only two out of the three circuit breakers to be closed at the same time. The unit substation ratings are as follows:

- Substation No.1 is a 3,000-kilovolt-ampere (kVA) double ended unit substation located at the back of the facility south of the bioreactors.

- Substation No.2 is a 3,000-kVA double ended unit substation located in front of the Raw Sewage Pump Station.

From the substations, power is distributed to double ended MCCs with main-tie-main key interlocked circuit breakers enabling vital components of the same type and serving the same function to be divided as equally as possible between at least two motor control centers, as required by EPA-430.

### 4.2.12.2 Standby Power

Standby power is supplied to the electrical distribution system through two, low voltage (480 V) diesel engine generators. The standby generators are located in proximity to Substation No.2 in front of the Raw Sewage Pump Station. The units were installed as part of the 1990s upgrade and were observed to be in good condition, visually. The generators are each rated 1,500 kW, 1,875 kVA, 3 phase, 4 wire, 0.8 power factor (PF). These generators distribute standby power, through paralleling switchgear, and a 4000 Amp automatic transfer switch (ATS) to Bus B of Switchgear No.2. It appears, based upon CDM Smith's preliminary review of the existing one-line diagrams, that critical process equipment is supplied with sufficient standby power to maintain treatment during utility outages.

The overall distribution system provides a reliable electrical distribution system that allows for flexibility in O&M, and in theory, is compliant with EPA 430-99-74-001, which requires two separate and independent sources of electric power to be provided to the works from either two separate utility substations or from a single substation and a works-based generator system. The 15 kV Outdoor Load Interrupter Switchgear has a single utility service; however, it appears that critical process equipment is supplied with sufficient standby power to maintain treatment during utility outages, from the standby generator. If the existing standby generator is adequately sized and automatically provides standby power to all critical process equipment, then the system is EPA 430 complaint and is not susceptible to common mode failures.  The term "common mode failure" is an industry term used to identify a point in the electrical distribution system where a single failure will result in the complete loss of critical process or pumping systems.

### 4.2.12.3 NFPA 820 Review

NFPA Standard 820, "*Standard for Fire Protection in Wastewater Treatment and Collection Facilities*" (2008 Edition), establishes the requirements for protections against fire and explosion hazards in WWTPs. The standard identifies the various areas of a WWTP based on the process and defines the potential hazard and the required electrical area classification. Additional requirements for fire protection (i.e. fire detection/fire alarm systems, combustible gas detection, etc.) are also dictated by this standard.

Electrical equipment installed in hazardous areas classified as Class I, Division 1 or Class I, Division 2, is required to be rated National Electrical Manufacturers Association (NEMA) 7, explosion proof. NFPA 820 does allow a declassification of Class I, Division 1 spaces to a Class I, Division 2 space.  In order to declassify this type of area, continuous ventilation at 12 air changes per hour needs to be provided.  Declassifying the area would still require NEMA 7, explosion proof electrical equipment to be provided.

Additionally, NFPA 820 does allow a declassification of Class I, Division 2 spaces to an unclassified location. In order to declassify this area, continuous ventilation at six air changes per hour needs to be provided. Declassifying the area by ventilation would allow the use of unclassified, standard electrical equipment to be installed in the space.

Due to the nature of some of the spaces in the existing plant, continuous ventilation to declassify the space is required in order to allow the installation of electrical equipment including switchgear, MCCs, and VFDs in existing spaces.

Many buildings' compliance with NFPA 820 are in question. Continuous ventilation at specific rates and combustible gas detection is required along with an alarming system to declassify a space and allow general duty electrical equipment in these locations. Where declassification is required by code, the recommendation shall include HVAC upgrades, gas detection upgrades, and supervised alarm systems with notification appliances.

In addition, there are numerous electrical rooms that are open, through a corridor, to hazardous classified areas per NFPA 820. The hazardous classification extends to all common or adjacent areas unless physically separated. Per NFPA 820 the definition of physically separated is as follows: a gastight partition between two adjacent spaces, or two nonadjacent spaces, with no means of gas communication between the spaces and where personnel entry into the spaces is by individual, exterior access ports with no physical connection, or an airlock.

### 4.2.12.4 Arc Flash Labeling

The existing equipment does not include arc flash labeling on equipment.

### 4.2.12.5 Lighting

The majority of the lighting fixtures throughout the WWTP, including in the process areas and gallery areas, are either energy efficient T-8 or high bay high pressure sodium fixtures. There are many areas with multiple fixtures that are inoperable.

Emergency battery powered egress lighting was noted throughout the facilities, including within the galleries. However, the operability of the egress lighting was not observed and the fixtures appear to be original.

Exterior lighting appeared to be adequate; however, the operation of the exterior lighting was not observed during the site visit.

### 4.2.12.6 Fire Alarm System

It was noted by plant staff that the existing plant fire alarm system is not operational. Per NFPA 820 and the building code there are numerous areas within the plant where a fire alarm system is required to meet current codes.

### 4.2.12.7 Page Party System

There is an existing page party system; however, it was noted by plant personnel that much of the system is currently not functional.

#### 4.2.12.8 Security System

The existing plant has numerous security or process monitoring cameras; however, it was noted by plant personnel that many of these cameras are not operational. The facility includes card access at numerous buildings; however, it was noted by plant personnel that much of this system is inoperable.

As part of the overall plant upgrade the entire electrical system should be upgraded to meet current codes and standards.

### 4.2.13 Instrumentation and Controls

In general, most of the instrumentation and controls (I&C) around the plant site is past its useful life, dysfunctional and/or obsolete. With respect to the general I&C systems the following deficiencies were noted:

- Buildings throughout the plant are connected via 10Base2/T, known as Thinnet. The physical cabling is RG-58 coaxial with Bayonet Neill-Concelman (BNC) connections. Thinnet has been considered obsolete technology since the release of 10Base-T in the early 1990s.
- Security throughout the plant includes security cameras mounted on multiple buildings. The cameras can be monitored by several televisions in the Control Building's Control Room, but it was noted that not all cameras were functional.
- Programmable logic controllers (PLCs) throughout the plant are GE 90-30. The 90-30 product line was discontinued by GE in 2018, and the GE PLC product line was acquired by Emerson in 2019. The GE 90-30 PLCs were installed to serve as a replacement for previously discontinued PLCs in the plant, including Bailey INFI 90 PLCs in the Primary Control Unit (PCU) racks and Control Building Control Room, and SquareD Sy/MAX PLCs installed in various MCC PLC enclosures throughout the plant. It is generally recommended to replace the GE 90-30 PLCs with new PLC hardware.

As part of the overall plant upgrade the entire I&C system should be upgraded to meet current codes and standards.

## 4.3 East Side Wastewater Treatment Plant

The East Side WWTP is located at 695 Seaview Avenue and discharges into Long Island Sound via Bridgeport Harbor. The East Side treatment facility was originally designed as a primary treatment facility in the 1950s and then later in 1969 was upgraded to a secondary treatment facility. A schematic of the existing facility is presented on **Figure 4.3-1.** The East Side plant is a sister plant to the West Side plant, just slightly smaller, consisting of influent screens, pumping station, three rectangular primary settling tanks, six aeration tanks, three rectangular secondary settling tanks, and chlorine contact tanks. Generally, primary sludge is thickened in the GTs and WAS is thickened with a gravity belt thickener (GBT). As with the West Side plant, a mechanical overhaul was performed in the 1995 Improvements Project. The East Side plant was later modified for nitrogen reduction in the early 2000s and dechlorination added. Limited mechanical improvements have been made since that time. An aerial view of the East Side WWTP is presented in **Figure 4.3-2**.

The East Side WWTP was originally designed to achieve secondary effluent quality at annual average design flow capacity of 10 mgd and a peak secondary treatment capacity of 24 mgd. All wet weather flows in excess of the secondary treatment capacity up to 40 mgd receives primary treatment before recombining with secondary effluent prior to effluent disinfection.

### 4.3.1 Influent Screening

Flow to the East Side WWTP collects in a manhole about 120-ft east of the Screening Building. The structure receives flow from a 54-inch sewer line, a 36-inch sewer line, and a 12-inch sewer line. Discharge from the manhole is a 60-inch pipe to the Screening Building.

The plant has two influent screens located in the Screening Building: a Main Screen and a Bypass Screen, located ahead of the influent pumps to protect them from large solids, rags, and other stringy material in the influent flow.

The two screens are climber style bar screens by FMC Corporation with a bar spacing of ¾-inch. Each screen is capable of passing the peak influent flow of 40 mgd. While the screens are the identical, they are designated as a Main Screen and a Bypass Screen. There is a Control Well No. 1 just upstream of the two screens that directs flow to the screens. Hydraulically operated sluice gates isolate the flow to the two screen channels. Normal operation is for the Main Screen to be in operation. The Bypass Screen is brought into service when maintenance or repairs are needed on the Main Screen, with flow diverted by the sluice gates. The screens are set to primarily operate automatically on a timer with a water level differential override. Each screen is equipped with a dedicated screenings washer compactor located directly below the screen discharge point to wash, dewater, and compact the screenings prior to discharge to disposal bins.

Prior to the 1995 improvement project, the channel downstream of the Main Screen served as a horizontal grit channel with longitudinal flight collection system and vertical elevator system to collect and lift grit out of the channel. The grit removal equipment was removed in the 1995 project and fill was added to give the channel its current width. To help prevent grit and other solids from setting out in this channel, it is equipped with coarse bubbler diffusers.

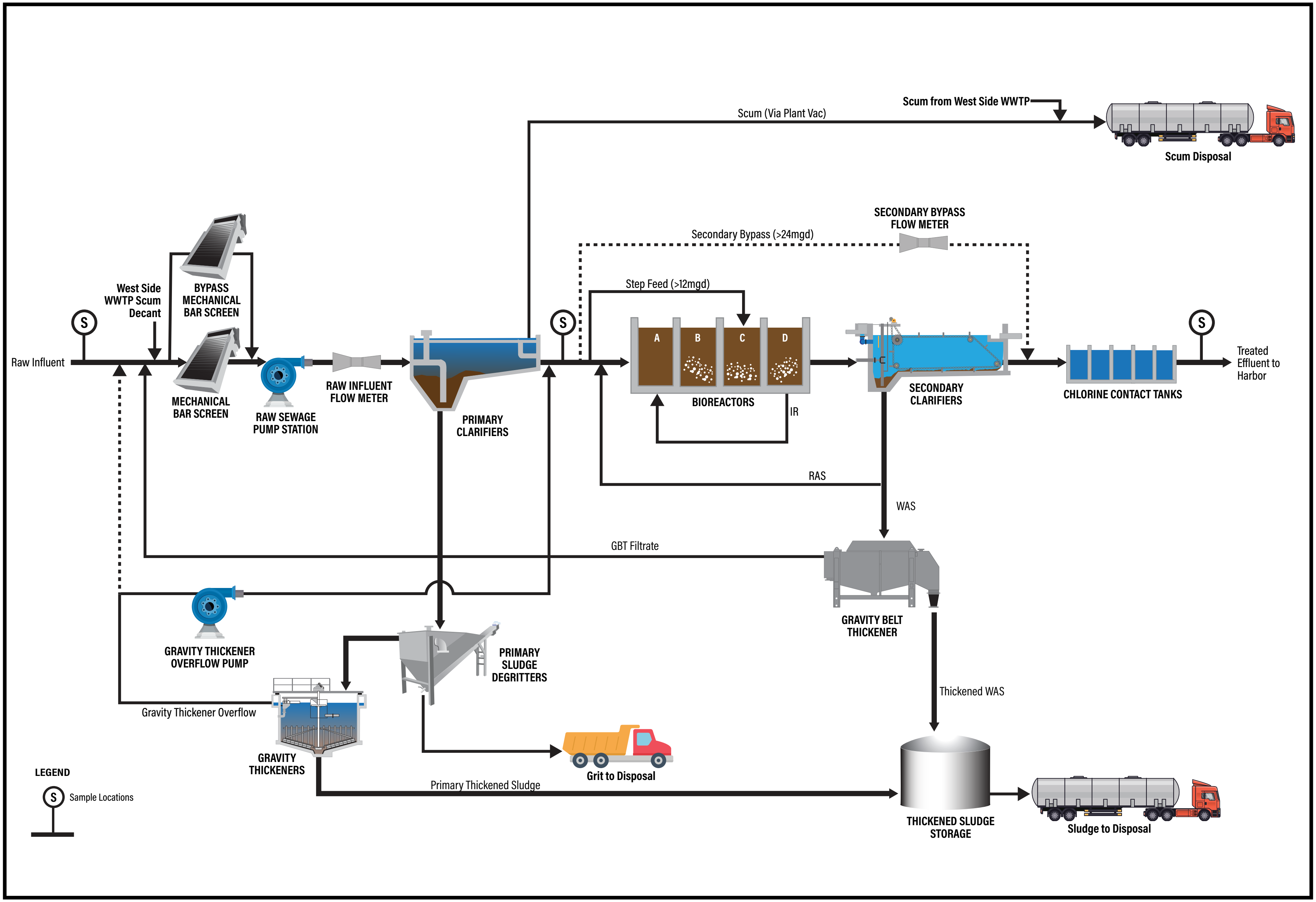

Figure 4.3-1
East Side Process Flow Diagram

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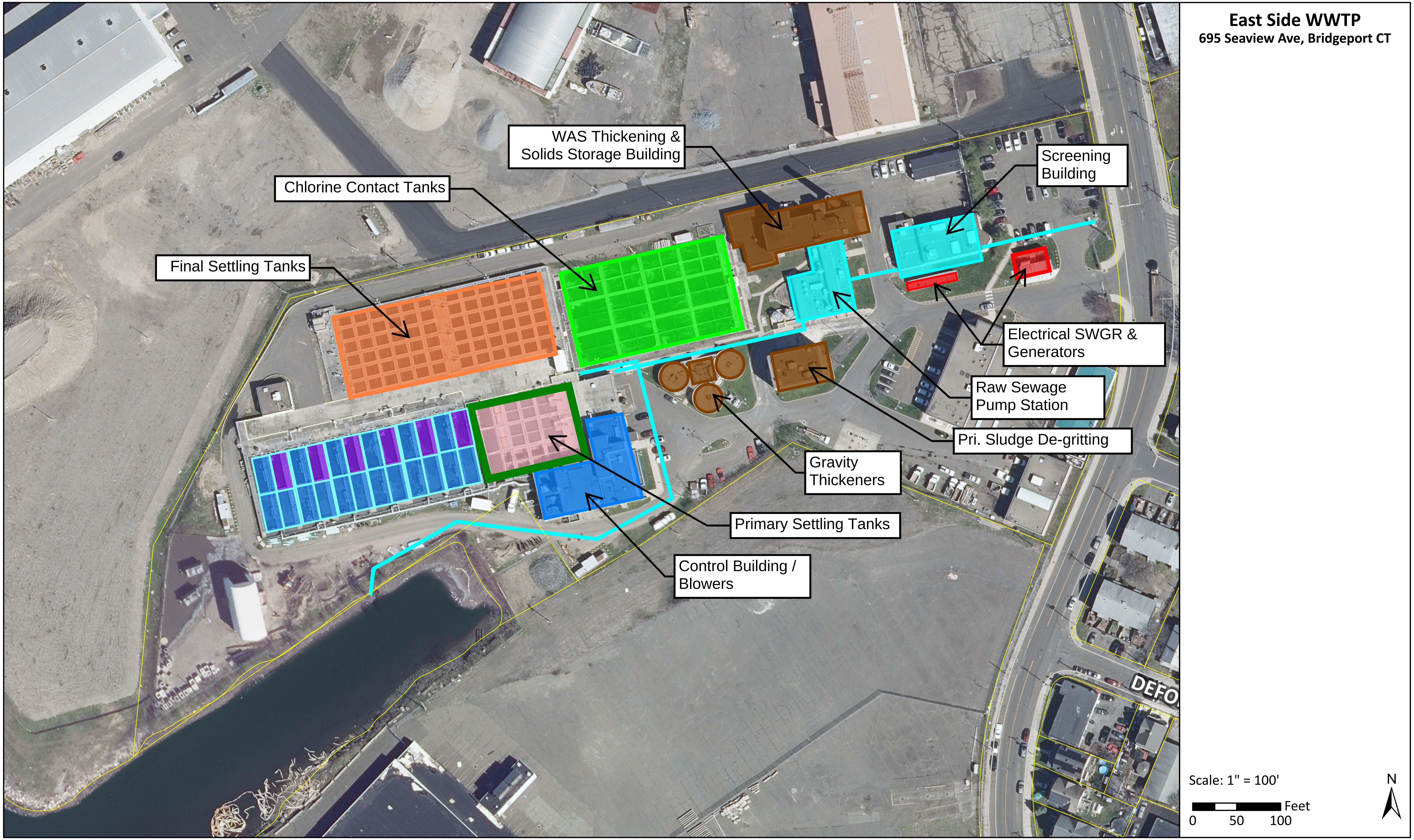

Figure 4.3-2
Existing Conditions - East Side WWTP

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There is a similar enclosed channel/box conduit downstream of the Bypass Screen, however, there is no mixing air in this channel to keep solids in suspension.

There are five, hydraulically operated sluice gates that control flow to/from the main and bypass channels; three for the Main Screen and two for the Bypass Screen. The two gates directly upstream of the screens are set up for automatic operation, while the other three gates are manually open/close service gates.

#### 4.3.1.1 Assessment and Deficiencies – Influent Screening

The Screening Building was constructed prior in the 1950s and the existing structural elements are nearly 70 years old. There are issues with concrete, metal stairs, and there is standing water in areas of the lower levels. The screens, screenings compactors, hydraulic gates and power pack, channel aeration system and blowers, and other associated mechanical equipment were installed in the 1990s and are nearly 25 years old. The overall preliminary treatment system lacks grit removal, necessitating de-gritting of the primary sludge which causes solids buildup in downstream channels that are difficult to maintain.

The Screening Building is interconnected to the Main Pump Station dry well and motor room, electrical room, sludge storage area, and GBT area through a series of doors and air locks. There are no physical barriers separating these areas, which triggers area classification concerns in the spaces between the Screening Building and the Main Pump Station per the latest NFPA 820 fire code. It also appears that the various rooms are not being ventilated properly in accordance with Chapter 9 of NFPA 820.

The loading area floor and the entrance door elevation to the Screening Building is currently susceptible and during a 100-year flood event.

#### 4.3.1.2 Recommendations – Influent Screening

Due to the age, configuration and elevation of the existing influent screening system it is recommended that the influent screening system be replaced with a modern preliminary treatment system with grit removal.

### 4.3.2 Influent Pumping

Following the influent screens, flow is conveyed to the Influent Pump Station. This station lifts flow up to the primary settling tanks to allow gravity flow to downstream treatment processes and the effluent outfall.

The original pump station structure (wet wells, dry well, and building enclosure) was constructed in the 1950s. The existing pumps were installed as part of the 1995 Rehabilitation Contract No. 2.

The station consists of four, vertical, non-clog centrifugal, single stage, end suction pumps (three duty, one standby). Each pump is sized for 9,300 gpm (approximately 13.39 mgd) at 35-ft TDH, for a total station pumping capacity of approximately 40 mgd. The pump motors are 125 hp equipped with VFDs. There is a single wet well in the station, servicing all four pumps.

The peak capacity of the WWTP is 40 mgd. The maximum pump speed is set such that this flowrate is not exceeded as measured at the Raw Sewage Parshall flume upstream primary tanks. The hydraulic influent gates upstream of the two influent screens can also be utilized to limit the plant flow to 40 mgd. If a high-level alarm is detected in the wet well, the gates will modulate to limit the flow into the plant.

#### 4.3.2.1 Assessment and Deficiencies – Influent Pumping

The Influent Pump Station building was constructed in the 1950s and the existing structural elements are nearly 70 years old. There are issues with concrete, metal stairs, corrosion on metal floors, decks, and support beams, and there is standing water in areas of the lower levels, deteriorating concrete walls and columns, and concerns with the pump shaft support beams. The pumps (three of the four), discharge check valves, pump isolation valves, suction piping, discharge piping, and sump agitation/drain pump were installed as part of the 1995 contract and are nearly 25 years old and are no longer pumping at their rated capacity (9,300 gpm or 13.5 mgd). Plant staff report that the current pumps each appear to only have a peak capacity of 8 mgd, for a total station flow of only about 24 mgd. (Pump No. 1 was replaced in the Spring of 2020).

The configuration of pumps, piping, valves and wet wells makes O&M of the pump station challenging, due to inadequate clearances and inadequate isolation capabilities. There are a number of aspects of the existing station that are not compliant with Hydraulic Institute (HI) standards for well-designed pumping stations.

A lack of area separation in the pump station are not compliant with current NFPA 820 requirements. It is also appears that the electrical room and the room where the MCC is located are not properly ventilated, in accordance with Chapter 9 of NFPA 820 to allow for them to be declassified from a Class 1, Division 2 area.

The upper floor (including MCC room) is susceptible to flooding which would inundate lower levels of the station.

#### 4.3.2.2 Recommendations – Influent Pumping

Due to the age, configuration and elevation of the existing influent pumping station it is recommended that a new pumping station be constructed that meets current codes and standards be constructed.

### 4.3.3 Primary Treatment

The East Side WWTP has three 99.5-ft by 26-ft rectangular primary settling tanks with concrete support center walls with an operating sidewater depth of 10.7-ft. Screened influent is pumped through a 42-inch force main into the primary settling tank influent distribution channel. The influent flow is measured by a Parshall flume located just upstream of the influent distribution channel. Flow isolation to each primary settling tank is provided through a slide gate along the channel. After the gate the flow enters a second channel which distributes flow to each tank through six influent ports. The influent channels are aerated to keep solids (including grit) in suspension.

The overflow rate with all primary settling tanks operational exceeds conventional design standards. TR-16 recommends a design average surface overflow rate not to exceed 1,200 gpd/ft$^2$ and a peak overflow rate of 3,000 gpd/ft$^2$. The overflow rates at average and peak design flow with all tanks operational is 1,288 gpd/ft$^2$ and 5,154 gpd/ft$^2$, respectively. Typically, the plant operates with only two of the three tanks, which results in an even higher overflow rate. In addition to high overflow rates, the sidewater depth of 10.7 feet is on the low end of recommended tank depth.

Each tank has two sets of non-metallic chain and flight collector systems that guide floating solids into a manually operated rotating scum trough at the effluent end of the tanks and settled sludge to the head of the tanks. Sludge is moved to a central hopper by cross collectors and then pumped to the sludge degritting system.

The primary effluent flows into fiberglass effluent troughs over adjustable V-notch weirs. Flow in excess of 24 mgd, up to 40 mgd, is redirected from the primary effluent channel to a secondary bypass channel, which conveys flow to the chlorine contact tanks.

Without a grit removal system, grit settles in the channel preceding the primary settling tanks and is flushed during peak flows into the primary settling tanks, increasing the risk of overloading the collection system mechanisms, and clogging primary sludge pumps and piping. In addition, scum, rags and other floating debris accumulates along the weirs due to ineffective screenings removal and dysfunctional scum collection equipment.

There are five primary sludge pumps, one dedicated to each of the three primary clarifier and two standby pumps. These are located in the below-grade pump gallery, adjacent to the influent side of the primary settling tanks. The primary sludge pumps are all Hayward Gordon pumps, with the characteristics shown in **Table 4.3-1** below.

**Table 4.3-1 East Side WWTP Primary Pump Information**

| Primary Sludge Pump Information | |
|---|---|
| Manufacturer | Hayward Gordon |
| Model | XR3-11 |
| Type | Non-Clog, Recessed Impeller, Vortex-Type |
| Number of Pumps | Five |
| Approximate Age | Over 20 years old, some pumps have been rebuilt. |
| rpm | 1,200 rpm |
| Capacity | 200 gpm, 49 ft TDH |
| Motor: | 15 hp |

The primary sludge pumps convey primary sludge from the three primary settling tank hoppers to the primary sludge degritters. The Degritter Building was constructed during the 1995 plant rehabilitation project. Typically, only two of the three primary settling tanks are operational at a time, and two primary sludge pumps are used to pump to the degritters.

The Degritter Building, located adjacent to the GTs, was constructed in the 1995 plant rehabilitation project to house the primary sludge degritting equipment. The East Side WWTP has three grit classifiers, each with two cyclones to remove grit from the primary sludge. Typical operation requires two units running, each essentially dedicated to one primary clarifier but with sufficient cross-connections to provide operational flexibility. Each classifier is sized to handle a maximum of 91 $ft^3$ per hour. Degritted primary sludge then flows by gravity to the Mixed Sludge Wet Well before it is pumped to the GTs.

Primary scum was designed to be collected from the top of the primary settling tanks and conveyed to a primary scum pit adjacent to the primary settling tanks. The rotating scum collection troughs at the effluent side of the primary settling tanks were intended to be periodically rotated (manually) just below the water level to collect and convey the scum to the scum pit. Scum was intended to be decanted in the pit and the remaining solids would be collected by vactor truck and discharged to the sludge holding tanks.

In practice, there were multiple issues with the operation of the primary settling tank scum removal system rendering the system inoperable. Today, WWTP operators are required to periodically remove scum from the surface of the primary tanks directly using a vactor truck. The lack of proper scum removal in the primary tanks results in the accumulation of large amounts of floating scum on the surface of the primary tanks that pass through to the subsequent unit processes under high flows. **Figure 4.3-3** shows a photograph of such accumulation observed during a site visit.

**Figure 4.3-3**
**Inoperable Primary Scum Collection Troughs and Accumulated Scum**

### 4.3.3.1 Assessment and Deficiencies – Primary Treatment

The primary settling tanks are undersized based on overflow rate for the design average and peak flow at the East Side plant. The primary sludge pumps and associated valving are nearing the end of their useful life. Although they are generally reliable, they are over 20 years old and most have been rebuilt over time. The primary scum system has not worked well since the time of installation in 1995 and is currently inoperable.

Access to the primary settling tank pumping gallery is well below the design flood elevation making the pumps susceptible to flooding.

#### 4.3.3.2 Recommendations – Primary Treatment

The existing conventional primary settling tanks are marginally adequate for design average daily flow conditions and undersized for peak flow conditions. Primary settling tank equipment and primary sludge pumps and piping are past their useful life. It is recommended that the system be rehabilitated and expanded or replaced in its entirety. Tankage could be reused if feasible.

### 4.3.4 Secondary Treatment

The original activated sludge process at the East Side plant was designed to treat an average daily flow of 10 mgd and a maximum daily flow of 24 mgd to achieve conventional secondary treatment standards. The tanks were constructed in 1969. A secondary treatment bypass exists to direct flow in excess of 24 mgd, up to 40 mgd, from the primary effluent channel directly to the chlorine contact tanks. The process is currently operated as three individual treatment trains, each consisting of two bioreactors and one secondary clarifier. Each treatment train has a dedicated RAS pumping system from its designated secondary clarifier and WAS is removed from each train separately. Each bioreactor is divided into four, evenly sized compartmentalized cells referred to as zones “A” through “D”, Fine bubble diffused aeration was incorporated in the 1990s.

The bioreactors were modified in 2002 to operate in an MLE process configuration to achieve some level of nitrogen removal. Zone A was converted to an anoxic zone for denitrification with mechanical mixing and IR pumping to pump return nitrate from the end of the aerobic zone to the anoxic zone as shown in **Figure 4.3-4**. Each bioreactor is comprised of a one-stage anoxic zone (A), followed by a three-stage aerobic zone (B, C, and D). The six bioreactors have a total volume of 2.12 MG (0.35 MG per bioreactor). Each anoxic zone has one submersible mechanical mixer. IR pumps are designed to provide a recycle of up to three times the average design flow.

The aeration basins and influent channels are configured such that it is possible to operate in a step feed mode for wet weather management by introducing a portion of the primary effluent to Zone B, C, or D. When the WWTP flow exceeds 12 to 15 mgd, the plant is operated in step-feed mode by directing 50% of the primary effluent to Zone C, as shown in Figure 4.3-4.

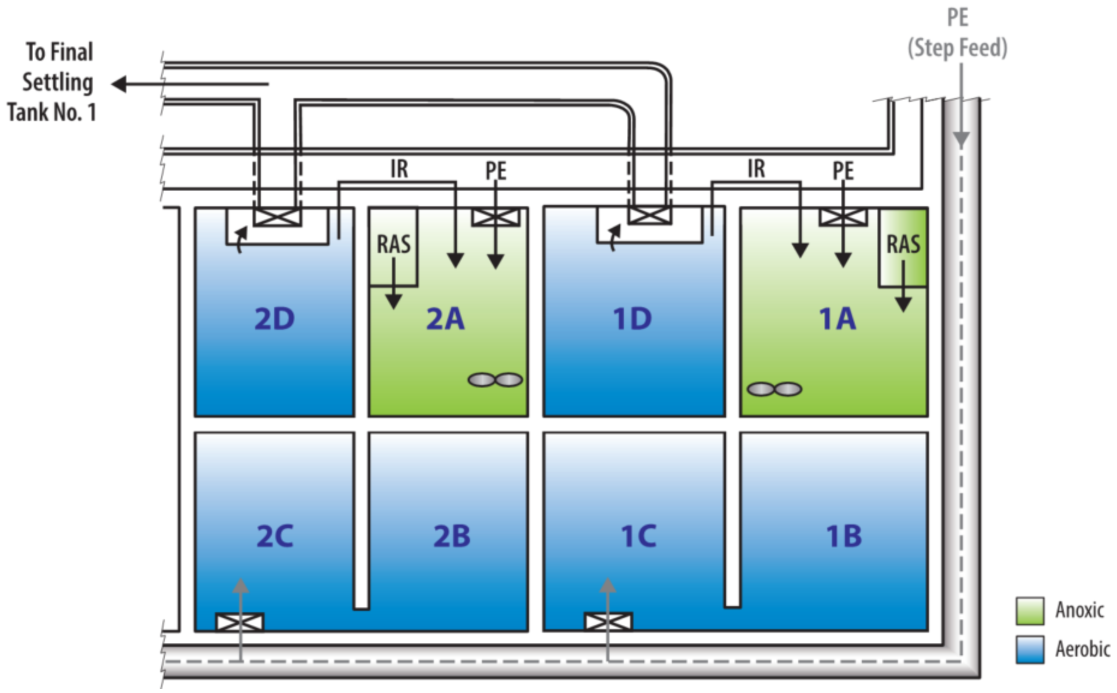


**Figure 4.3-4**
**Schematic of Existing MLE Process: Bioreactors No. 1 and No. 2**

**Table 4.3-2** summarizes the BNR system design criteria, including the size of the main process units and individual zones.

### Table 4.3-2 Bioreactor Design Details

| Design Element | Value |
|---|---|
| **Secondary Flows** | |
| Design Average | 10 mgd |
| Peak Flow | 24 mgd |
| Number of Trains | 6 |
| Total Process Volume | 2.12 MG |
| Process Volume, each train | 0.35 MG |
| Average Aerobic SRT (2017-2019) | 22 days |
| Average MLSS (2017-2019) | 5,100 mg/L |
| **Pre-Anoxic Zone** | |
| Total Anoxic Volume | 0.53 MG |
| Anoxic Volume, each train | 0.09 MG |
| Number of Stages, each train | One (A) |
| Zone Dimensions (L x W x SWD[1]) | A: 43 ft x 20 ft x 13.8 ft |
| Diffusers (not used) | |
| Type | Fine bubble 9-inch ceramic disk diffusers |
| Number of Installed Diffusers, per stage | --- |
| Number of Diffuser Holders, per stage | 240 |
| Mixer | |
| Type | Submersible mechanical mixers |
| Number of units, per train | 1 |
| Motor Size | 4.9 |
| **Aerobic Zone** | |
| Total Aerobic Volume | 1.59 MG |
| Aerobic Volume, each train | 0.27 MG |
| Number of Stages, each train | Three (B, C, and D) |
| Zone Dimensions (L x W x SWD[1]) | B: 43 ft x 20 ft x 13.8 ft<br>C: 43 ft x 20 ft x 13.8 ft<br>D: 43 ft x 20 ft x 13.8 ft |
| Internal Recycle Pumps | |
| Type | Submersible propeller |
| Number of Units, total | 6 |
| Capacity | 1,850 gpm @ 2 ft (typically operated at 25 Hz) |
| Motor Size, each | 3.6 hp |
| Diffusers | |
| Type | Fine bubble 8.75-inch ceramic disk diffusers |
| Number of Installed Diffusers, per stage | B: 248<br>C: 136<br>D: 80 |
| Number of Diffuser Holders, per stage | B: 288<br>C: 240<br>D: 240 |

[1] Sidewater depths (SWD) are based on average water surface elevation in the 1992 hydraulic profile by Kasper Associates, Inc.

As shown in Table 4.3-2, the existing bioreactors provide a total of 1.59 MG of aeration volume. Based on 2017-2019 primary effluent $BOD_5$ loads and required aerobic SRT required for nitrification, this volume is adequate.

In a conventional activated sludge system, the WWTP's secondary clarification process limits the allowable MLSS concentration in the upstream bioreactor tankage – and therefore impacts the bioreactor volume that will be required to meet process goals. As presented below, the maximum allowable MLSS concentration, under current flows and loads, and current reported SVI, to avoid overloading the secondary clarifiers is estimated to be 3,900 mg/L (the WWTP operates at a higher MLSS concentration, likely to maximize nitrogen removal within the existing tankage).

With the maximum allowable MLSS of 3,900 mg/L the maximum allowable biomass under air is about 52,000 lbs. The required SRT at minimum month temperature is 9.9 days. Based on the historic Net Yield the existing capacity of the aeration volume is 20.4 mgd (versus the permitted peak flow of 24 mgd). This analysis was conducted assuming conventional plug flow operation. When influent flow approaches 12 to 15 mgd the operators shift to a step feed operation, the loadings on the clarifiers could be reduced allowing them to manage flows up to the permitted 24 mgd.

The primary function of the anoxic zone is to maintain true anoxic conditions (low to no DO and ample soluble cBOD) to promote effective denitrification of the high nitrate IR flow. A secondary function of the anoxic zone is filamentous organism control. The anoxic reactor is a contiguous tank, not compartmentalized into smaller zones to create a gradient of F/M ratios. This design is not an ideal design of a pre-anoxic volume to select against filamentous organisms.

### 4.3.4.1 Blowers

Air is supplied to the six aeration tanks by three, 200 hp, multi-stage centrifugal blowers manufactured by Hoffmann that are inlet throttled to control flow.

Blowers were installed in 2003. Each blower has the capacity to provide 4,200 cfm at a discharge pressure of 7.7 psig. Currently, the air supply is adjusted manually based on DO readings that are taken once per shift with handheld probes. Based on the readings, the inlet guide vanes on the blowers are positioned to regulate the amount of air that goes through the unit's impeller. Typically, one blower operates with the inlet guide vane fully open. Two blowers typically operate in the summer months to meet the increased air requirements. The aeration blowers also provide air to the aeration influent channels to keep solids in suspension.

Blower discharge air flows through a 16-inch stainless steel header. There is a flowmeter on the 16-inch line which is no longer operational. The 16-inch pipe splits to 6, 12-inch header pipes that service each of the BNR basins. The 12-inch headers each have a butterfly valve used to isolate each BNR basin. The diffuser grid in Zone A was completely blanked off as part of the BNR Retrofit Project. As part of the BNR Retrofit Project Hach LDO DO probes were installed in each aerobic zone, but these probes have not been maintained and are no longer used to monitor or control the aeration system.

Based on MORs and daily operating log sheets, DO levels within the aeration basins often reach concentrations as high as 6 mg/L. High DO readings indicate that the aeration system is likely receiving excess air (target maximum DO setpoints in automated systems typically range from 1.5 to 2 mg/L). By over-aerating the aerobic zones, energy is wasted, and there is the risk of pumping high DO to the anoxic zones via the IR.

There is no automated control of the air supply to the aeration tanks. The air flow to the basins is maintained constant and there are no functioning air flow meters on any of the aeration piping. It is very uncommon for a biological nutrient removal plant of this size to rely solely on manual operations.

### 4.3.4.2 Secondary Clarifiers

The East Side WWTP has three, rectangular, dual channeled secondary settling tanks, each 240 feet long and 28 feet wide. Three influent distribution channels typically carry mixed liquor from a pair of aeration tanks to a dedicated secondary clarifier, thus creating three independent parallel trains. When one clarifier is out of service, aeration basin effluent may be redirected with slide gates located along the distribution channels. The tanks operate with a sidewater depth of 10.25 feet and a combined surface area of 20,160 square feet. At the design average daily flow rate of 10 mgd, the hydraulic loading rate (surface overflow rate) is 496 gal/day/ft$^2$.

Non-metallic chain and flight systems manufactured by Polychem scrape settled sludge into the center sludge trough and a cross collector directs sludge to the sludge hopper. Each tank has two 4-shaft longitudinal collectors at the front-end run, two 3-shaft longitudinal collectors at the back end run and one 3-shaft cross collector in the center trough. The 4-shaft collectors move floating scum from the surface of the tanks into a scum trough at the influent end of the tank. Sludge collected is returned to the aeration tanks or wasted. Treated secondary effluent flows over v-notch weirs and is directed to the chlorine contact tanks.

Based on this assumed sludge settleability at the secondary system's capacity (or maximum day flow) of 24 mgd, the maximum allowable MLSS to be maintained within the secondary process is 2,500 mg/L. The facility operates at a much higher MLSS concentration to facilitate biological nutrient removal within the bioreactors leaving the secondary system susceptible to solids washout at higher flows through secondary.

### 4.3.4.3 Return and Waste Activated Sludge Pumps

The RAS system returns secondary sludge from the final settling tanks back to the aeration tanks to adjust microorganism concentration and maintain biological treatment. The WAS system directs flow from the final settling tanks to either the GT via the mixed sludge wet well or to the GBT. There are six, RAS pumps, installed in the secondary clarifier gallery. The RAS pumps draw sludge from the final settling tanks and pump it to the return sludge control boxes at Zone A in each aeration tank. Under normal operation, two RAS pumps are dedicated to each final settling tank; one operates as a lead pump and the other acts as a standby pump.

Each set of RAS pumps feed into a common RAS header. There are multiple valves that can be manipulated to direct flow to different aeration tanks and isolate pumps. The original design intent and normal operation was for one set of pumps to serve two aeration tanks, feeding RAS flow into Zone A. These pumps were installed in the 1995 upgrade and are nearing the end of their useful life. RAS flow is chlorinated year-round to avoid filamentous growth and control bulking in the aeration tanks, operators have reported 50 gallons per day of chlorine usage, this equates to a dose of approximately 2.3 mg/L. Chlorination of the three RAS headers is rotated on a daily basis so that each header is chlorinated for 24-hours every third day.

The available data from the MORs indicate that between 2017 and 2019, the East Side WWTP has had an average RAS flow rate of 58% of the plant influent.

There are three existing WAS pumps, which were installed during the 1995 plant upgrade. The WAS pumps are also located in the secondary clarifier gallery. These pumps draw sludge from the RAS pump suction line and can discharge sludge to either the mixed sludge wet well (which eventually reaches the GTs), or to the GBT. During normal operations, WAS flow is directed towards the GBT, and when the GBT is offline, WAS flow is directed to the GTs via the mixed sludge wetwell. Operators can also use valves to waste using the RAS pumps directly, isolating the WAS pumps if maintenance is required.

### 4.3.4.4 Secondary Scum

The secondary scum system at the East Side WWTP was designed to remove scum from the top of the final settling tanks and convey it to a secondary scum pit on the western side of the final settling tanks. The scum is directed towards the influent side of the tank via a flight system and pushed to a scum weir. Periodically the six secondary scum gates (one for each bay of the final tanks) can be manually opened to allow the scum to pass into a collection trough and flow into the secondary scum pit and a subcant pit. As with the West Side plant, several issues were observed with the secondary scum system, which make operation of this system challenging.

### 4.3.4.5 Assessment and Deficiencies – Secondary Treatment

The East Side secondary treatment system has been performing well over the past few years. The plant has been able to achieve better nitrogen removal than would be expected from an MLE process configuration. The plant has adequate aeration volume to maintain nitrification through the winter under current flows and loads.

The pre-anoxic reactor is not ideally configured, as one single pre-anoxic zone per tank. There are no baffles to create smaller, compartmentalized zones which would create a F:M gradient that would inhibit the growth of filamentous organisms within the downstream aeration reactors. The BNR mechanical equipment (mixers, IR pumps, aeration diffusers) are nearly 20 years old and at the end of its useful life.

The aeration blowers are currently more than 20-years old and should be replaced with more appropriately sized, efficient, modern blowers. Blowers are inadequately sized to achieve airflow requirements that fall between one and two blowers. Aeration piping has reached the end of its useful life, and the current aeration piping configuration requires an entire basin to be taken offline to service a flow meter or air flow control valve. No operable automated blower control exists resulting in over-aerating, return of DO to the anoxic zone in the IR piping, and wasted energy.

The secondary system operates at a higher MLSS concentration than desired to facilitate biological nutrient removal within the bioreactors, leaving the secondary clarifiers susceptible to solids washout at higher flows through secondary system. In addition, the secondary clarifiers currently operate with a sidewater depth of 11 feet, which is below the recommended design minimum of 12 feet from TR-16. The secondary clarifier sludge and scum collection equipment is past its useful life.

The RAS and WAS pumps are 25 years old and have reached the end of their useful service life. The associated piping and valving are in poor condition and should also be replaced. In addition, there is no accurate method of quantifying the amount of sludge that is wasted.

Aeration blowers and associated MCCs, located in the control building, are installed more than 3 feet below the design flood elevation. In addition, the secondary gallery, housing the RAS pumps, is susceptible to flooding. The access elevation to the secondary gallery is approximately 5.5 feet below the design flood elevation.

#### 4.3.4.6 Recommendations – Secondary Treatment

The secondary system in its current state is slightly undersized and design could be optimized to consistently achieve low level nitrogen removal year-round under design flow and load conditions. Most equipment and controls are beyond their useful life. The system should be upgraded and expanded to enable year-round nitrogen removal and improve operational efficiency.

### 4.3.5 Disinfection

The East Side WWTP uses sodium hypochlorite for disinfection and sodium bisulfite for dechlorination. The plant has three identical chlorine contact tanks, each with two passes. The tanks were originally constructed in the 1950s as primary clarifiers. In 1969, they were converted to chlorine contact tanks and in 1992, the walls and weirs were raised. Each pass is 190-feet long by 17-feet wide and the side water depth is approximately 16 feet, resulting in a length to width ratio of 22 to 1. This is about half the TR-16 recommendation of a minimum ratio of 40 to 1 which could result in short-circuiting. Effluent from the chlorine contact tanks discharges into an effluent channel which connects to the 60-inch plant outfall pipe.

The chlorine contact tanks receive all plant flow including secondary treated effluent and all flow bypassed around secondary treatment. At 10 mgd, the chlorine contact tanks provide over 100 minutes of detention time with only one tank in service. This is well in excess of the 30-minute minimum recommended by TR-16. At 40 mgd, the chlorine contact tanks provide over 50 minutes of detention time with two tanks in service. Typically, two tanks are in operation.

Overall, the existing tanks far exceed the detention time required to provide proper contact time for disinfection. Despite the adequate sizing, the plant has had sporadic NPDES permit limit violations, perhaps due to dead zones within the large tanks and regrowth. The enterococci monthly geometric mean has exceeded the permit limit four times in the past three years. Fecal coliform and total residual chlorine limits have not been violated over the past three years.

### 4.3.5.1 Sodium Hypochlorite Storage and Feed System

The chlorination system was originally designed with the use of chlorine gas; however, it was converted to sodium hypochlorite in 1999. The sodium hypochlorite feed system consists of two storage tanks and three liquid injectors.

The system has two, 6,400-gallon crosslinked polyethylene tanks for a total storage capacity of 12,800 gallons providing approximately 30 days of storage. The tanks are equipped with a leak detection system and are located in an outdoor concrete containment area to contain any spills. The containment area is capable of holding the total storage volume of one tank. The plant currently receives 12.5 to 15.6% sodium hypochlorite by weight, however degradation likely occurs quickly in the summer months since tanks are not inside or shaded.

Three liquid, vacuum injectors distribute the sodium hypochlorite to multiple application points throughout the plant. The injectors vary the flowrate of sodium hypochlorite addition using flow pacing or by manual control. The plant water system supplies the injectors with water and sodium hypochlorite is added to the carrier water in the injectors. Sodium hypochlorite is can be injected at the following locations: (1) the common secondary clarifier effluent, (2) the primary effluent bypass channel upstream of the chlorine contact tanks, (3) to the raw sewage influent for odor control and (4) to the RAS piping for bulking control.

All chlorine contact tanks are aerated with a single multistage centrifugal blower located in a fiberglass enclosure at the east end of the chlorine contact tanks. Aerating chlorine contact tanks is not typical in wastewater treatment, and likely required due to the oversized chlorine contact tanks.

### 4.3.5.2 Dechlorination Storage and Feed System

The plant has an effluent chlorine residual maximum daily limit of 0.1 mg/L and an average monthly limit of 0.05 mg/L. To achieve these limits, the plant dechlorinates using a sodium bisulfite feed system installed in 2009. The chemical is stored in two, 750-gallon storage tanks and is conveyed to the effluent via two, variable rate dosers operating in duty/standby configuration. The rate of dosing is controlled by the influent flowrate or by manual control. Sodium bisulfite is discharged into the downstream end of the chlorine contact effluent channel, after the chlorine contact tank weirs, with two induction mixers. A chlorine residual analyzer is located downstream of the sodium bisulfite application point to confirm compliance with the permit.

The sodium bisulfite tanks and dosers are located in the dechlorination building located on the southwest corner of the chlorine contact tanks. The tanks have a containment wall providing enough storage for the entire volume of both tanks in the event of a spill. The fill lines for the sodium bisulfite tanks are located adjacent to the sodium hypochlorite tanks.

Assuming a maximum of 2 mg/L of chlorine residual to remove, the existing storage tanks provide eight days of storage, which is not sufficient per the TR-16 requirement of 30 days minimum storage.

#### 4.3.5.3 Assessment and Deficiencies - Disinfection

The fatal flaw with the existing chlorination system is the potential for flooding. The top of the chlorine contact tank walls is at elevation 27.00 which is below the current 100-year flood plain elevation of 27.60 and well below the design flood elevation of 30.60. The storage and feed systems including tanks and dosing units are also below the design flood elevation.

The chlorine contact tanks, originally constructed in the 1950s are beyond their useful life and are over-sized for contact time at current average and peak flows, and do not have a sufficient length to width ratio to meet TR-16 requirements.

The sodium hypochlorite storage and feed system is over 20 years old and approaching the end of its useful life. Additionally, the storage tanks are located in an outdoor containment area with no protection from the sun which can expedite sodium hypochlorite degradation. The sodium bisulfite storage tanks are undersized for the required 30 days of storage.

#### 4.3.5.4 Recommendations - Disinfection

Given the flood vulnerability of the existing chlorine contact tanks and the age of the existing chemical storage and feed systems, the current disinfection system must be replaced with a new system at a higher elevation.

### 4.3.6 Effluent Outfall

The East Side effluent outfall pipe is a 60-inch RCP that conveys plant effluent by gravity from the chlorine contact tanks, to the outfall at the powerhouse channel, adjacent to the plant site. The powerhouse channel flows to Bridgeport Harbor and ultimately Long Island Sound.

The total length of the East Side WWTP outfall, from the chlorine contact tanks to the discharge is approximately 600 feet. Two-thirds of the outfall pipe (approximately 400 feet) was constructed during the 1969 design, while approximately 200 feet dates back to the original primary treatment facility (circa 1950). The discharge point is protected with steel sheet piling and can be either partially or completely submerged depending on the tide elevation. Operators have stated that the effluent piping can be inundated quite easily by tidal flows, and they frequently observe eels in the effluent pipe as far upstream as the chlorine contact tanks.

The outfall pipe follows a circuitous route and there are several junction chambers along the way where the outfall pipe direction changes significantly, due to the numerous plant upgrades that have occurred over time.

The velocity through the outfall pipe is less than 1 foot per second under average design flow and about 3.2 feet per second under design peak flow.

#### 4.3.6.1 Outfall Inspection

On January 29, 2020, the East Side WWTP outfall pipe was inspected via underwater inspection by ASI Marine using a small ROV. The ROV was equipped with a high-definition camera, imaging sonar, and profiling sonar. A complete report of the inspection work is included in **Appendix E**. The overall observations and findings from the ROV inspection are as follows:

- Two gaps in joints were observed between the outfall and the first upstream chamber. Inspectors suggested that these could be expansion joints. These defects are located within the older portion of the outfall pipe.
- Debris and sediment piles were observed throughout the pipe. These were most noticeable at the junction chamber bends. A max debris depth of 15 inches was noted on the sonar profiles. Concrete aggregate was visible within the video footage at some points.
- A metal band for a flow meter was observed in the outfall pipe between the chlorine contact tanks and the first downstream junction chamber. This flow meter is not believed to be operational.

#### 4.3.6.2 Assessment and Deficiencies – Effluent Outfall

Overall, the East Side WWTP outfall pipe is aging, but appear intact. Although no structural testing was conducted, no major defects were observed. Debris and sediment were observed throughout the outfall pipe, as well as some exposed concrete aggregate, and a few offset joints were observed.

#### 4.3.6.3 Recommendations – Effluent Outfall

If maintained, it is recommended that the existing outfall be cleaned and lined to extend the life of the outfall pipe.

### 4.3.7 Sludge Processing and Disposal

The East Side plant has employed various solids handling systems over the years. Currently, under normal operating conditions, primary sludge is pumped through the sludge de-gritting facility then to the GTs. Thickened primary sludge is then pumped to the sludge storage tanks in the Sludge Building. WAS is pumped directly to a GBT. Thickened WAS is then pumped to the one of two sludge storage tanks and mixed with the primary sludge. Combined sludge is then pumped to a tanker truck and hauled off-site for disposal.

#### 4.3.7.1 Gravity Thickeners

The East Side WWTP has three GTs located south of the chlorine contact tanks. These GTs were constructed during the 1969 facility upgrade and were upgraded and rehabilitated during the1995 plant rehabilitation project. Typically, two of the three GTs are operated, and one is used as a standby. Design details are presented in **Table 4.3-3.**

Table 4.3-3 Gravity Thickener Design Information

| Gravity Thickener Information | |
|---|---|
| Number | 3 |
| Diameter (ft) | 30 ft |
| Side Water Depth (ft) | 10 ft |
| Surface Area ($ft^2$), each | 707 $ft^2$ |
| Volume (gal), each | 52,880 gal |
| Design Overflow Rate (gpd/sf)[1] | 736 gpd/sf |
| Sludge Received | Primary Sludge |
| Solids Loading Rate (lb/sf/day)[1] | 36 lb/sf/day |

[1] Assumes Two Thickener Tanks in Service

Degritted primary sludge is pumped to the GTs. In the GTs, the solids settle to the bottom of the tank. The supernatant flows over a weir from each tank into a central supernatant wet well. In 2009, a Flygt dry-pit submersible pump was installed in this wet well to convey the GT supernatant to the primary effluent channel.

The thickened sludge at the bottom of each tank is pushed to a center hopper by a center drive rake mechanism with squeegee blades. A single Penn Valley double disk pump, pumps thickened sludge to the sludge storage tanks.

Each GT tank is 30 feet in diameter with a depth 10 feet. Based on data from 2017 to 2019, the average overflow rate of the GTs is 736 gpd/sf and the average solids loading rate is 36 poundsper square foot per day with two thickeners online. TR-16 design criteria recommends an overflow rate for primary sludge thickening of 400-800 gpd/$ft^2$. Additionally, a solids loading rate between 20 and 30 pounds per square foot per day for primary sludge is recommended. The data provided from the MORs would indicate the thickeners are operating at an acceptable hydraulic loading rate, but the solids loading is slightly higher than recommended when utilizing only two of the three thickeners. It is reported that typically they thicken to 3.5% solids.

Each thickener tank is covered with a fiberglass cover and was originally designed with an activated carbon odor control system. This odor control system in no longer in use.

The original scum conveyance system is inoperable. Floating scum on the top of the tanks is believed to overflow with the supernatant flow and is recirculated to the primary effluent.

The GT tanks show signs of concrete degradation. There are areas where cracking has been repaired on the outside of the tanks. Active concrete degradation, exposed aggregate, and corrosion can be seen on the inside of the tanks. It is believed that the tank covers trap $H_2S$ gases within the tank which may contributing to the concrete degradation.

### 4.3.7.2 Gravity Belt Thickener

The East Side WWTP has a single Komline-Sanderson 1.0-meter Gravabelt GSC-1 Series III GBT located in a plywood room within the Sludge Building for thickening WAS. Typically, the GBT is operated three to four days per week for approximately six hours per day, depending on secondary solids loading. WAS is pumped directly from the secondary clarifiers to the GBT. There is no intermediate storage tank, so WAS builds up in the secondary clarifiers when the plant is not actively wasting. The WWTP typically operates with a 5-ft sludge blanket in the secondary clarifiers.

WAS ranging from 0.5 to 1.0% solids is dosed with a liquid polymer prior to entering the feed end of the GBT. The thickened WAS, typically around 4% solids, drops into a hopper at the end of the belt and is pumped by a simplex plunger pump to the thickened sludge storage tanks to be stored with thickened primary sludge. There is no redundant GBT unit, requiring WAS to be co-thickened with primary sludge in the GTs when the GBT is out of service.

The GBT filtrate and wash water flows by gravity to the adjacent Screening Building, where it enters the process stream upstream of the Main Screen.

#### 4.3.7.3 Thickened Sludge Storage, Pumping and Hauling

Thickened primary sludge pumped from the GTs is stored with thickened WAS in two concrete tanks located in an area of the Sludge Building separated from the mechanical equipment.

Combined thickened sludge is stored until it is pumped into tanker trucks and hauled off-site for disposal by Synagro. Each storage tank has a holding capacity of 82,600 gallons. The tanks have been in service for over 50 years and are showing significant signs of deterioration. An air compressor is located in the adjacent mechanical room to provide mixing air to the storage tanks but is no longer in service. The storage tank room does not appear to have functional HVAC nor odor control. As a result, there is a notable accumulation of $H_2S$ in the room and the area is not safe for human occupancy.

Hauling data provided through Synagro invoices is the only means for monitoring thickened sludge production quantities. Current average day combined thickened sludge production suggests that the East Side WWTP has approximately 10 days of storage between the two storage tanks. On average, the East Side WWTP fills 87 truckloads per month averaging 1 ton (dry solids) per load. More than 90% of the loads are greater than 2% solids. Demurrage charges at the East Side are uncommon but do occur about once per month.

The rotary lobe truck loading pump is located in the lower level of the Sludge Building. The pumping rate of the constant speed pump varies with the solids content of the sludge being pumped, which typically ranges from 4 to 5% solids. The pump typically fills a single truck in 25 to 30 minutes. The sludge tanker fill station is located on the northeast corner of the Sludge Building. The fill station consists of a manual valve and a 6-in flexible fill hose, with a drainage area and catch basin to contain small spills. The piping and valve are heat traced to prevent freezing. Additionally, there is a small drum-style odor control system, but it does not appear to be operational. Truck operators pull up to the station, connect the fill hose to the truck and open the valve. A plant operator then starts the Truck Loading Pump and monitors the filling process throughout.

### 4.3.7.4 Assessment and Deficiencies – Sludge Processing

The GTs were constructed as part of the plant upgrade project designed in 1969. The tanks were rehabilitated in the 1990s. The GT equipment is past the end of its useful life and the concrete is exhibiting signs of deterioration likely attributed to $H_2S$ degradation.

The Sludge Building was constructed in the 1950s and has supported the solids disposal processes at the plant through multiple process upgrades. The building currently houses the GBT, Sludge Storage Tanks, and sludge hauling processes. Overall, the building is deteriorating and much of the supporting building mechanical equipment (HVAC, electrical, structural, etc.) has not been maintained, making some areas hazardous.

The system lacks redundancy of much of the equipment and equipment capacity is not sufficient for future growth or proposed operating conditions.

The entire sludge facility is within the 100-year floodplain and has many openings below the design flood elevation, which if not protected could result in inundation of the entire lower level of the building which would cause significant damage to equipment and thickening operations.

### 4.3.7.5 Recommendations – Sludge Processing

The existing sludge processing and disposal system is inadequate and is in need of rehabilitation to upgrade existing equipment and to provide permanent and reliable means of sludge management.

## 4.3.8 Odor Control

Odor control at the East Side plant consists of chemical scrubbers and carbon canisters. Two, vertical countercurrent flow scrubbers serving the screen building and the Sludge Degritter Building and one, three-stage packed bed scrubber to treat odorous air captured from the GTs.

Each scrubber was designed for 10 feet of packing that creates a gas/liquid interphase between the influent air and the scrubbing water. The scrubbing liquid was created by mixing plant water with solutions of 15% sodium hypochlorite and 50% sodium hydroxide, which was sprayed from a nozzle at the top of the device to absorb the $H_2S$ and other gases from the odorous air stream introduced at the bottom of the scrubber. Each scrubber location had two recirculating pumps to cycle scrubbing liquid that accumulates in the sump of the scrubber back to the spray nozzle. Plant water was added to the cycling scrubbing liquid at a rate not to exceed 10% of the recycle rate through the scrubber, while excess flow was wasted through an overflow connection in the sump and flows to the plant drain. The addition of hypochlorite and hydroxide was automated based on the ORP and pH, respectively, of the cycling scrubbing liquid, monitored by probes within the scrubber.

In addition to the air scrubbing system, the East Side WWTP uses 11 Air Phase Carbon drums to treat odorous air vented from various locations around the WWTP. Exhaust fans draw odorous air from an enclosed or covered space to the inlet of one of the 55-gallon carbon drums. The air passes through the carbon bed and then exits the outlet of the drum. The removal efficiency of the drum decreases over time, and spent drums are disposed and replaced by maintenance personnel. The release of odor as $H_2S$ concentration from the unit is monitored and recorded in the MORs. As the monitored levels reveal that efficiency has decreased, the drum should be replaced.

### 4.3.8.1 Assessment and Deficiencies – Odor Control

The plant no longer uses sodium hypochlorite and sodium hydroxide in the chemical scrubbers, instead the WWTP pumps chlorinated plant water through the scrubbers. This system has been somewhat effective at removing odor from the airstream and eliminates the need for pH adjustment. The chemical feed pumps and tanks have been abandoned in place and the equipment is generally in poor condition.

The odorous air from the GTs is no longer vented likely increasing the rate of deterioration of concrete in these covered tanks.

It is unclear whether or not the existing carbon canisters are monitored and replaced on a regular basis.

### 4.3.8.2 Recommendations – Odor Control

Due to the age and condition of the existing odor control systems it is recommended that these systems be removed and replaced with systems designed to manage odorous air from newly designed systems, namely including preliminary treatment (headworks), primary treatment, and sludge processing areas.

## 4.3.9 Structural

The oldest structures on site are the main pump station, the existing chlorine contact tanks, (the original primary settling tanks), and the Sludge Building. The existing primary settling tanks, bioreactors, secondary clarifiers and GTs were constructed in the 1969 upgrade. Since then only a few minor structures have been added to the site for chemical storage.

The general deficiencies with the structures observed to date include:

- Deteriorated concrete with exposed aggregate in some locations;
- Concrete spalling and exposed, rusted rebar in some locations;
- Roof hatches in poor condition and leaking into the gallery;
- Significant concrete deterioration at the gravity thickeners and sludge holding tanks most likely caused by $H_2S$ corrosion;
- Significant corrosion in structural steel beams in the pump station;
- Significant cracking in the brick façade above the steel lintels over the top windows on the Sludge Building as well as significant freeze thaw damage of the concrete slab in the Sludge Building.

It was also noted that the vertical steady bearing pump shafts on the influent pumps were very loud which could likely be causing unnecessary wear to the shaft and bearings.

### 4.3.10 Architectural

From an architectural standpoint, in general, many of the buildings are in poor condition and exhibit deficiencies, vulnerabilities and code compliance issues.

- Within the Vehicle Complex, multiple doors exhibited intense corrosion and were deemed unusable, flammable liquids being stored, unrestrained, and near other fuel related equipment presenting a potential life safety hazard, skylights were rusted, and an elevator was inoperable.

- The Control Building Roof parapet was in poor condition overall, the upper level only had one of its two restrooms functioning and were not ADA compliant, portions of the upper level ceiling had visible moisture accumulation and missing ceiling tiles and some stairwells lacked adequate rail protection and extensions under accessibility guidelines.

- The exterior of the raw sewage pumping station was in very poor condition and in most cases, in need of immediate repair, doors in the gallery areas had inaccessible and/or broken hardware, the roof areas that were accessible were in poor condition and would require replacement and laboratory casework and equipment needs replacement.

### 4.3.11 HVAC, Plumbing, and Fire Protection

In general, most HVAC equipment at the East Side WWTP is past its useful life, in poor condition, and/or inoperable. This includes exhaust fans, air handling units, unit heaters and duct work much of which shows corrosion due to $H_2S$ and damp environments. Some roof mounted units are located too close to the roof edge without proper fall protection.

The exceptions to these observations are the boiler systems, which appear to be in good condition. HVAC equipment in the Vehicle Complex is in good conditions. Within the Control Building, the gas-fired roof-mounted air handling units are less than two years old and remain in good condition.

With respect to plumbing and fire protection, the lack of proper fire protection systems is a life safety issue and needs to be addressed. Storage of chemicals, lubricants, waste oils and yard equipment without proper fire protection is a violation of code. Cross connections on the water system were noted throughout the buildings on site, potentially in violation of the State of Connecticut Plumbing Code. Many emergency eyewash and showers were not functional. Other deficiencies include inadequate or not existent floor drains, no hose bibs for washdown, inadequate number or bathroom facilities and/or water supplies shut off to service sinks and bathrooms.

### 4.3.12 Electrical

The majority of the electrical distribution equipment was installed as part of the plant upgrade in the 1990s and was noted during the site visit to be in fair to poor physical condition. As mentioned, the average useful life of the electrical equipment including cables is approximately 25-30 years. Therefore, many of the components (i.e. circuit breakers, contactors, transformers etc.) on this equipment may be worn beyond the manufacturers recommended limits. It is not known whether the main switchgear and MCCs are tested and maintained annually, however there were no maintenance stickers on any of the equipment which leads us to believe that preventative maintenance has not been provided. It is not unreasonable to expect an increase of failures above normal levels of maintenance as this equipment is extended to and beyond its anticipated life cycle.

### 4.3.12.1 Electrical Service

Electrical service is provided via an overhead primary line at 13.8kV to a riser pole located outside the main gate entrance. From the riser pole, the primary 13.8kV service is routed, via an underground duct bank system, to a 15 kV Outdoor Load Interrupter Switchgear, located in the grass area southeast of the Screen Building between the Administration Building and parking lot just inside the main access gate. The utility revenue metering is provided at the 15 kV Outdoor Load Interrupter Switchgear. The switchgear was installed in the late 1990s and is nearing the end of its useful life.

From the 15 kV Outdoor Load Interrupter Switchgear 13.8 kV is distributed, via underground duct banks and manholes, to three outdoor 13.8kV-480/277-volt units substations. The double ended unit substations are all outdoor walk-in type with 4000 Amp main-tie-main 480-volt switchgear located in the walk-in enclosure between the two outdoor transformers. (Transformers 1R, 2L and 3L have been replaced since they were originally installed). The mains and tie circuit breakers are key interlocked to allow only two out of the three circuit breakers to be closed at the same time. The unit substation ratings are as follows:

- Substation No.1 is a 3000-kVA double ended unit substation, which is located at the back of the facility west of the aeration tanks on the opposite side of the access road.

- Substation No.2 is a 3000-kVA double ended unit substation, which is located to the south of the Control Building on the opposite side of the access road.

- Substation No.3 is a 3000-kVA double ended unit substation, which is in the grass area south of the Screen Building on the opposite side of the access road from the Administration Building.

From the substations power is distributed to double ended MCCs with main-tie-main key interlocked circuit breakers enabling vital components of the same type and serving the same function to be divided as equally as possible between at least two motor control centers as required by EPA-430.

### 4.3.12.2 Standby Power

Standby power is supplied to the electrical distribution system through a low voltage (480V) diesel engine generator. The standby generator is in the grass area southeast of the Screen Building between the Administration Building and parking lot just inside the main access gate and adjacent to the 15 kV Outdoor Load Interrupter Switchgear. The unit was installed as part of the 1990s upgrade and was observed to be in good condition visually. The generator rating is 1500 kW, 1875kVA, 3 phase, 4 wire, 0.8 pf. This generator distributes standby power through a

3000 Amp ATS that is connected to Bus B of Switchgear No. 3. It appears, based upon our preliminary review of the existing one-line diagrams that critical process equipment is supplied with sufficient standby power to maintain treatment during utility outages. However, this should be confirmed.

The overall distribution system provides a reliable electrical distribution system that allows for flexibility in operation and maintenance, and in theory, is compliant with EPA 430-99-74-001, which states that two separate and independent sources of electric power shall be provided to the works from either two separate utility substations or from a single substation and a works-based generator system. As stated above, the 15 kV Outdoor Load Interrupter Switchgear has a single utility service but as indicated it appears that critical process equipment is supplied with sufficient standby power to maintain treatment during utility outages, from the standby generator. If the existing standby generator is adequately sized and automatically provided standby power to all critical process equipment, then the system is EPA 430 complaint and is not susceptible to common mode failures. The term "common mode failure" is an industry term used to identify a point in the electrical distribution system where a single failure will result in the complete loss of critical process or pumping systems.

It is unknown whether the existing system has sufficient capacity to accommodate any future plant process and facility loads for future upgrades.

### 4.3.12.3 NFPA 820 Review

As discussed in Section 4.2.12 NFPA Standard 820, "Standard for Fire Protection in Wastewater Treatment and Collection Facilities" (2008 Edition), establishes the requirements for protections against fire and explosion hazards in wastewater treatment plants.

Due to the nature of some of the spaces in the existing plant, continuous ventilation to declassify the space is required in order to allow the installation of electrical equipment including switchgear, MCCs, and VFDs in existing spaces.

As at the West Side WWTP many buildings' compliance with NFPA 820 are in question at the East Side WWTP. Continuous ventilation at specific rates and combustible gas detection is required along with an alarming system to declassify a space and allow general duty electrical equipment in these locations. Where declassification is required by code the recommendation shall include HVAC upgrades, gas detection upgrades and supervised alarm systems with notification appliances.

In addition, there are numerous electrical rooms that are open, through a corridor, to hazardous classified areas per NFPA 820. The hazardous classification extends to all common or adjacent areas unless physically separated. Per NFPA 820 the definition of physically separated is as follows: A gastight partition between two adjacent spaces, or two nonadjacent spaces, with no means of gas communication between the spaces and where personnel entry into the spaces is by individual, exterior access ports with no physical connection, or an airlock.

### 4.3.12.4 Arc Flash Labeling

The existing equipment does not include arc flash labeling on equipment.

### 4.3.12.5 Lighting

The majority of the lighting fixtures throughout the treatment facility including in the process areas and gallery areas are either energy efficient T-8 or high bay high pressure sodium fixtures. There are many areas with multiple fixtures that are inoperable.

Emergency battery powered egress lighting was noted throughout the facilities, including within the galleries. However, the operability of the egress lighting was not observed, and the fixtures appear to be original.

Exterior lighting appeared to be adequate; however, the operation of the exterior lighting was not observed during the site visit.

It was noted that the facilities generally contain exit signs at egress doors. NFPA 101 (Life Safety Code). Chapter 7.10.1.2. indicates, "Exits, other than main exterior doors that obviously and clearly are identified as exits, shall be marked by an approved sign that is readily visible from any direction of exit access.

### 4.3.12.6 Fire Alarm System

It was noted by plant staff that the existing plant fire alarm system is not operational. Per NFPA 820 and the building code there are numerous areas within the plant where a fire alarm system is required to meet current codes.

### 4.3.12.7 Page Party System

There is an existing page party system; however, it was noted by plant personnel that much of the existing system is currently not functional.

### 4.3.12.8 Security System

The existing plant has numerous security or process monitoring cameras. However, plant personnel noted that many of these cameras are not operational.  The facility includes card access at numerous building; however, plant personnel noted that much of this system is inoperable.

## 4.3.13 Instrumentation and Controls

In general, most of the I&C around the plant site is past its useful life, dysfunctional and/or obsolete. This includes gas sensors, level elements, DO probes, pH sensors, pressure and temperature gages, and air flow meters. Most equipment is operated locally. With respect to the general I&C systems the following deficiencies were noted:

- Buildings throughout the plant are connected via redundant OM1 fiber optic cable. There are intermittent communication outages between buildings.

- Security throughout the plant includes card access control in the Administration Building and Pelco Security Cameras mounted throughout the plant. A Dell Optiplex workstation installed in the Engineering Department of the Administration Building hosts the security software, EntraPass Corporate Edition. It is unknown if this workstation is used to monitor the Pelco cameras, or if it is only used to control the card access system. The security workstation appears to be in good condition, but it is using the Windows 7 operating system, which is no longer supported by Microsoft.

- PLCs throughout the plant are GE 90-70. The 90-70 product line was discontinued by GE in 2011, and the GE PLC product line was acquired by Emerson in 2019. The plant has started to upgrade the existing PLCs by using the GE RX3i conversion kit (the first PCU that was converted was PCU400). With this conversion kit, the field terminal blocks from the GE 90-70 are removed from the rack, and the existing GE 90-70 rack is removed from the enclosure. The GE RX3i conversion kit has an adapter strip that fits the 90-70 field terminal blocks. Six PCUs are located throughout the plant site.
- Control room contains SCADA HMI workstations for monitoring plant equipment. SCADA HMI software is GE iFix. SCADA HMI software is licensed using a legacy hardware key. This is a physical vulnerability because the workstation tower is exposed, and the key can easily be removed.

As part of the overall plant upgrade the entire I&C system should be upgraded to meet current codes and standards.

## 4.4 Plant Performance

Treatment plant performance is measured by the ability of the facility to meet or beat effluent permit limits and receive nitrogen credits. However, plant performance also relates back to the performance measures outlined in Table 1.2-1, including the frequency and volume of CSOs and primary effluent discharges, the number of odor or water quality complaints, the power and chemicals required to achieve the permit limits based on influent flow and loads, and the overall unit cost per million gallon to treat the influent flow.

Plant data from 2017 through 2019 was analyzed for both plants and the results of the analysis are presented herein. More detailed plant audits documenting the performance of the two facilities are presented in **Appendix F.**

### 4.4.1 West Side Wastewater Treatment Plant

As presented in Section 4.2, the West Side WWTP suffers from aging, undersized and inadequate treatment processes which directly and indirectly impact the ability of the treatment facility to meet permit limits. Specifically, this includes ineffective screening and no in-line grit removal, undersized primary settling tanks, a secondary treatment system that was reconfigured to impart nitrogen removal but is undersized to do so under all flow and load conditions year-round, inappropriately sized chlorine contact tanks, and the inability to effectively and efficiently remove scum and solids from the treatment system. In addition, limited operational instrumentation and controls exist making operations and control even more difficult.

It should be noted, the assessment of three years of data, revealed significant data gaps which complicated the data analysis and required certain assumptions to be made to piece together available information.

### 4.4.1.1 Influent Conditions and Effluent Requirements

As further detailed in Section 5, average influent conditions from 2017 through 2019 at the West Side WWTP are presented in **Table 4.4-1.** As presented, the average influent flow for this period was about 25% less than the design influent average daily influent flow of 30 mgd. Influent $BOD_5$ is on the weak side, averaging about 152 mg/L, while TSS averages over 228 mg/L, is considered medium strength. The difference in average $BOD_5$ and TSS concentrations is a bit unusual but could be attributed to the combined sewer system and the long detention time in the large diameter piping that can result in the wastewater settling in the collection system and being flushed in large slugs during storm events. Total Kjeldahl nitrogen (TKN) and total phosphorus (TP) concentrations also run on the weaker side, similar to $BOD_5$.

**Table 4.4-1 West Side WWTP Historic Influent Conditions (2017-2019)**

| | West Side WWTP[1] |
|---|---|
| Average daily flow (mgd) | 22.1 |
| Biochemical oxygen demand ($BOD_5$) (lb/day) | 28,000 (152) |
| Total suspended solids (TSS) (lb/day) | 42,000 (228) |
| Total Kjeldahl nitrogen (TKN) (lb/day) | 4,500 (24.4) |
| Total phosphorus (TP) (lb/day) | 780 (4.2) |

[1] concentration (mg/L) shown parenthetically

As presented in Section 2, the plant was designed to achieve conventional secondary treatment standards for $BOD_5$ and TSS: 30/30 mg/L on a monthly average basis. In the early 2000's the aeration system was upgraded to achieve some level of nitrogen removal on an interim basis. Today, an annual average effluent TN load of 1,041 lb/day is the goal set for the West Side plant, which equates to about 5.6 mg/L under current average daily flow and 4.2 mg/L at design average daily flow. Disinfection limits are consistent with coastal outfalls discharging to sensitive receiving waters: 88/100mL fecal coliform and 35/100mL enterococci.

### 4.4.1.2 Permit Compliance

A summary of permit compliance is presented in **Table 4.4-2.** Note this does not include days when permit compliance is waived when excessive flow results in secondary treatment bypass events. As presented, 2018 was a particularly challenging year for the West Side WWTP it appears that significant rainfall during that year generated many of the violations. Given sampling frequency is only times per week, the 19 violations for maximum day TSS in 2018 represents a frequency of non-compliance greater than 12%. **Figures 4.4-1 through 4.4-4** present daily flow, effluent $BOD_5$, TSS and TN concentrations for the three-year period. As eluded to above, the permit violations are related to the size, condition and operation of the existing facility as presented in more detail below.

Table 4.4-2 West Side WWTP Permit Violations (2017-2019)

| Parameter | Permit Limit | Limit Type | Number of Permit Violations[1] | | | |
|---|---|---|---|---|---|---|
| | | | 2017 | 2018 | 2019 | Total |
| $BOD_5$ | 30 mg/L | Maximum Monthly | 0 | 0 | 0 | 0 |
| | 50 mg/L | Maximum Daily | 3 | 5 | 3 | 11 |
| | 85% Removal | Average Monthly Minimum | 0 | 1 | 1 | 2 |
| TSS | 30 mg/L | Maximum Monthly | 1 | 4 | 1 | 6 |
| | 50 mg/L | Maximum Daily | 5 | 19 | 6 | 30 |
| | 85% Removal | Average Monthly Minimum | 0 | 1 | 6 | 7 |
| Fecal Coliform | 88 CFU/100 mL | Monthly Geometric Mean | 0 | 0 | 0 | 0 |
| | 10% of Samples Exceeding 260 colonies/100 mL | Maximum Monthly | 0 | 0 | 0 | 0 |
| | 2,400 CFU/100 mL[b] | Maximum Daily | 0 | 1 | 1 | 2 |
| Enterococci | 35 CFU/100 mL | Monthly Geometric Mean | 4 | 8 | 5 | 17 |
| | 500 CFU/100 mL | Maximum Daily | 1 | 4 | 2 | 7 |
| Total Residual Chlorine | 0.05 mg/L | Average Monthly | 0 | 0 | 0 | 0 |
| | 0.10 mg/L | Maximum Daily | 0 | 0 | 0 | 0 |
| | 0.20 mg/L | Instantaneous | 0 | 0 | 0 | 0 |

[1] Number of permit violations presented represents permit violations recorded in monthly MORs.

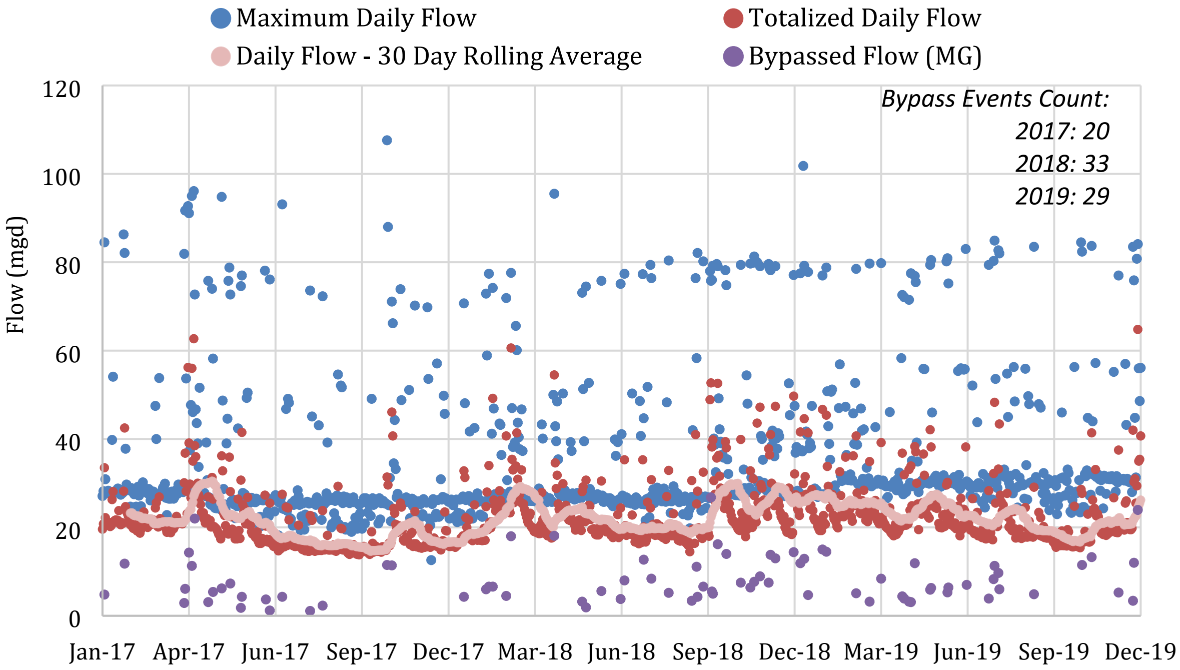
### 4.4.1.3 Unit Process Performance

*Influent Pumping*

As presented in Section 4.2, the influent pumping station with four pumps (23,500 gpm each) has a design firm capacity of about 90 mgd (peak capacity with one unit out of service). Influent gates are partially closed during high flow events to limit the peak flow to the plant to avoid flooding of the influent pumping station. As presented in Figure 4.4-1, by the cluster of data points at 80 mgd, it appears that the current influent pumping capacity is lower than the design capacity. This limitation can result in more frequent CSO discharges in the collection system.

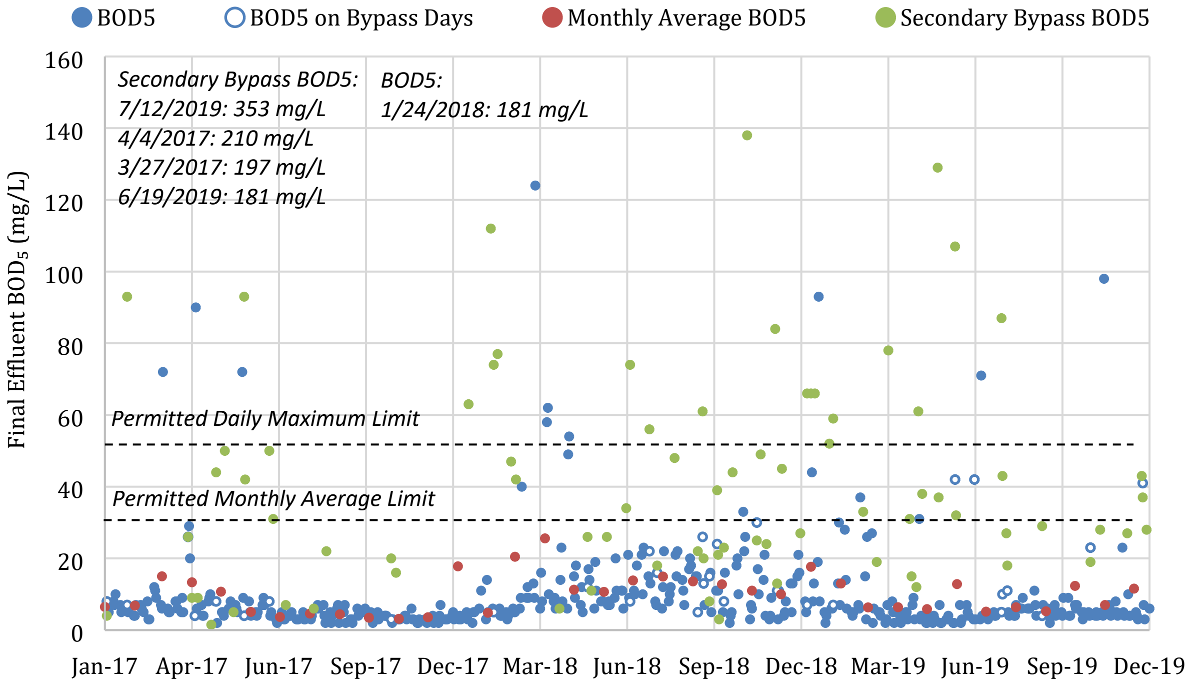
### *Screening System*

The performance of a screening system is typically characterized by the quantity of screenings removed per million gallon of wastewater treated. The range can vary widely based on whether or not the system has combined sewers and the size of openings in the installed screens. Based on industry standards, the typical range of screenings captured is from 3 to 10 cf/MG treated for a ¾-inch screen. Given the average daily flow at the West Side facility, this would equate to about 2.5 to 8 cubic yards per day of screenings. Data on the cubic yards of screenings currently removed and sent to landfill was not readily available, since much of the screenings pass through the screening facility and end up being removed with the scum removed from downstream processes. The implications of poor screening in the headworks are numerous including fouling of instruments making them inoperable, clogging of pumps, and ragging on mixers all impacting the reliability and operability of the system.



*Bypass Events Count:*
*2017: 20*
*2018: 33*
*2019: 29*

**Figure 4.4-1**
**West Side Historic Influent Flows (2017-2019)**

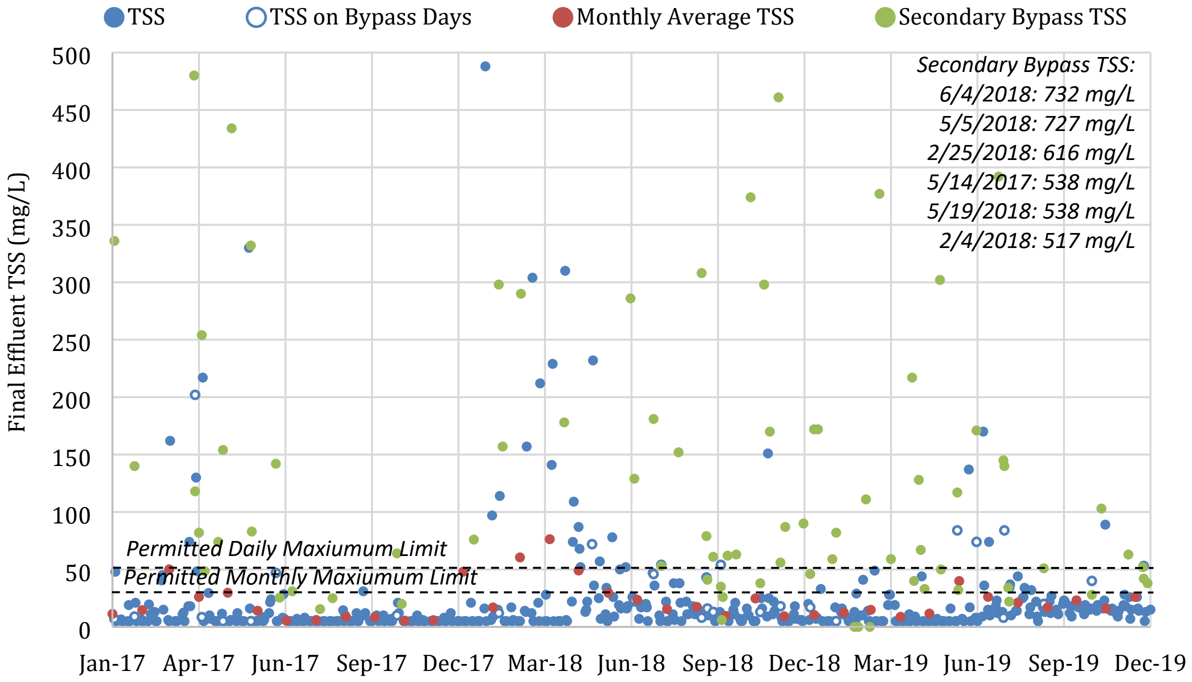


*Secondary Bypass BOD5:*
*7/12/2019: 353 mg/L*
*4/4/2017: 210 mg/L*
*3/27/2017: 197 mg/L*
*6/19/2019: 181 mg/L*

*BOD5:*
*1/24/2018: 181 mg/L*

**Figure 4.4-2**

**West Side Historic Effluent BOD$_5$ Concentration (2017-2019)**



*Secondary Bypass TSS:*
*6/4/2018: 732 mg/L*
*5/5/2018: 727 mg/L*
*2/25/2018: 616 mg/L*
*5/14/2017: 538 mg/L*
*5/19/2018: 538 mg/L*
*2/4/2018: 517 mg/L*

**Figure 4.4-3**

**West Side Historic Effluent TSS Concentration (2017-2019)**

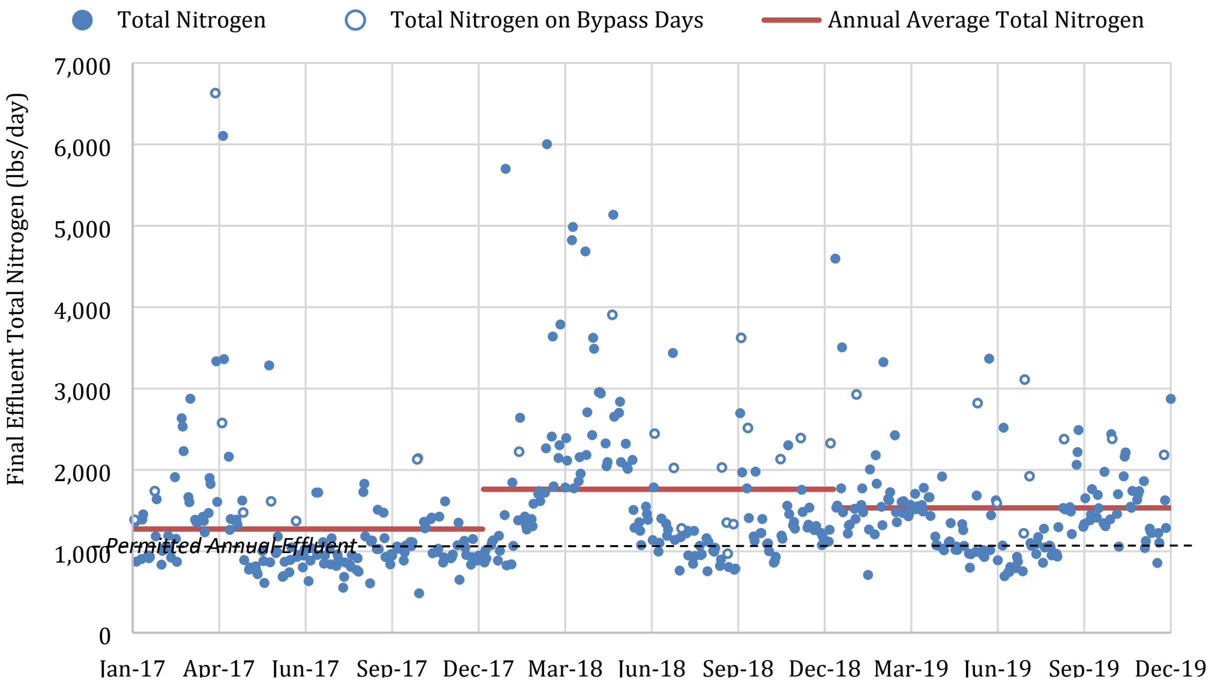


**Figure 4.4-4**
**West Side Historic Effluent Total Nitrogen Load (2017-2019)**

### *Grit Removal*

The performance of a grit removal system is also typically characterized by the quantity of grit removed per million gallons of wastewater treated. Grit removal is notoriously difficult to quantify. A grit characterization study should be conducted prior to any grit removal upgrade for a better understanding of influent grit, removal expected and sizing of grit handling equipment. Manual of Practice 8 (MOP-8) states that a combined sewer system can expect 0.5 to 20 cf/MG treated. It is likely that grit yield increases with wet weather flow due to the flushing effect of a wet weather event. For a number of larger combined systems in New England grit quantities have ranged from 1 to 2 cf/MG treated. Based on this, the West Side plant would be expected to remove about 0.8 to 1.6 cubic yards of grit per day. Data regarding grit removal from the sludge degritters is not available.

The implications of poor grit removal include excessive wear on, and therefore shortened life of downstream pumps and equipment, build-up of grit in pipes and channels impacting plant hydraulics and gate operability, high grit loadings during peak flows that overcome sludge collection mechanisms and clog of sludge pumps, and build-up of grit in aeration tanks and sludge holding tanks.

### *Primary Clarifiers*

The performance of a primary clarifier is represented as percent $BOD_5$ and percent TSS removed across the system, and percent solids of the primary sludge.  Typically, a well operated conventional primary clarifier can achieve 30 to 40% removal of $BOD_5$ and 60 to 70% removal of TSS. Sludge removed from a primary clarifier typically has a concentration of 2 to 4 percent solids. Percent removal is calculated by knowing influent and effluent concentrations into and out of the primary clarifiers. At the West Side plant this calculation is complicated by the sampling and flow measurement locations and sidestream returns, as well as the fact that the flow and concentration of the sidestreams are not measured. As shown on Figure 4.2-1, influent samples are measured before septage and sidestreams from the GTs and RDT are introduced. Influent flow is measured after the introduction of this extraneous flow. The primary effluent sample is also located downstream from an alternative discharge location of GT sidestream. The vast majority of sidestream flow comes from the gravity thickener decant, however, since the gravity thickeners are under performing, (due, in part, to the under performance of the primary clarifiers), the return contains a higher quantity of solids than desired. So, in effect, the solids are circulated in the treatment system, until a high flow even flushes them out the outfall. Using the best available information, performance of the primary clarifiers is presented in **Table 4.4-3.** As shown, the negative percent removal is a function of sampling locations, however, this is extreme. In addition, to poor percent removal rates, the solids concentration of the primary sludge is extremely low (<0.1 percent). It has been indicated by the operators that they like to keep the primary sludge concentration at about 1% solids to improve the function of the sludge degritters.

**Table 4.4-3 West Side Primary Clarifier Removal Efficiency**

| Constituent | Percent Removal |
|---|---|
| $BOD_5$ | -14% |
| TSS | -64% |
| TKN | 0% |

As mentioned in Section 4.2 the primary clarifiers at the West Side plant are grossly undersized, based on the operating overflow rate. The implications of this is that more $BOD_5$ and TSS is transmitted to the undersized secondary treatment system.

### *Secondary Treatment*

All things considered, the WPCA does a reasonable job achieving NPDES permit compliance for $BOD_5$ and TSS, given the age, condition and size of the existing system. The fact the plant is operating 25% below its design flow capacity helps the situation. When the West Side plant is not bypassing secondary treatment, the average final effluent $BOD_5$ and TSS is shown in **Table 4.4-4** and equates to approximately 89 to 95% $BOD_5$ removal and 85 to 93% TSS removal. Bypass events are described in a subsequent section.

**Table 4.4-4 West Side Average Effluent Quality (no bypass)**

| | Total Flow Treated through Secondary (MG) | Average Final Effluent $BOD_5$ (mg/L) | Average. Final Effluent TSS (mg/L) |
|---|---|---|---|
| 2017 | 6,591 | 6.6 | 15 |
| 2018 | 7,255 | 14 | 32 |
| 2019 | 7,336 | 8.6 | 17 |

In terms of Total Nitrogen, the annual total pounds of nitrogen discharged as reported to CT DEEP for the nitrogen credit program over the last seven years is presented in **Figure 4.4-5.** Again, since the interim plant upgrades were likely not designed to achieve complete nitrogen removal under all flow and load events, especially during the cold weather months, the WPCA does a reasonable job removing nitrogen, however, the effectiveness has fallen off in recent years as further described in Appendix F.

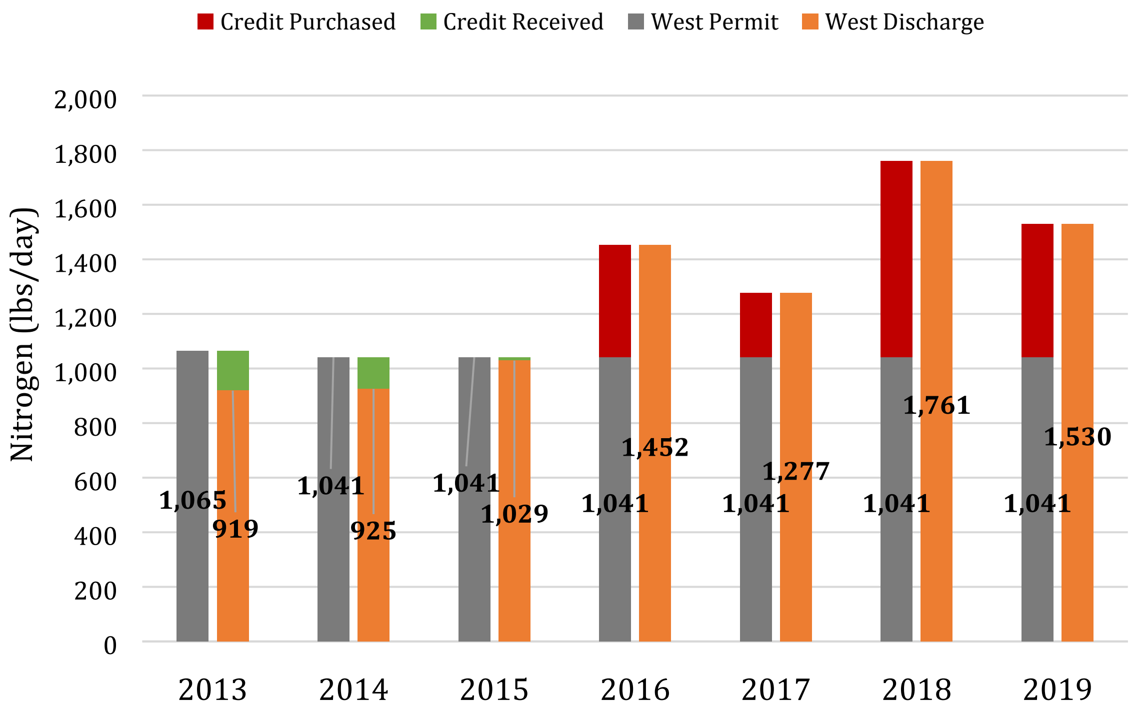


**Figure 4.4-5**
**Historic Total Effluent Nitrogen – West Side Plant**

As discussed in Section 4.2, it is likely that the aeration efficiency at the West Side plant suffers from leaking aeration piping and the lack of automated controls. However, this is difficult to assess due to the lack of data regarding air flow from the blower system and air flow to the aeration tanks.

### *Disinfection*

As presented in Table 4.4-2 (above) the West Side plant has exceeded both the fecal coliform but more frequently the enterococci disinfection limits. So, although the existing chlorine contact tanks provide more than adequate detention time, the dimensioning of the tanks (which were originally primary settling tanks) does not allow for efficient or effective disinfection. This is further demonstrated by assessing the average sodium hypochlorite dosage as presented in **Table 4.4-5**. Over the three year period sodium hypochlorite dosage ranges from 2 to 8 mg/L, averaging about 4 mg/L, whereas a well operated disinfection system may operate with a dosage of 1 to 2 mg/L. Note that sodium hypochlorite is also added to the RAS to reduce filaments and believed to be included in the usage below. The more sodium hypochlorite added, typically the more sodium bisulfite is required to quench the chlorine residual.

Table 4.4-5 Historic Sodium Hypochlorite Dosage

| | Total Flow Treated (MG) | Average Sodium Hypochlorite Dosage (mg/L) |
|---|---|---|
| 2017 | 6,728 | 4.4 |
| 2018 | 7,552 | 4.1 |
| 2019 | 7,571 | 3.7 |

### *Bypass Events*

As presented, the West Side WWTP is designed for an average daily flow of 30 mgd, can pass up to 58 mgd through the secondary system, and when influent flows exceed 58 mgd the secondary system is bypassed. The plant was designed to accept a peak flow of up to 90 mgd, however, as presented in **Figure 4.4-1** it is clear that the plant suppresses flow to the plant by partially closing the influent gates when influent flow approaches 80 mgd to avoid flooding in the influent pumping station and/or taxing the treatment system. Throttling the plant influent flow results in system backups which likely results in CSOs in the system. A summary of the bypass events by year (2017-2019), including volume discharged and average primary effluent $BOD_5$ and TSS concentration during the discharge, are presented in **Table 4.4-6**. Details of the individual events are presented in **Table 4.4-7.** As presented, at times, the effluent concentration in the bypass exceeds the average influent concentration, indicative of “washing out” of solids in the primary settling tanks during a storm event. Also, at times, final effluent concentrations exceed primary effluent concentrations indicating wash-out of the secondary clarifiers. By increasing the influent pumping capacity and upgrading and expanding the primary treatment system, while concurrently providing upgraded and expanded sludge processing facilities, the West Side plant can come a long way in decreasing the loading to the receiving waters in the Bridgeport area.

Table 4.4-6 Summary of Historic Bypass Events (2017-2019) – West Side WWTP

| | Number of Bypass Days | Total Flow Bypassed (MG) | Average Primary Effluent $BOD_5$ (mg/L) | Average Primary Effluent TSS (mg/L) |
|---|---|---|---|---|
| 2017 | 20 | 137 | 202 | 344 |
| 2018 | 33 | 297 | 136 | 309 |
| 2019 | 29 | 235 | 170 | 424 |

## Table 4.4-7 Detail of Historic Bypass Events (2017-2019) West Side

| Date | Rain (in) | Total Daily Flow (mgd) | Max Day Flow (mgd) | Bypass Volume (MG) | $BOD_5$ Bypass (mg/L) | TSS Bypass (mg/L) | $BOD_5$ Final Effluent (mg/L) | TSS Final Effluent (mg/L) | $BOD_5$ Final Effluent Load (lbs/day) | TSS Final Effluent Load (lbs/day) | $BOD_5$ Bypass Load (lbs/day) | TSS Bypass Load (lbs/day) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1/3/2017 | 0.76 | 33.5 | 84.5 | 4.80 | 4.00 | 336 | 8.00 | 7.00 | 2,240 | 1,960 | 160 | 13,500 |
| 1/24/2017 | 1.16 | 42.5 | 82.1 | 11.8 | 93.0 | 140 | 7.00 | 9.00 | 2,480 | 3,190 | 9,150 | 13,800 |
| 3/27/2017 | 0.43 | 30.1 | 81.9 | 2.90 | 197 | 480 | | | | | 4,770 | 11,600 |
| 3/28/2017 | 0.93 | 36.8 | 91.7 | 6.10 | 26.0 | 118 | 26.0 | 202 | 7,980 | 62,000 | 1,320 | 6,000 |
| 4/1/2017 | 1.25 | 39.1 | 91.1 | 14.3 | 9.00 | 82.0 | | | | | 1,070 | 9,780 |
| 4/4/2017 | 1.67 | 56.0 | 95.0 | 11.3 | 210 | 254 | 4.00 | 8.50 | 1,870 | 4,200 | 19,800 | 23,900 |
| 4/7/2017 | 0.89 | 38.5 | 72.7 | 22.0 | 9.00 | 49.5 | | | | | 1,650 | 8,990 |
| 4/21/2017 | 0.33 | 26.9 | 75.8 | 3.10 | 1.50 | 74.0 | | | | | 52.0 | 1,910 |
| 4/26/2017 | 0.73 | 26.4 | 58.2 | 5.40 | 44.0 | 154 | 8.00 | 5.00 | 1,760 | 1,100 | 1,980 | 6,940 |
| 5/5/2017 | 1.33 | 36.2 | 94.8 | 6.20 | 50.0 | 434 | | | | | 2,590 | 22,400 |
| 5/14/2017 | 1.23 | 28.1 | 72.7 | 7.30 | 5.00 | 538 | | | | | 304 | 32,800 |
| 5/25/2017 | 0.35 | 30.7 | 74.6 | 1.80 | 93.0 | 332 | 4.00 | 5.00 | 1,020 | 1,280 | 1,400 | 4,980 |
| 5/26/2017 | 0.70 | 41.5 | 77.0 | 4.30 | 42.0 | 83.0 | | | | | 1,500 | 2,980 |
| 6/20/2017 | 0.70 | 20.3 | 26.9 | 3.70 | 50.0 | 142 | 8.00 | 47.0 | 1,350 | 7,960 | 1,540 | 4,380 |
| 6/24/2017 | 0.58 | 25.5 | 76.1 | 1.20 | 31.0 | 26.5 | | | | | 310 | 260 |
| 7/7/2017 | 1.20 | 27.5 | 93.1 | 4.30 | 7.00 | 31.0 | | | | | 251 | 1,110 |
| 8/5/2017 | 0.61 | 22.7 | 73.6 | 1.10 | 6.00 | 16.5 | | | | | 55.0 | 1470 |
| 8/18/2017 | 0.52 | 23.8 | 72.3 | 2.30 | 22.0 | 25.0 | | | | | 422 | 480 |
| 10/24/2017 | 2.74 | 31.4 | 108 | 11.5 | 20.0 | 64.0 | 3.00 | 10.0 | 786 | 2,620 | 1,910 | 6,140 |
| 10/29/2017 | 3.35 | 46.1 | 71.1 | 11.4 | 16.0 | 20.0 | | | | | 1,520 | 1,900 |
| 1/12/2018 | 0.14 | 32.8 | 70.7 | 4.30 | 63.0 | 76.0 | | | | | 2,260 | 2,730 |
| 2/4/2018 | 1.06 | 24.7 | 72.9 | 6.00 | 112 | 517 | | | | | 5,600 | 25,900 |
| 2/7/2018 | 0.84 | 34.0 | 77.4 | 6.60 | 74.0 | 298 | 5.00 | 12.0 | 1,420 | 3,400 | 4,070 | 16,400 |
| 2/11/2018 | 1.18 | 49.2 | 74.2 | 6.60 | 77.0 | 157 | | | | | 4,240 | 8,640 |
| 2/25/2018 | 0.54 | 40.7 | 71.9 | 4.50 | 47.0 | 616 | | | | | 1,760 | 23,100 |
| 3/2/2018 | 1.75 | 60.6 | 77.6 | 18.0 | 42.0 | 290 | | | | | 6,310 | 43,500 |
| 4/16/2018 | 2.19 | 54.5 | 95.5 | 18.1 | 6.00 | 178 | | | | | 906 | 26,870 |
| 5/15/2018 | 0.55 | 28.1 | 73.1 | 3.20 | 26.0 | 727 | 12.0 | 72.0 | 2,810 | 16,900 | 694 | 19,400 |
| 5/19/2018 | 0.51 | 30.7 | 74.5 | 1.90 | 11.0 | 538 | | | | | 174 | 8,530 |
| 6/4/2018 | 0.79 | 30.5 | 75.8 | 5.60 | 26.0 | 732 | | | | | 1,210 | 34,200 |
| 6/24/2018 | 0.54 | 24.4 | 75.1 | 3.80 | 34.0 | 286 | | | | | 1,080 | 9,060 |
| 6/28/2018 | 0.81 | 35.3 | 77.4 | 8.00 | 74.0 | 129 | 8.00 | 23.0 | 2,360 | 6,770 | 4,940 | 8,610 |
| 7/18/2018 | 1.83 | 20.1 | 44.7 | 12.7 | 56.0 | 181 | 22.0 | 46.0 | 3,690 | 7,710 | 5,930 | 19,200 |
| 7/26/2018 | 1.29 | 30.8 | 76.4 | 8.40 | 18.0 | 53.0 | 16.0 | 54.0 | 4,110 | 13,900 | 1,260 | 3,710 |
| 8/13/2018 | 0.88 | 32.9 | 80.4 | 5.20 | 48.0 | 152 | | | | | 2,080 | 6,590 |
| 9/6/2018 | 1.14 | 30.6 | 26.9 | 3.40 | 22.0 | 308 | 5.00 | 8.00 | 1,280 | 2,040 | 624 | 8,730 |
| 9/11/2018 | 1.30 | 25.3 | 58.3 | 11.1 | 61.0 | 79.0 | 26.0 | 43.0 | 5,490 | 9,070 | 5,650 | 7,310 |
| 9/12/2018 | 0.76 | 28.2 | 82.1 | 4.30 | 20.0 | 41.0 | 13.0 | 16.0 | 3,060 | 3,760 | 717 | 1,470 |
| 9/18/2018 | 1.21 | 32.5 | 80.2 | 6.60 | 8.00 | 61.0 | 15.0 | 13.0 | 4,070 | 3,520 | 440 | 3,360 |

## Table 4.4-7 Detail of Historic Bypass Events (2017-2019) West Side (*continued*)

| Date | Rain (in) | Total Daily Flow (mgd) | Max Day Flow (mgd) | Bypass Volume (MG) | $BOD_5$ Bypass (mg/L) | TSS Bypass (mg/L) | $BOD_5$ Final Effluent (mg/L) | TSS Final Effluent (mg/L) | $BOD_5$ Final Effluent Load (lbs/day) | TSS Final Effluent Load (lbs/day) | $BOD_5$ Bypass Load (lbs/day) | TSS Bypass Load (lbs/day) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 9/26/2018 | 3.32 | 52.7 | 75.8 | 26.8 | 39.0 | 35.0 | 24.0 | 54.0 | 10,500 | 23,700 | 8,720 | 7,820 |
| 9/27/2018 | 0.46 | 38.2 | 76.0 | 5.50 | 21.0 | 6.00 | 11.0 | 9.00 | 3,500 | 2,870 | 963 | 275 |
| 9/28/2018 | 0.55 | 39.8 | 79.2 | 5.00 | 3.00 | 26.0 | | | | | 125 | 1,080 |
| 10/3/2018 | 3.80 | 52.6 | 79.1 | 16.2 | 23.0 | 62.0 | 8.00 | 9.00 | 3,510 | 3,950 | 3,110 | 8,380 |
| 10/12/2018 | 1.57 | 38.0 | 74.8 | 14.0 | 44.0 | 63.0 | | | | | 5,140 | 7,360 |
| 10/27/2018 | 0.82 | 39.9 | 79.4 | 7.50 | 138 | 374 | | | | | 8,630 | 23,400 |
| 11/6/2018 | 0.69 | 34.6 | 79.7 | 6.40 | 25.0 | 38.0 | 30.0 | 13.0 | 8,210 | 3,560 | 1,330 | 2,030 |
| 11/10/2018 | 0.77 | 29.0 | 81.3 | 7.70 | 49.0 | 298 | | | | | 3,150 | 19,100 |
| 11/16/2018 | 0.65 | 49.0 | 79.1 | 8.93 | 24.0 | 170 | | | | | 1,780 | 12,600 |
| 11/25/2018 | 1.02 | 39.6 | 79.6 | 7.50 | 84.0 | 461 | | | | | 5,250 | 28,800 |
| 11/27/2018 | 0.96 | 42.8 | 78.2 | 13.8 | 13.0 | 56.0 | 11.0 | 18.0 | 3,760 | 6,160 | 1,500 | 6,450 |
| 12/2/2018 | 1.42 | 49.2 | 79.1 | 13.0 | 45.0 | 87.0 | | | | | 4,880 | 9,430 |
| 12/21/2018 | 1.72 | 51.5 | 77.1 | 14.4 | 27.0 | 90.0 | | | | | 3,240 | 10,800 |
| 12/28/2018 | 0.82 | 43.4 | 77.5 | 11.9 | 66.0 | 46.0 | 7.00 | 17.0 | 2,430 | 5,900 | 6,550 | 4,570 |
| 1/1/2019 | 1.43 | 46.4 | 79.2 | 12.9 | 66.0 | 172 | | | | | 7,100 | 18,500 |
| 1/5/2019 | 0.67 | 43.2 | 77.8 | 4.70 | 66.0 | 172 | | | | | 2,590 | 6,740 |
| 1/20/2019 | 1.36 | 48.5 | 77.0 | 15.0 | 52.0 | 59.0 | | | | | 6,510 | 7,380 |
| 1/24/2019 | 1.86 | 47.2 | 78.8 | 14.5 | 59.0 | 82.0 | 7.00 | 5.00 | 2,650 | 1,890 | 7,140 | 9,920 |
| 2/24/2019 | 0.50 | 42.5 | 78.5 | 5.10 | 33.0 | 111 | | | | | 1,400 | 4,720 |
| 3/10/2019 | 0.44 | 36.7 | 79.7 | 3.20 | 19.0 | 377 | | | | | 507 | 10,100 |
| 3/22/2019 | 0.76 | 41.0 | 79.8 | 8.40 | 78.0 | 59.0 | | | | | 5,460 | 4,130 |
| 4/13/2019 | 0.48 | 38.6 | 72.6 | 4.40 | 31.0 | 217 | | | | | 1,140 | 7,960 |
| 4/15/2019 | 0.47 | 33.7 | 72.1 | 3.90 | 15.0 | 40.0 | | | | | 488 | 1,300 |
| 4/20/2019 | 0.46 | 34.9 | 71.5 | 3.20 | 12.0 | 128 | | | | | 320 | 3,420 |
| 4/22/2019 | 0.59 | 35.7 | 77.5 | 3.10 | 61.0 | 67.0 | | | | | 1,580 | 1,730 |
| 4/26/2019 | 1.12 | 40.1 | 76.9 | 11.9 | 38.0 | 33.0 | | | | | 3,770 | 3,280 |
| 5/12/2019 | 0.97 | 43.9 | 79.4 | 5.90 | 129 | 302 | | | | | 6,350 | 14,900 |
| 5/13/2019 | 0.37 | 40.0 | 80.5 | 6.30 | 37.0 | 50.0 | | | | | 1,940 | 2,630 |
| 5/30/2019 | 0.77 | 32.0 | 80.9 | 5.30 | 107 | 117 | 42.0 | 84.0 | 10,580 | 21,200 | 4,730 | 5,170 |
| 5/31/2019 | 0.59 | 26.0 | 75.2 | 6.40 | 32.0 | 32.0 | | | | | 1,710 | 1,700 |
| 6/19/2019 | 0.10 | 23.6 | 30.4 | 7.00 | 181 | 171 | 42.0 | 74.0 | 7,640 | 13,500 | 10,570 | 9,990 |
| 7/12/2019 | 0.53 | 30.1 | 79.4 | 3.90 | 353 | 392 | | | | | 11,480 | 12,800 |
| 7/17/2019 | 2.06 | 34.0 | 80.3 | 8.30 | 87.0 | 145 | 5.00 | 8.00 | 1,340 | 2,150 | 6,020 | 10,000 |
| 7/18/2019 | 3.38 | 50.1 | 84.9 | 11.3 | 43.0 | 140 | 10.0 | 84.0 | 4,030 | 33,800 | 4,050 | 13,200 |
| 7/22/2019 | 2.27 | 35.0 | 82.7 | 9.70 | 27.0 | 34.0 | | | | | 2,180 | 2,750 |
| 7/23/2019 | 0.60 | 45.2 | 82.0 | 6.00 | 18.0 | 22.0 | 11.0 | 34.0 | 3,980 | 12,300 | 901 | 1,100 |
| 8/28/2019 | 1.10 | 30.9 | 83.5 | 4.88 | 29.0 | 51.0 | 4.00 | 20.0 | 971 | 4,850 | 1,190 | 2,080 |
| 10/17/2019 | 1.35 | 29.5 | 82.4 | 11.5 | 19.0 | 28.0 | 23.0 | 40.0 | 5,310 | 9,240 | 1,820 | 2,690 |
| 10/27/2019 | 2.40 | 43.2 | 83.7 | 13.3 | 28.0 | 103 | | | | | 3,110 | 11,400 |
| 11/24/2019 | 0.89 | 39.3 | 77.0 | 5.26 | 27.0 | 63.0 | | | | | 1,190 | 2,790 |
| 12/9/2019 | 1.46 | 43.8 | 83.5 | 3.40 | 43.0 | 52.0 | | | | | 1,220 | 1,480 |
| 12/10/2019 | 0.59 | 32.1 | 75.9 | 12.0 | 37.0 | 42.0 | 41.0 | 53.0 | 10,400 | 13,400 | 3,700 | 4,200 |
| 12/14/2019 | 1.44 | 66.6 | 84.1 | 24.0 | 28.0 | 38.0 | | | | | 5,600 | 7,610 |

### *Sludge Handling*

As described in Section 4.2 the two existing gravity thickeners were originally designed to thicken a mixture of both primary sludge and waste activated sludge and were operating in such a way (although slightly modified) until recently when the new rotary drum thickener was installed and made operational in the summer of 2020. As discussed above, the performance of the gravity thickeners is compromised due to the extremely thin primary sludge conveyed to these tanks which in essence require the GTs to perform more as a primary clarifier than a gravity thickener. Solids concentration off a gravity thickener is expected to be 3 to 6% solids, but at the West Side thickened sludge concentration averages about 2 to 3% solids. Also as mentioned the solids concentration in the decant is not measured, but in the two-week sampling program conducted as a part of facilities planning decant solids concentration averaged about 140 mg/L. **Table 4.4-8** presents the reported waste sludge generated (primary plus waste activated sludge) at the West Side plant based on operating report. A common engineering guideline for a secondary treatment plant is 1 dry ton of sludge generated per million gallon of flow treated. As presented, data falls well below this range. It is thought that the recycling of solids through the plant results in some level of aerobic digestion of solids in the system which would reduce sludge quantities, and it is also evident under high flow scenarios that solids are washed out of both the primary and secondary clarifiers into the harbor.

**Table 4.4-8 Waste Sludge Generated – West Side Plant**

| | Total Flow Treated (MG) | Waste Sludge Generated (ton/year) | Waste Sludge Generated (ton/MG) |
|---|---|---|---|
| 2017 | 6,728 | 3,167 | 0.44 |
| 2018 | 7,552 | 3,106 | 0.36 |
| 2019 | 7,571 | 3,399 | 0.37 |

### *Power*

The amount of power used by a wastewater treatment facility varies widely based on flow, required permit limits, the need for influent, effluent or intermediate pumping, type of treatment processes employed, and the need for odor control. Typically, power costs at a plant are driven by pumping requirements and the blowers required to provide air to the aeration system. An assessment of the existing power use at the West Side plant was performed and is presented in **Figure 4.4-6**. As presented, power use rose dramatically in the August-October 2018 timeframe, likely in response to the permit violations occurring concurrently, but generally hovers around 800,000 kWh/month. One would typically expect more seasonal variation in electrical use based on plant flow and air requirements, however, because influent flow is suppressed during high flow events and the aeration system is not automatically controlled, the constant power use is likely what would be expected here. For reference, this power use corresponds to normalized use of about 1,300 to 1,400 kWh/MG treated.

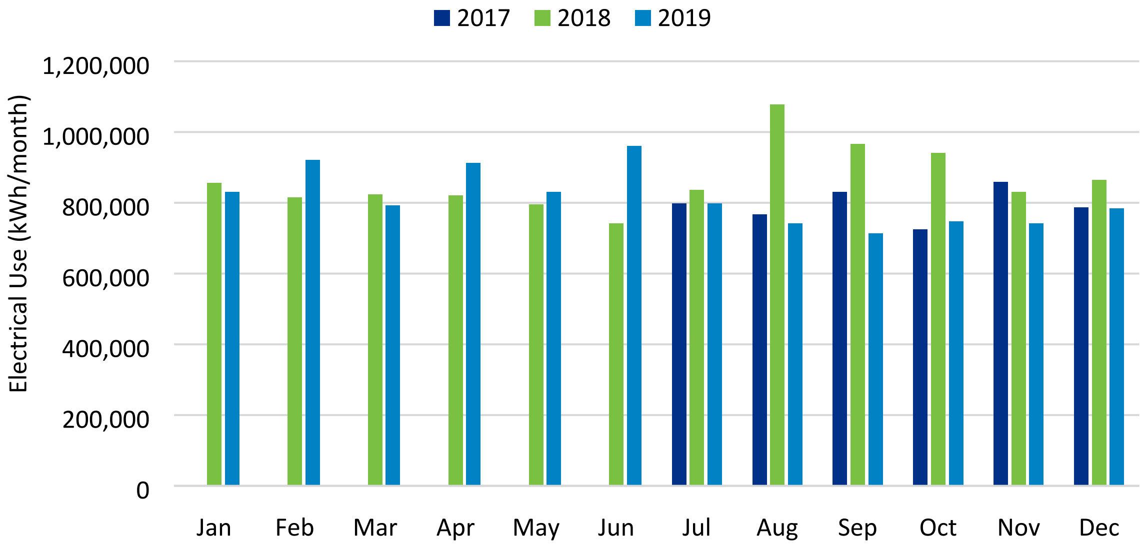


**Figure 4.4-6**
**West Side Electric Use (kWh/month)**

## 4.4.2 East Side Wastewater Treatment Plant

As presented in Section 4.3, the East Side WWTP also suffers from aging, undersized and inadequate treatment processes which directly impact the ability of the treatment facility to meet permit limits, however, the East Side facility is less stressed than the West Side. Similarly, the East Side WWTP includes ineffective screening and no in-line grit removal, slightly undersized primary settling tanks, a secondary treatment system that was also reconfigured to impart nitrogen removal but is undersized to do so under all flow and load conditions year-round, inappropriately sized chlorine contact tanks, and the inability to effectively remove scum and solids from the treatment system, although solids management is more effective at this facility, the East Side WWTP receives scum from the West Side facility which can overload this system. Again, limited instrumentation and controls are functional making operations and control even more difficult.

The assessment of three years of data, revealed data gaps which complicated the data analysis and required certain assumptions to be made to piece together available information.

### 4.4.2.1 Influent Conditions and Effluent Requirements

As detailed further in Section 5, average influent conditions from 2017 through 2019 at the West Side WWTP are presented in **Table 4.4-9.** As presented, the average influent flow for this period was about 43% less than the design influent average daily influent flow of 10 mgd. Both influent $BOD_5$ and TSS concentrations are on the weak side, averaging about 120 and 131 mg/L, respectively, and represent a more typical relationship between the two constituents as compared to the West Side influent conditions. TKN and TP concentrations also run on the weaker side.

Table 4.4-9 East Side WWTP Historic Influent Conditions (2017-2019)

| | East Side WWTP |
|---|---|
| Average daily flow (mgd) | 5.7 |
| Biochemical oxygen demand ($BOD_5$) (lb/day) | 5,700 (120) |
| Total suspended solids (TSS) (lb/day) | 6,200 (131) |
| Total Kjeldahl nitrogen (TKN) (lb/day) | 1,200 (25.2) |
| Total phosphorus (TP) (lb/day) | 160 (3.4) |

[1] concentration (mg/L) shown parenthetically

As presented in Section 2, the plant was designed to achieve conventional secondary treatment standards for $BOD_5$ and TSS: 30/30 mg/L on a monthly average basis. In the early 2000's the aeration system was upgraded to achieve some level of nitrogen removal on an interim basis. Today, an annual average TN load of 362 lb/day is the goal set for the East Side plant, which equates to about 7.6 mg/L under current average daily flow and 4.3 mg/L at design average daily flow. Disinfection limits consistent with coastal outfalls discharging to sensitive receiving waters: 88/100mL fecal coliform and 35/100mL enterococci.

### 4.4.2.2 Permit Compliance

A summary of permit compliance is presented in **Table 4.4-10.** Note this does not include days when permit compliance is waived when excessive flow results in secondary treatment bypass events. As presented, the East Side plant does a commendable job meeting its permit limits. **Figures 4.4-7 through 4.4-10** present daily flow, effluent $BOD_5$, TSS and TN concentrations for the three-year period. The performance of each individual unit process is presented in more detail below.

Table 4.4-10 East Side WWTP Permit Violations (2017-2019)

| Parameter | Permit Limit | Limit Type | Number of Permit Violations[1] | | | |
|---|---|---|---|---|---|---|
| | | | 2017 | 2018 | 2019 | Total |
| $BOD_5$ | 30 mg/L | Maximum Monthly | 0 | 0 | 0 | 0 |
| | 50 mg/L | Maximum Daily | 0 | 0 | 0 | 0 |
| | 85% Removal | Average Monthly Minimum | 0 | 1 | 0 | 1 |
| TSS | 30 mg/L | Maximum Monthly | 0 | 0 | 0 | 0 |
| | 50 mg/L | Maximum Daily | 0 | 1 | 0 | 1 |
| | 85% Removal | Average Monthly Minimum | 0 | 2 | 0 | 2 |
| Fecal Coliform | 88 CFU/100 mL | Monthly Geometric Mean | 0 | 0 | 0 | 0 |
| | 10% of Samples Exceeding 260 colonies/100 mL | Maximum Monthly | 0 | 0 | 0 | 0 |
| | 2,400 CFU/100 mL | Maximum Daily | 0 | 0 | 1 | 1 |
| Enterococci | 35 CFU/100 mL | Monthly Geometric Mean | 1 | 4 | 1 | 6 |
| | 500 CFU/100 mL | Maximum Daily | 1 | 2 | 0 | 3 |
| Total Residual Chlorine | 0.05 mg/L | Average Monthly | 0 | 0 | 0 | 0 |
| | 0.10 mg/L | Maximum Daily | 0 | 0 | 0 | 0 |
| | 0.20 mg/L | Instantaneous | 0 | 0 | 0 | 0 |

[1] Number of permit violations presented represents permit violations recorded in monthly MORs.

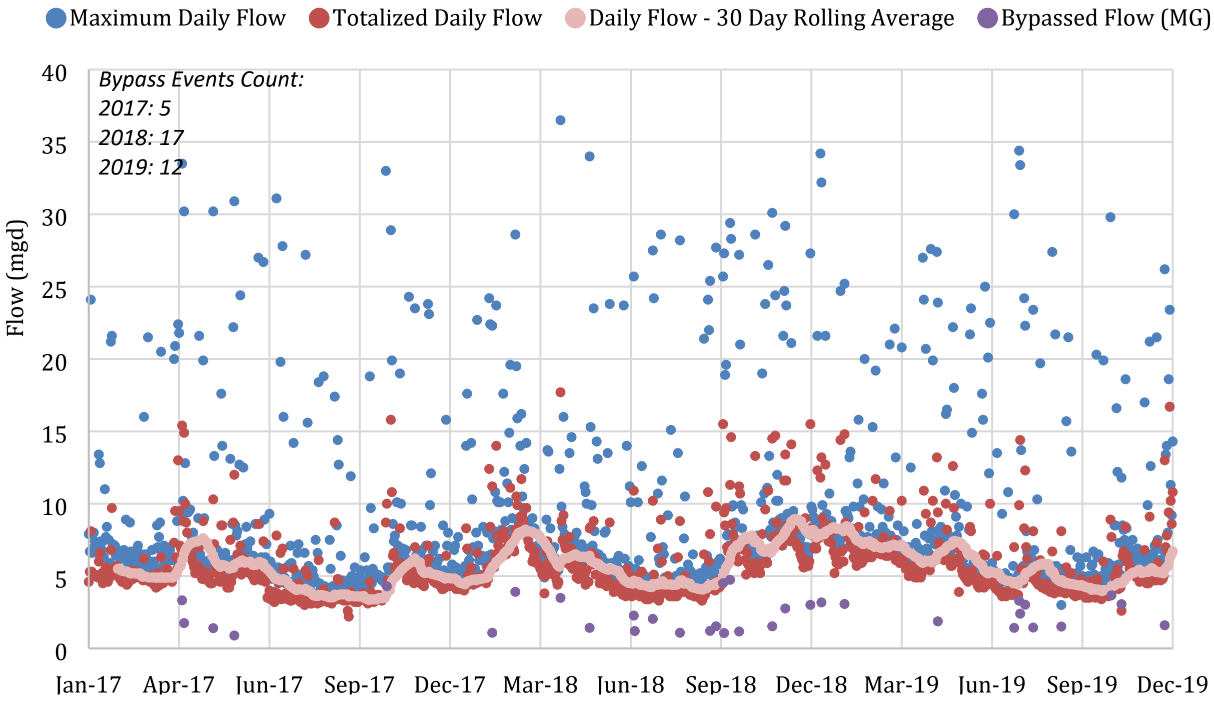


**Figure 4.4-7**
**East Side Historic Influent Flows (2017-2019)**

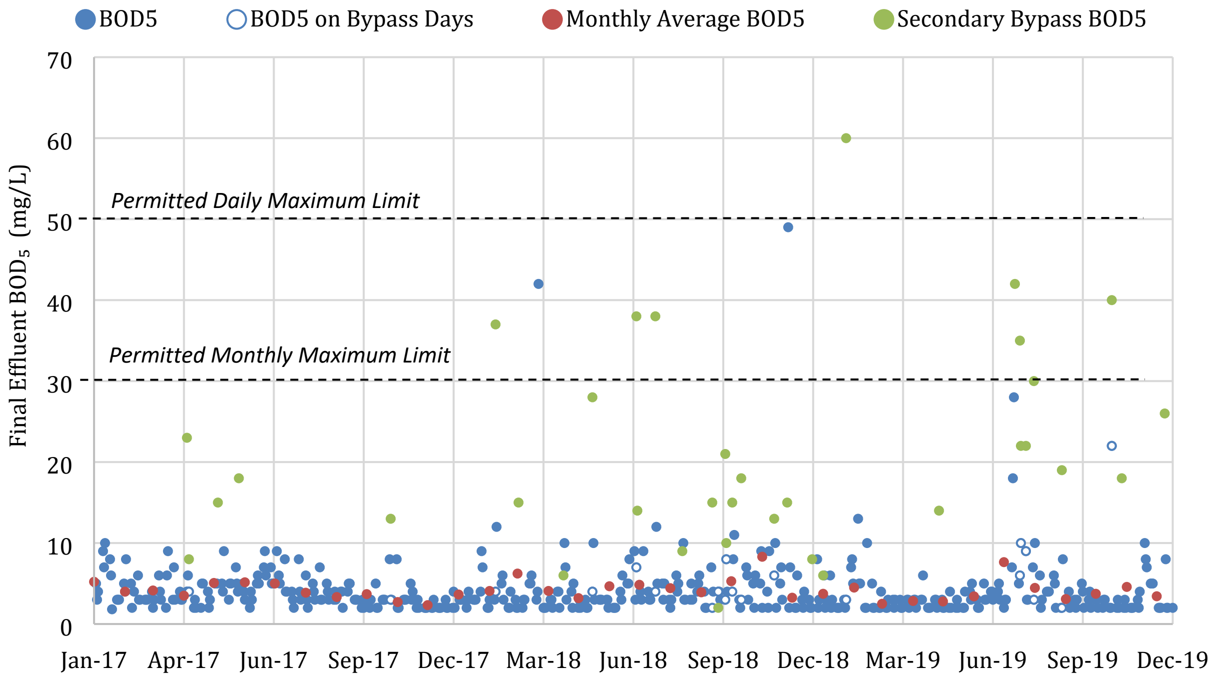


**Figure 4.4-8**
**East Side Historic Effluent BOD$_5$ Concentration (2017-2019)**

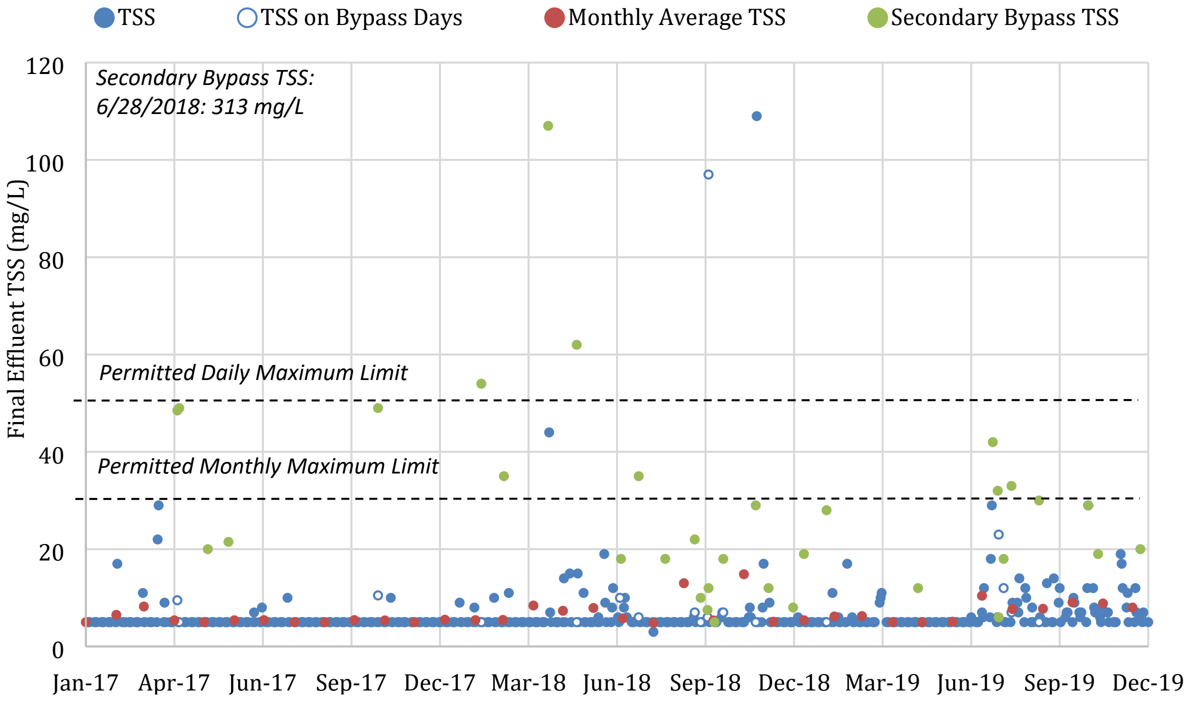


**Figure 4.4-9**
**East Side Historic Effluent TSS Concentration (2017-2019)**

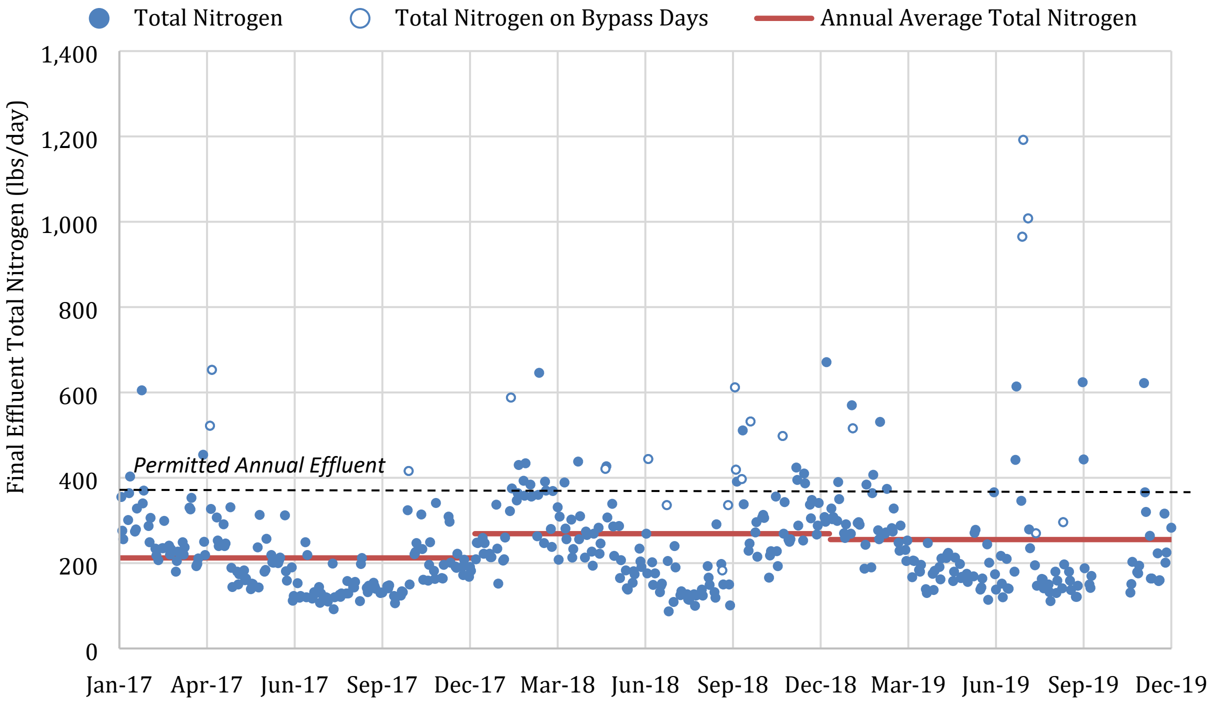


**Figure 4.4-10**
**East Side Historic Effluent Total Nitrogen Load (2017-2019)**

### 4.4.2.3 Unit Process Performance

*Influent Pumping*

As presented in Section 4.3, the influent pumping station with four pumps (9,300 gpm each) has a design firm capacity of 40.2 mgd (peak capacity with one unit out of service). Based on Figure 4.4-7, discussions with plant operators, and assessment of the sewer system modeling it appears that during rain events flow is restricted to the plant by partially closing the influent gates.

*Screening System*

As presented previously, based on industry standards, the typical range of screenings captured in a combined collection system is from 3 to 10 cf/MG treated for a ¾-inch screen. Given the average daily flow at the East Side facility, this would equate to about 0.6 to 2.1 cubic yards per day of screenings. Data on the cubic yards of screenings currently removed and sent to landfill was not readily available, since much of the screenings pass through the screening facility and end up being removed with the scum removed from downstream processes.

The implications of poor screening in the headworks are numerous including fouling of instruments making them inoperable, clogging of pumps, and ragging on mixers all impacting the reliability and operability of the system.

*Grit Removal*

Manual of Practice 8 (MOP-8) states that a combined sewer system can expect 0.5 to 20 cf/MG treated. It is likely that grit yield increases with wet weather flow due to the flushing effect of a wet weather event. For a number of larger combined systems in New England grit quantities have ranged from 1 to 2 cf/MG treated. Based on this the East Side plant would be expected to remove about 0.2 to 0.4 cubic yards of grit per day. Data regarding grit removal from the sludge degritters is not readily available.

The implications of poor grit removal include excessive wear on, and therefore shortened life of downstream pumps and equipment, build-up of grit in pipes and channels impacting plant hydraulics and gate operability, high grit loadings during peak flows that overcome sludge collection mechanisms and clog of sludge pumps, and build-up of grit in aeration tanks and sludge holding tanks.

*Primary Clarifiers*

The performance of a primary clarifier is represented as percent $BOD_5$ and percent TSS removed across the system. Typically, a well operated primary clarifier can achieve 30 to 40% removal of $BOD_5$ and 60 to 70% removal of TSS. Sludge removed from a primary clarifier typically has a concentration of 2 to 4 percent solids. Percent removal is calculated by knowing influent and effluent concentrations into and out of the primary clarifiers. Similar to the West Side plant, at the East Side this calculation is complicated by the sampling and flow measurement locations and sidestream returns, as well as the fact that the flow and concentration of the sidestreams are not measured. As shown on Figure 4.3-1, influent samples are measured before West Side scum decant and sidestream from the GBT are introduced. Influent flow is measured after the introduction of this extraneous flow. The primary effluent sample is also located downstream from the discharge location of GT sidestream. Using the best available information, performance of the primary clarifiers is presented in **Table 4.4-11.** Primary clarifier performance is much better at the East Side plant, however, $BOD_5$ removal is still on the low side. Primary sludge solids concentration averages 1 % solids.

**Table 4.4-11 East Side Primary Clarifier Removal Efficiency**

| Constituent | Percent Removal |
|---|---|
| $BOD_5$ | 16% |
| TSS | 65% |
| TKN | 17% |

### *Secondary Treatment*

The East Side plant does a good job achieving NPDES permit compliance for $BOD_5$ and TSS, given the age and condition of the existing system. The fact the plant is operating well below its design flow capacity helps the situation. When the East Side plant is not bypassing secondary treatment, the average final effluent $BOD_5$ and TSS is shown in **Table 4.4-12** and equates to approximately 96 to 97% $BOD_5$ removal and 95 to 96% TSS removal. Bypass events are described in a subsequent section.

**Table 4.4-12 East Side Final Effluent Quality (no bypass)**

| | Total Flow Treated through Secondary (MG) | Average Final Effluent $BOD_5$ (mg/L) | Average Final Effluent TSS (mg/L) |
|---|---|---|---|
| 2017 | 1,765 | 4.0 | 5.5 |
| 2018 | 2,064 | 4.7 | 6.8 |
| 2019 | 2,051 | 3.7 | 6.8 |

In terms of Total Nitrogen, the annual total pounds of nitrogen discharged as reported to CT DEEP for the nitrogen credit program over the last seven years is presented in **Figure 4.4-11.** Again, since the interim plant upgrades were likely not designed to achieve complete nitrogen removal under all flow and load events, especially during the cold weather months, the WPCA does a commendable job removing nitrogen at the East Side plant.

As discussed in Section 4.3, it is likely that the aeration efficiency at the East Side plant also suffers from leaking aeration piping and the lack of automated controls. However, this is difficult to assess due to the lack of data regarding air flow from the blower system and air flow to the aeration tanks.

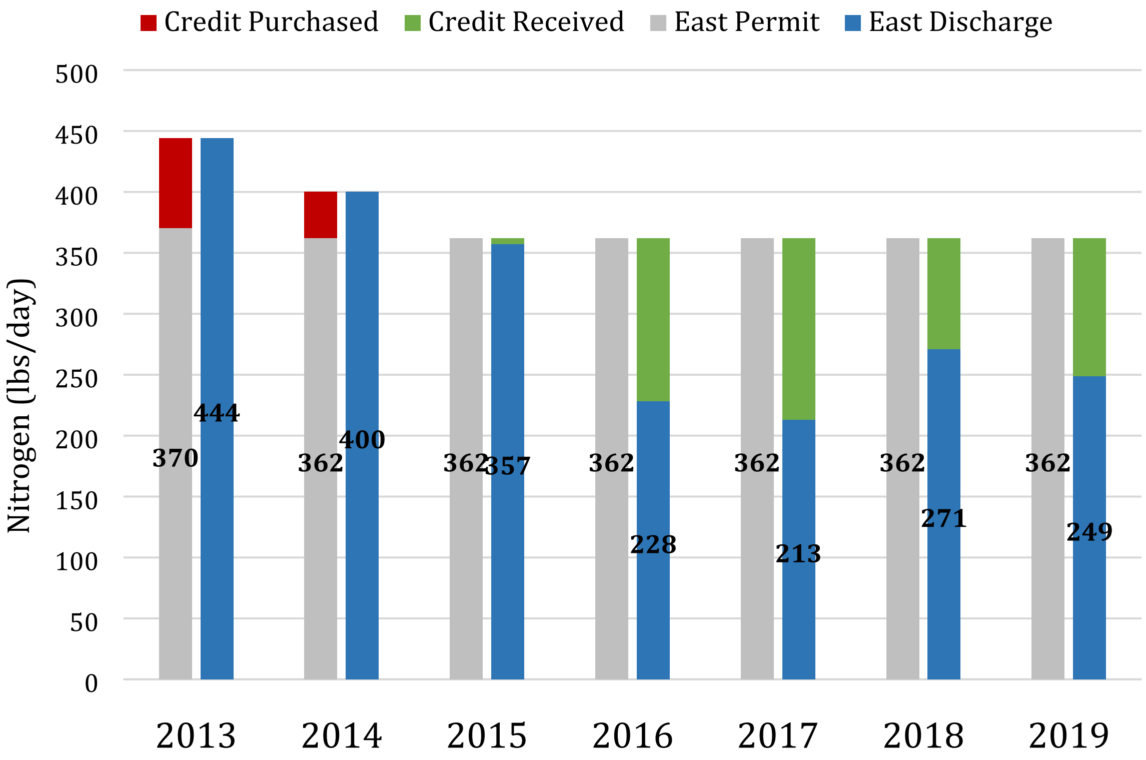


**Figure 4.4-11**
**Historic Effluent Total Nitrogen – East Side Plant**

### *Disinfection*

As presented in Table 4-15 (above) the East Side plant has exceeded both the fecal coliform and enterococci disinfection limits a handful of times over the last three years. So, although the existing chlorine contact tanks provide more than adequate detention time, the dimensioning of the tanks (which were originally primary settling tanks) does not allow for efficient or effective disinfection. This is further demonstrated by assessing the reported sodium hypochlorite dosage as presented in **Table 4.4-13**. Based on the last three years of data, the sodium hypochlorite dosage typically ranges from 2 to 15 mg/L, averaging 5 to 6 mg/L, whereas a well operated disinfection system may operate with a dosage of 1 to 2 mg/L. Note that sodium hypochlorite is also added to the RAS to reduce filaments and believed to be included in the usage below. The more sodium hypochlorite added, typically the more sodium bisulfite is required to quench the chlorine residual.

**Table 4.4-13 Average Sodium Hypochlorite Dosage**

| | Total Flow Treated (MG) | Average Sodium Hypochlorite Dosage (mg/L) |
|---|---|---|
| 2017 | 1,777 | 6.0 |
| 2018 | 2,102 | 5.1 |
| 2019 | 2,081 | 5.2 |

### *Bypass Events*

As presented, the East Side WWTP is designed for an average daily flow of 10 mgd, can pass up to 24 mgd through the secondary system, and when influent flows exceed 24 mgd the secondary system is bypassed. The maximum day flow reported at the East Side facility over the last three years was 36.5 mgd. However, as noted above, influent gates are throttled to limit flow to the plant during high flow conditions. A summary of the bypass events by year (2017-2019), including volume discharged and average primary effluent BOD$_5$ and TSS concentration during the discharge, are presented in **Table 4.4-14**. Details of the individual events are presented in **Table 4.4-15.**

**Table 4.4-14 Summary of Historic Bypass Events (2017-2019) – East Side WWTP**

| | Number of Bypass Days | Total Flow Bypassed (MG) | Average Primary Effluent BOD$_5$ (mg/L) | Average Primary Effluent TSS (mg/L) |
|---|---|---|---|---|
| 2017 | 5 | 12 | 112 | 50 |
| 2018 | 17 | 38 | 89 | 39 |
| 2019 | 12 | 30 | 91 | 48 |

**Table 4.4-15 Detail of Historic Bypass Events (2017-2019) East Side**

| Date | Rain (in) | Total Daily Flow (mgd) | Max Day Flow (mgd) | Bypass Volume (MG) | BOD$_5$ Bypass (mg/L) | TSS Bypass (mg/L) | BOD$_5$ Final Effluent (mg/L) | TSS Final Effluent (mg/L) | BOD$_5$ Final Effluent Load (lbs/day) | TSS Final Effluent Load (lbs/day) | BOD$_5$ Bypass Load (lbs/day) | TSS Bypass Load (lbs/day) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 4/4/2017 | 1.35 | 15.4 | 33.5 | 3.32 | 23.0 | 48.5 | 4.00 | 9.50 | 514 | 1,220 | 637 | 1,340 |
| 4/6/2017 | 1.03 | 14.9 | 30.2 | 1.75 | 8.00 | 49.0 | 4.00 | 5.00 | 497 | 621 | 117 | 715 |
| 5/5/2017 | 1.40 | 10.3 | 30.2 | 1.40 | 15.0 | 20.0 | | | | | 175 | 234 |
| 5/26/2017 | 1.40 | 12.0 | 30.9 | 0.89 | 18.0 | 21.5 | | | | | 134 | 160 |
| 10/25/2017 | 0.03 | 10.0 | 3.80 | 4.29 | 13.0 | 49.0 | 3.00 | 10.5 | 250 | 876 | 465 | 1,750 |
| 2/7/2018 | 0.75 | 11.2 | 22.3 | 1.08 | 37.0 | 54.0 | 4.00 | 5.00 | 374 | 467 | 333 | 486 |
| 3/2/2018 | 0.24 | 8.20 | 28.6 | 3.91 | 15.0 | 35.0 | | | | | 489 | 1,140 |
| 4/16/2018 | 1.80 | 17.7 | 36.5 | 3.49 | 6.00 | 107 | | | | | 175 | 3,110 |
| 5/15/2018 | 0.67 | 8.30 | 34.0 | 1.42 | 28.0 | 62.0 | 4.00 | 5.00 | 277 | 346 | 332 | 734 |
| 6/28/2018 | 0.51 | 10.9 | 25.7 | 2.27 | 38.0 | 313 | 7.00 | 10.0 | 636 | 909 | 719 | 5,930 |
| 6/29/2018 | 0.90 | 4.10 | 5.30 | 1.20 | 14.0 | 18.0 | | | | | 140 | 180 |
| 7/17/2018 | 2.33 | 10.2 | 27.5 | 2.04 | 38.0 | 35.0 | 4.00 | 6.00 | 340 | 510 | 647 | 595 |
| 8/13/2018 | 0.72 | 8.80 | 28.2 | 1.08 | 9.00 | 18.0 | | | | | 81.1 | 162 |
| 9/12/2018 | 0.94 | 7.90 | 25.4 | 1.21 | 15.0 | 22.0 | 2.00 | 7.00 | 132 | 461 | 151 | 222 |
| 9/18/2018 | 1.28 | 9.80 | 27.7 | 1.53 | 2.00 | 10.0 | 4.00 | 5.00 | 327 | 409 | 25.5 | 128 |
| 9/25/2018 | 3.48 | 15.5 | 25.7 | 4.51 | 21.0 | 7.50 | 3.00 | 6.00 | 388 | 776 | 790 | 282 |
| 9/26/2018 | 0.40 | 9.40 | 27.3 | 1.06 | 10.0 | 12.0 | 8.00 | 97.0 | 627 | 7,600 | 88.4 | 106 |
| 10/2/2018 | 3.62 | 11.3 | 29.4 | 4.74 | 15.0 | 5.00 | 4.00 | 5.00 | 377 | 471 | 593 | 198 |
| 10/11/2018 | 1.60 | 11.2 | 27.2 | 1.17 | 18.0 | 18.0 | 3.00 | 7.00 | 280 | 654 | 176 | 176 |
| 11/13/2018 | 0.65 | 14.5 | 30.1 | 1.53 | 13.0 | 29.0 | 6.00 | 5.00 | 726 | 605 | 166 | 370 |
| 11/26/2018 | 0.92 | 13.4 | 29.2 | 2.76 | 15.0 | 12.0 | | | | | 345 | 276 |

Table 4.4-15 Detail of Historic Bypass Events (2017-2019) East Side (*continued*)

| Date | Rain (in) | Total Daily Flow (mgd) | Max Day Flow (mgd) | Bypass Volume (MG) | BOD5 Bypass (mg/L) | TSS Bypass (mg/L) | BOD5 Final Effluent (mg/L) | TSS Final Effluent (mg/L) | BOD5 Final Effluent Load (lbs/day) | TSS Final Effluent Load (lbs/day) | BOD5 Bypass Load (lbs/day) | TSS Bypass Load (lbs/day) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 12/21/2018 | 1.44 | 15.5 | 27.3 | 3.01 | 8.00 | 8.00 | | | | | 201 | 201 |
| 1/1/2019 | 1.47 | 13.2 | 32.2 | 3.18 | 6.00 | 19.0 | | | | | 159 | 504 |
| 1/24/2019 | 1.66 | 14.8 | 25.2 | 3.07 | 60.0 | 28.0 | 3.00 | 5.00 | 370 | 617 | 1,540 | 717 |
| 4/27/2019 | 1.35 | 9.4 | 23.9 | 1.87 | 14.0 | 12.0 | | | | | 218 | 187 |
| 7/12/2019 | 0.65 | 7.00 | 30.0 | 1.42 | 42.0 | 42.0 | | | | | 497 | 497 |
| 7/17/2019 | 1.63 | 9.90 | 34.4 | 3.29 | 35.0 | 32.0 | 6.00 | 6.00 | 495 | 495 | 960 | 878 |
| 7/18/2019 | 1.90 | 14.4 | 33.4 | 2.40 | 22.0 | 6.00 | 10.0 | 23.0 | 1,200 | 2,760 | 440 | 120 |
| 7/23/2019 | 2.54 | 12.3 | 22.3 | 3.02 | 22.0 | 18.0 | 9.00 | 12.0 | 923 | 1,230 | 554 | 453 |
| 7/31/2019 | 0.75 | 8.10 | 23.4 | 1.44 | 30.0 | 33.0 | 3.00 | 7.00 | 203 | 473 | 360 | 396 |
| 8/28/2019 | 1.00 | 8.10 | 3.00 | 1.51 | 19.0 | 30.0 | 2.00 | 5.00 | 135 | 338 | 239 | 378 |
| 10/17/2019 | 1.63 | 7.70 | 5.40 | 3.67 | 40.0 | 29.0 | 22.0 | 29.0 | 1,410 | 1,860 | 1,220 | 888 |
| 10/27/2019 | 2.31 | 2.60 | 11.8 | 3.06 | 18.0 | 19.0 | | | | | 459 | 485 |
| 12/9/2019 | 1.57 | 13.0 | 26.2 | 1.60 | 26.0 | 20.0 | | | | | 347 | 267 |

### *Sludge Handling*

As described in Section 4.3 the two of three existing gravity thickeners are used to thicken primary sludge and waste activated sludge is thickened on a gravity belt thickener. Combined thickened sludge is stored and trucked off-site for disposal. Since operators prefer sending a thin primary sludge to the sludge degritters, primary sludge removed from the primary tanks is about 1% solids and is thickened to about 3.5% solids on the GTs. Waste activated sludge at 0.5 to 1.0% solids is sent to the GBT and thickened to about 4% solids. Sludge pumped to the truck for disposal typically ranges from 4-5% solids. The sludge management at the East Side is better than the West Side, however, due to the lack of redundancy in the GBT and no WAS storage the secondary clarifiers typically operate with a 5-ft blanket making them susceptible to "wash-out" under high flows. **Table 4.4-16** presents the reported waste sludge generated at the East Side plant. A common engineering guideline for a secondary treatment plant is 1 dry ton of sludge generated per million gallons of flow treated. As presented, 2017 and 2019 data appear to fall into this range.

Table 4.4-16 Waste Sludge Generated – East Side Plant

| | Total Flow Treated (MG) | Waste Sludge (ton/year) | Waste Sludge (ton/MG) |
|---|---|---|---|
| 2017 | 1,777 | 2,005 | 1.1 |
| 2018 | 2,102 | 1,875 | 0.83 |
| 2019 | 2,081 | 1,872 | 0.86 |

### *Power*

An assessment of the existing power use at the East Side plant was performed and is presented in **Figure 4.4-12**, and generally averages between 400,000 and 500,000 kWh/month. One would typically expect more seasonal variation in electrical use based on plant flow and air requirements, and some higher power use appears to correspond with higher flows in the spring months, however, because the aeration system is not automatically controlled, the constant power use is likely what would be expected here. For reference, this power use corresponds to normalized use of about 2,300 to 2,900 kWh/MG treated which is significantly higher than that measured at the West Side plant. This could be due to less efficient influent pumping, less efficient aeration, and/or more power use for ventilation and odor control at the East Side plant vs. the West Side plant.

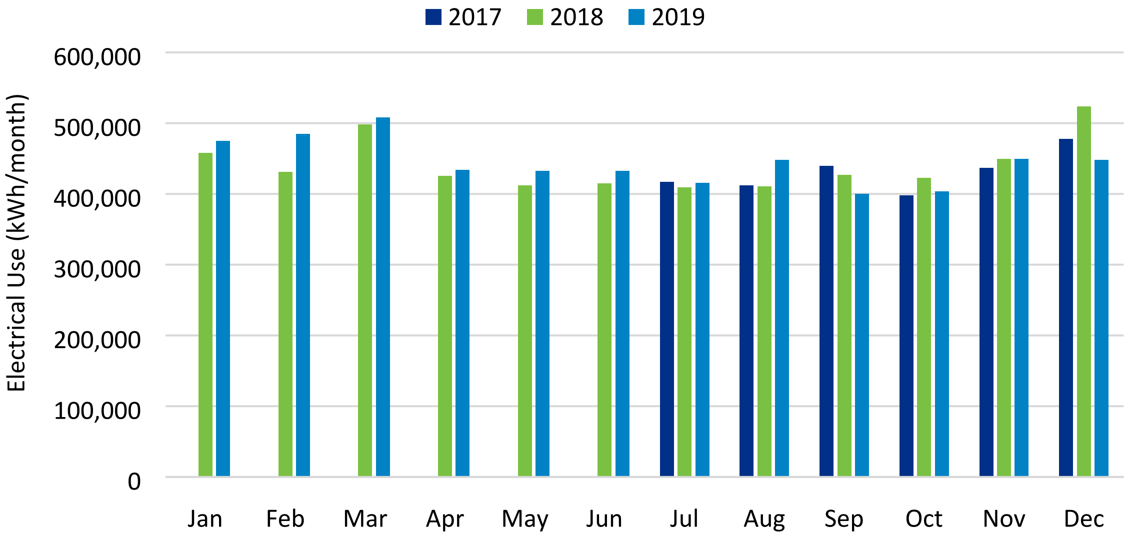


**Figure 4.4-12**
**East Side Electric Use (kWh/month)**

## 4.5 Environmental Site Assessments

As part of the facilities planning process, CDM Smith (assisted by Eolas Environmental LLC) completed a series of environmental investigations at both WWTP sites. These included Phase 1 Environmental Site Assessments, Hazardous Building Material (HBM) Surveys, and Subsurface Investigations. This section provides a summary of the investigation findings. The complete reports are appended to this facilities plan.

### 4.5.1 Phase 1 Investigations

Comprehensive Phase 1 Environmental Site Assessments of both WWTP sites were conducted and complete findings are presented in the West Side and East Side WWTPs Phase 1 Reports included as **Appendix G**. A review of federal, state, and local environmental regulatory databases records and available historical documents was conducted as well as a site reconnaissance visit and interviews with site personnel. A historical narrative of the sites' development was established. Ultimately the review of environmental records led to the identification of specific areas of concern (AOCs). These AOCs helped direct the subsurface investigations at each site, as well as inform the development of the facilities plan recommendations. A summary of the Phase 1 findings at both plants are presented below.

### 4.5.1.1 West Side Phase 1 Summary and Areas of Concern (AOCs)

The West Side WWTP site was comprised of undeveloped land and partly covered with water until it was first developed in 1918. Over time, settling basins, secondary treatment processes, a screening building, an incinerator building, and a sludge processing building were added to the site. The portions of the site covered by water were filled by 1972. The historical incinerator was demolished sometime between 1995 and 2005. The West Side WWTP was identified in multiple regulatory databases. A summary of the major findings and information about key AOCs is provided below (excerpts from the West Side Phase 1 Investigation Report). A table of the fourteen identified AOCs at the West Side WWTP is presented in **Table 4.5-1**, and a summary map of AOC's is included as **Figure 4.5-1**.

- Former USTs: Several former heating oil underground storage tanks (USTs) were identified from historical documentation, and at least one leaky underground storage tank (LUST) was identified in a regulatory database at the site's address.
- Staining: Staining was observed near multiple buildings and loading locations. Any historical release(s) of petroleum products have the potential to migrate into subsurface soils and/or groundwater. A machine shop was also operated in the southern portion of the raw sewage pump station building from circa 1939 until circa 1972.
- Historical Incinerator: In 1957 an incinerator building was built on the western portion of the site. The incinerator building was razed sometime between 1995 and 2005. Runoff associated with potentially contaminated incinerated wastes and ash generated in this building has the potential to adversely affect underlying shallow soil via migration from the surface to underlying soil and groundwater.
- Uncharacterized Fill: An area of fill along the western portion of the site is present, and its composition is unknown. Additionally, according to historical record sources, the site was partly covered by water in 1891. By 1959, portions of the site that had been below water were filled. The composition and quality of any fill material is unknown and represents a potential source of contaminants in the subsurface.
- Sewage Holding Pond and Chambers: According to historical record sources, it appears a series of holding ponds and detention chambers were present south of the raw sewage pump station building between approximately 1949 and 1959. The materials stored would have the potential to adversely affect underlying soils and/or groundwater.
- Upgradient Migration of Contaminated Groundwater: According to an Environmental Conditions Assessment Form (ECAF) document submitted for the adjacent property to the north, ash generated from the historical incinerator located on the site was believed to have been placed on the property. Investigations of this property resulted in the identification of petroleum hydrocarbons, metals, polychlorinated biphenyls (PCBs), and polyaromatic hydrocarbons (PAHs) in soil, and petroleum hydrocarbons, volatile organic compounds (VOCs), and PAHs in groundwater. Based on a presumed southerly groundwater flow direction, contaminated groundwater from this property is likely migrating onto the West Side WWTP site.

**Table 4.5-1 West Side WWTP Areas of Concern (AOC)**

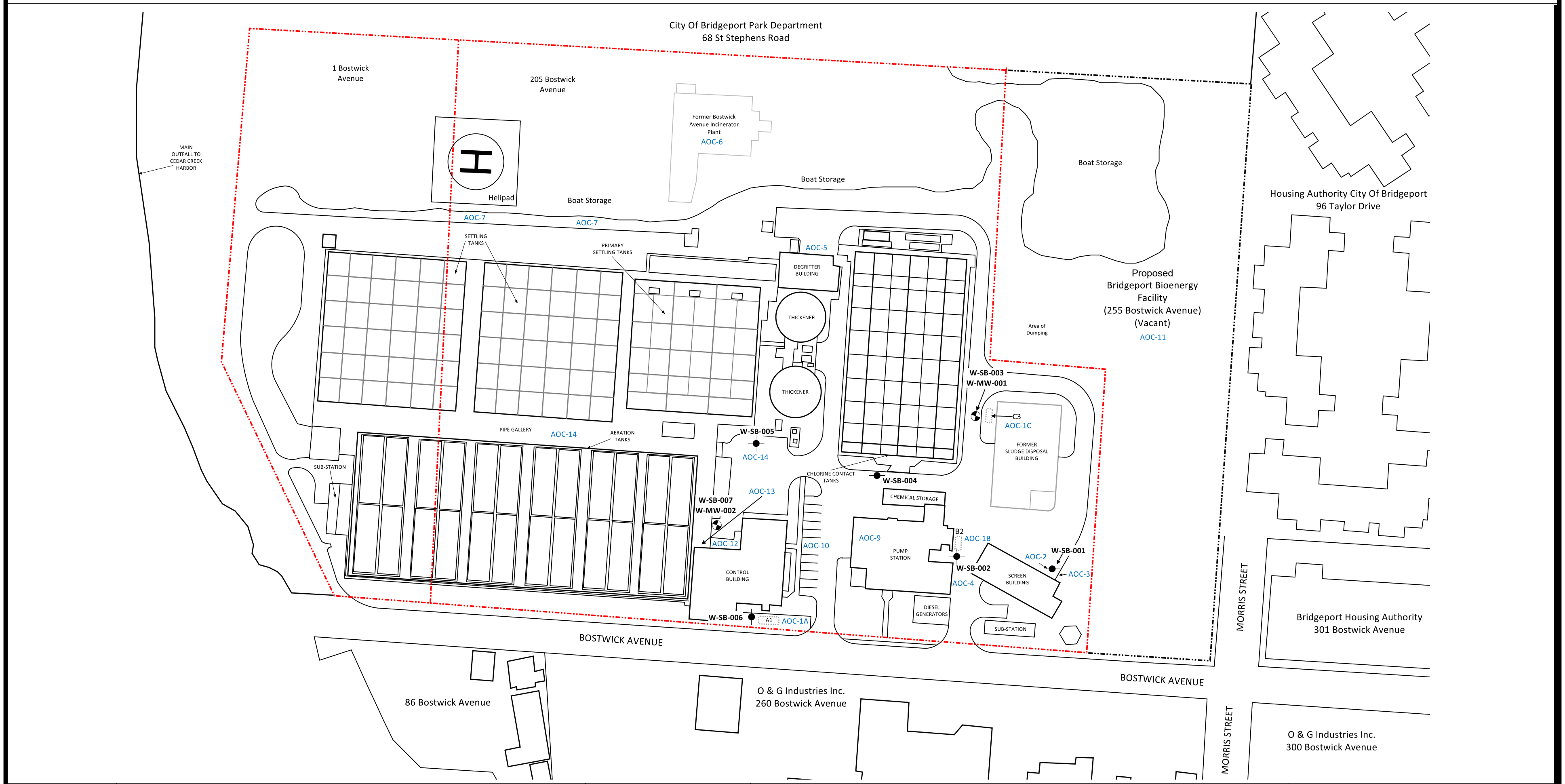
| AOC | Description |
|---|---|
| AOC-1A | Former Heating Oil UST – Tank A1, Northwest corner of Control Building |
| AOC-1B | Former Heating Oil UST – Tank B2, North of Pump Station |
| AOC-1C | Former Heating Oil UST – Tank C3, North of Chlorine Contact Tanks, Off southwest corner of former Sludge Disposal Building |
| AOC-2 | Septic Dump Station, Adjacent to Northwest corner of Screen Building |
| AOC-3 | Screen Building Staining, Adjacent to Screen Building |
| AOC-4 | Pump Station Screen Loading Area, Northeast corner of Pump Station |
| AOC-5 | Degritter Staining, Adjacent to Degritter Building |
| AOC-6 | Historical Incinerator, Western portion of the property |
| AOC-7 | Uncharacterized Fill, Area west of Settling Tanks |
| AOC-8 | Leaking Underground Storage Tank (LUST), Location Unknown |
| AOC-9 | Former Machine Shop, Pump Station Building |
| AOC-10 | Historical Sewage Holding Pond and Chambers, South of Pump Station Building |
| AOC-11 | Upgradient Migration of Contaminated Groundwater from parcel to the North, Northern Portion of Site |
| AOC-12 | Loading/Unloading Dock, West Side of Control Building |
| AOC-13 | Oil Storage Area, West Side of Control Building |
| AOC-14 | Uncharacterized Fill, Sitewide |

### 4.5.1.2 West Side WWTP – Northern Parcel

The 2.2-acre parcel immediately adjacent to the West Side WWTP to the north, with an address of 255 Bostwick Avenue, is currently vacant. This parcel is currently leased to the WPCA by the City of Bridgeport. Circa 2013 to 2016, it was planned to be developed as a Bio-Energy Facility; however, this project was ultimately abandoned. This northern parcel is identified on **Figure 4.5-1**. The parcel was identified in the Connecticut Engineering Controls (CT ENG CONTROLS), Connecticut Voluntary Cleanup Program (VCP), and Connecticut Solid Waste Facility/Landfill (CT SWF/LF) databases.

According to an ECAF prepared by AECOM in 2016, the 2.2-acre northern parcel has never been developed but a portion of the land has been used for storage by a nearby marina. In addition, the ECAF reports that ash generated from the historical incinerator at the West Side WWTP was believed to have been placed as fill on this northern parcel. Subsurface investigations of the property conducted by AECOM between 2013 and 2016 identified the presence of petroleum hydrocarbons, metals, PCBs, and PAHs in soil and petroleum hydrocarbons, VOCs, and PAHs in groundwater. Groundwater flow direction was presumed to be toward the south, in the direction of the existing West Side WWTP and Cedar Creek.

While this known area of contamination was included as an AOC in the Phase 1 environmental site assessment for the West Side WWTP, additional subsurface work was not completed in this area at the time of this facilities plan. It should be noted that this northern parcel is owned by the City of Bridgeport and is considered as a location for future WWTP improvements in later sections of this report. It is anticipated that this northern parcel will be investigated further prior to any construction in the area.



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**LEGEND:**

| | |
|---|---|
| -··-··- | PARCEL BOUNDARY |
| ⊕ | SOIL BORING LOCATION |
| ◐ | MONITORING WELL |
| -·-·- | SITE BOUNDARY |
| ▭ | UST (Closed-In-Place) |

| | |
|---|---|
| **PROJECT:** | Subsurface Investigation |
| **SITE LOCATION:** | 205 Bostwick Avenue (Portion) and 1 Bostwick Avenue, Bridgeport, Connecticut 06605 |
| **SOURCE:** | City of Bridgeport GIS |
| **SCALE:** | NOT TO SCALE |

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**SITE PLAN AND SAMPLE LOCATION MAP**

Figure 4.5-1
West Side WWTP, AOC & Subsurface Plan

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### 4.5.1.3 East Side Phase 1 Summary and Areas of Concern (AOCs)

The East Side WWTP site was first developed in approximately 1898 with a coal fired powerhouse on the eastern side of the site. The East Side WWTP was built out from the 1950s to the 1970s with the addition of a pump station, sludge building, incinerator, screening building, primary tanks, and secondary treatment processes. The East Side WWTP site was identified in multiple regulatory databases. A summary of the major findings and information about key AOCs is provided below (excerpts from the East Side Phase 1 Investigation Report). A table of the thirteen identified AOCs at the East Side WWTP is presented below in **Table 4.5-2**, and a summary map of AOC locations is included as **Figure 4.5-2**.

- Active and Former USTs: Several active USTs used for storage of gasoline and diesel were identified. These are used to fuel vehicles serving both the East and West Side WWTPs. Historical heating oil USTs on the site were also identified. Any releases from these USTs and/or ancillary piping have the potential to adversely affect shallow and deeper soil, and groundwater beneath the site.

- Staining: Staining was observed near multiple buildings and loading locations. Any historical release(s) of petroleum projects have the potential to migrate into subsurface soils and/or groundwater.

- Historical Powerhouse: A coal-fired powerhouse was present on the eastern portion of the site between approximately 1898 and 1949. Coal storage and transfer to the powerhouse occurred on the western side of the powerhouse and appears to have been transported to the area of the powerhouse via the powerhouse channel (into which the East Side WWTP currently discharges), located on the southern portion of the site. Storage of coal, operation of boiler units, and operation of ancillary equipment in the powerhouse has the potential to have resulted in a release to the ground, and to have migrated into subsurface soil and groundwater.

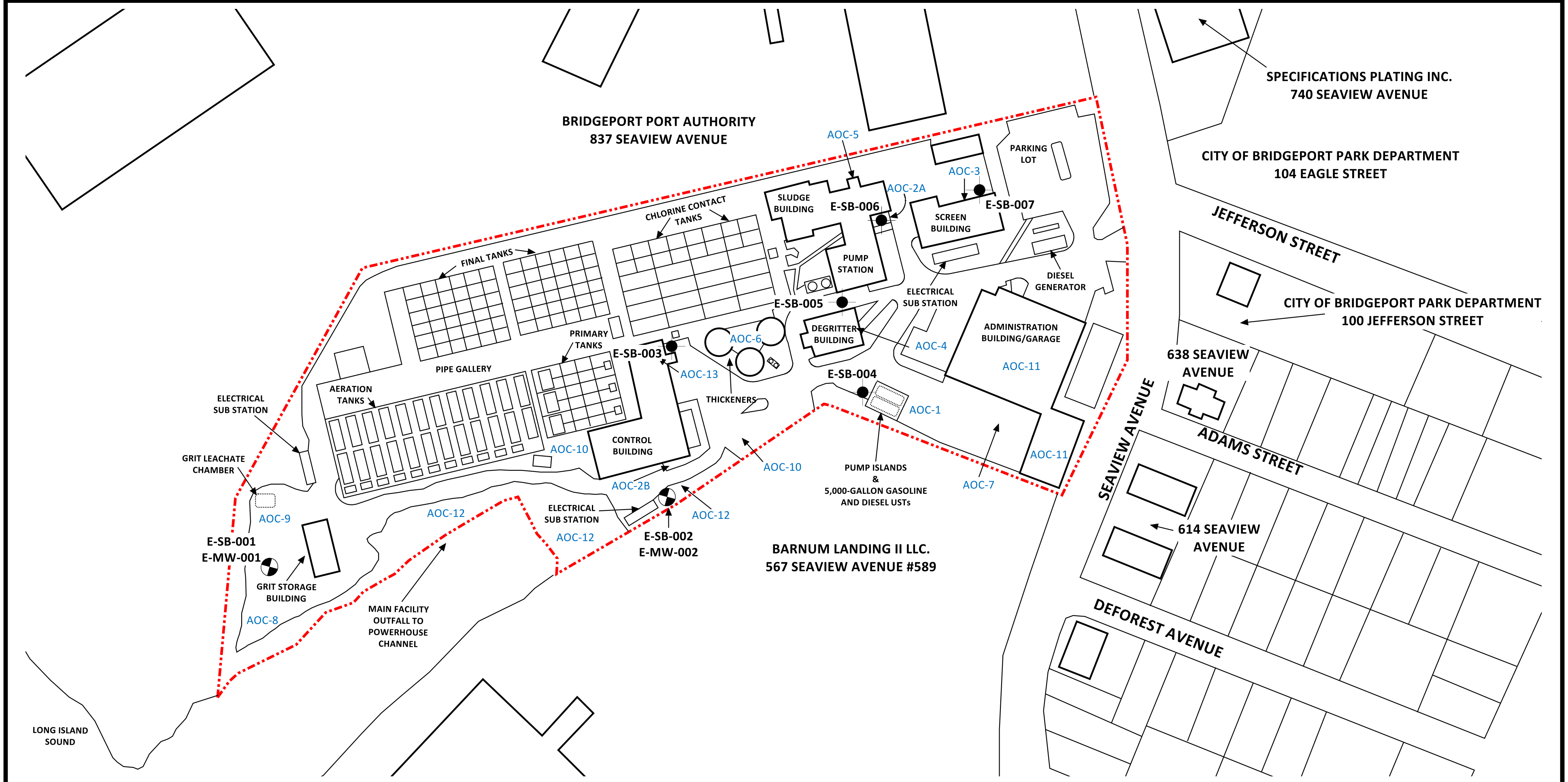
- Uncharacterized Fill/ Filled Inlet: Historical aerial photographs depict areas of fill piles located on the southwestern corner of the site. The composition of this material is unknown. Additionally, the inlet (known as the powerhouse channel) that is presently located to the south of the site formerly extended onto the southern portion of the plant site. Between approximately 1959 and 1979, the portion of the inlet that was located on the site was filled. The composition and quality of the fill materials is unknown and represents a potential source of contaminants in the subsurface.

- Grit Storage and Leachate Chamber: A Quonset hut-style storage building and a grit leachate chamber is present on the southwestern corner of the site. Grit and scum from both the East Side Plant and West Side Plant, along with collection system debris, is temporarily stored in this area to await loading, and transportation and disposal at a licensed disposal facility. Based on the nature of materials and influent entering the site, this material may contain a variety of contaminants.

- Hazardous Waste and Used Oil Storage Area: Bulk storage and use of virgin motor oil, hydraulic oil, transmission oil, antifreeze, foam vehicle cleaner, tracer dye, citrus degreaser, diesel fuel, gear oil, and waste oil is present in the garage and annex on the eastern portion of the site. Several flammables cabinets and work bench tops were observed to contain smaller quantities of oils and cleaners in both areas. Staining consistent with vehicle servicing and maintenance operations was observed on the floor of the garage and services bays in the annex. Any release(s) in this area has the potential to migrate into soils and/or groundwater.

**Table 4.5-2 East Side WWTP Areas of Concern (AOC)**

| AOC | Description |
|---|---|
| AOC-1 | Current Gasoline and Diesel USTs, Pump Island South of Degritter Building |
| AOC-2A | Former Heating Oil UST – Tank A1, East of Pump Station |
| AOC-2B | Former Heating Oil UST – Tank B2, South of Control Building |
| AOC-3 | Screen Building Staining, Adjacent to Screen Building |
| AOC-4 | Degritter Staining, Adjacent to Degritter Building |
| AOC-5 | Sludge Building Loading Platform, North side of Sludge Building |
| AOC-6 | Oil Staining Sludge Thickener Floor, Gravity Thickeners |
| AOC-7 | Historical Powerhouse, Eastern Side of Site |
| AOC-8 | Uncharacterized Fill, Southwest Corner of Site |
| AOC-9 | Grit Storage and Leachate UST, Southwest Side of Site |
| AOC-10 | Filled Inlet, South Side of Site |
| AOC-11 | Equipment Maintenance and Oil Storage, Maintenance Garage |
| AOC-12 | Exterior Materials Storage and Surface Staining, South Side of Site |
| AOC-13 | Hazardous Waste and Used Oil Storage Area, North Side Control Building |



N

**LEGEND**

-·-·-·- PARCEL BOUNDARY
⊕ SOIL BORING LOCATION
◐ MONITORING WELL
-·-·-·- SITE BOUNDARY
▭ UST (Closed-In-Place)

| | |
|---|---|
| **PROJECT:** | Subsurface Investigation |
| **SITE LOCATION:** | 601 (695) Seaview Avenue, Bridgeport, Connecticut |
| **SOURCE:** | City of Bridgeport GIS |
| **SCALE:** | NOT TO SCALE |

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**SITE PLAN AND SAMPLE LOCATION MAP**

Figure 4.5-2
East Side WWTP, AOC & Subsurface Plan

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## 4.5.2 Hazardous Building Material Surveys

The purpose of a hazardous building material (HBM) survey is to identify potentially hazardous materials that could be an obstacle during construction due to specialized handling or disposal processes. For the purposes of this facilities plan, a limited HBM survey was completed to identify potential areas that could pose an issue to any future WWTP upgrade alternatives.

Samples for asbestos and PCB containing material were collected for laboratory analysis, while lead screening was completed using X-Ray Fluorescence (XRF) instrumentation in the field. A visual inventory of other miscellaneous HBM was also completed. These investigations were completed at both the West and East Side WWTPs. The full HBM survey reports, including sample locations, field sampling results, laboratory results, and certification documentation are included in **Appendix H**.

A brief summary of the HBM survey findings at each WWTP is included in the following sections. It should be noted that limited samples of each material were taken from limited locations on the site. This effort was intended to guide the facilities planning process and help in screening different alternatives for development. These results should be considered preliminary, and more comprehensive investigation and testing will need to be conducted prior to any renovation or demolition of existing structures.

### 4.5.2.1 West Side Hazardous Building Material Survey

The HBM survey was completed at the West Side WWTP on February 18, 19, and 20, 2020. This survey identified lead, and PCB containing materials throughout the plant. No asbestos containing material (ACM) was detected during the limited testing program, although historical information indicates the asbestos containing materials have been identified at the West Side WWTP in the past. Lead based paint was detected in several different locations throughout the plant, including the control building, pipe gallery, and raw sewage pump station. PCBs were detected in three of the five locations tested and was found in paint and caulk in the control building, raw sewage pump station and screening building. The highest PCB concentration of 9,300 milligrams per kilogram (mg/kg) was detected on wall paint within the raw sewage pump station building. The other two PCB concentrations were below 50 mg/kg.

Due to the preliminary PCB results at the West Side WWTP, 29 supplementary PCB samples were collected on June 25, 2020. Of the 29 samples collected and analyzed, 24 contained PCBs at concentrations above laboratory detection limits and concentrations in six of the samples were greater than 50 mg/kg. This secondary round of PCB sampling identified additional areas of the West Side WWTP where PCB concentrations were found greater than 50 mg/kg, including additional paint locations in the raw sewage pump station building and in the caulk on the exterior of the control building. Notification to US EPA and CT DEEP was required due to concentrations greater than 50 mg/kg, which US EPA designates as “unauthorized use”. CT DEEP prohibits the use of PCBs at levels above 1 mg/kg.  Both agencies require action if concentrations are detected beyond these levels. The complete PCB results (initial and supplementary samples) are included in a notification letter from the Bridgeport WPCA to US EPA and CT DEEP (included in **Appendix H**).

### 4.5.2.2 East Side Hazardous Building Material Survey

The HBM survey was completed at the East Side WWTP on February 20, 21, and 25, 2020. This survey identified asbestos, lead, and PCB containing materials throughout the plant. ACMs were identified in the incinerator building, sludge building, degritter building, and control building. Lead based paint was detected in the incinerator building, sludge building, and control building. PCBs were detected in four of the five samples collected and was found in paint, caulk and window glaze in the control building, pipe gallery, sludge building and primary tank building. The highest PCB concentration of 2,000 mg/kg was detected in expansion joint caulk at the control building. The other three detected PCB concentrations were below 50 mg/kg. The full East Side WWTP HBM survey report, including sample locations, field sampling results, laboratory results, and certification documentation is included in **Appendix H**.

Following the initial HBM survey findings, 31 supplementary PCB samples were collected on June 25, 2020. Of the 31 samples collected and analyzed from the East Side Plant, 21 (including one shallow soil sample) contained PCBs at concentrations above laboratory detection limits and concentrations in four of the samples were greater than 50 mg/kg. The secondary round of PCB sampling identified other areas where PCB concentrations were found greater than 50 mg/kg including more areas of expansion joint caulk on the exterior of the control building, and window glaze on the exterior of the sludge building. The complete PCB results (initial and supplementary samples) are included in the attached notification letter from the Bridgeport WPCA to US EPA and CT DEEP (included in **Appendix H**). Lower level results (less than 50 mg/kg) were also found in multiple other areas throughout the East Side WWTP.

### 4.5.2.3 Ongoing Characterization

The WPCA has informed employees of the presence of the PCB materials throughout both the East and West Side WWTPs. The locations have been marked and labeled. The WPCA is currently working with both the US EPA and CT DEEP to further evaluate and mitigate any potential risk posed by these materials.

It is also important to note that a limited number of samples were taken of each type of material. Although a second round of PCB sampling occurred, these efforts were intended only to guide the facilities planning process. These results should be considered preliminary, and more comprehensive investigation and testing will be needed prior to any renovation or demolition of existing structures.

## 4.5.3 Subsurface Investigations

Based on the findings of the Phase 1 Environmental Site Assessment and the AOCs that were identified during this initial investigation, subsurface investigations were targeted in order to further evaluate select AOCs. Soil and groundwater samples at both sites were evaluated for various constituents of concern (COCs). A summary of the subsurface investigations at both the West Side and East Side WWTP are presented in the subsequent sections. Locations of soil and groundwater sampling locations at the West Side WWTP and East Side WWTP are included on **Figure 4.5-1** and **Figure 4.5-2,** respectively. The complete subsurface investigation reports are included as **Appendix I**.

The environmental analytical results were compared to the Connecticut Remediation Standard Regulations (RSRs) (RCSA Section 122k-1 to 122k-3). The RSRs include baseline numerical criteria that may be used at a property to determine whether remediation is necessary. Since the site has not engaged in a formal State cleanup program, the RSRs may not apply but are used for guidance purposes only. Comparison to the CT RSR criteria was used to provide a baseline understanding and guidance relative to potential environmental concerns and exposures that may exist at the Site.

Soil remediation criteria established in the RSRs are risk-based and designed to (1) protect human health and the environment from risks associated with direct exposure and (2) protect groundwater quality from contaminants that may migrate into from soil into groundwater. Relative to protection of human health and the environment from risks associated with direct exposure, the CT DEEP established two sets of criteria using exposure assumptions based on land use type; these include the Residential Direct Exposure Criteria (RDEC) and Industrial/Commercial Direct Exposure Criteria (IDEC).

Relative to protection of groundwater from migration or leaching of soil contaminants into groundwater, the CT DEEP established the pollutant mobility criteria (PMC), further classified by the quality and classification of groundwater (i.e. GA, GB). The site is located in an area with a GB classification. In general, the PMC applies to all soil in the unsaturated zone from the ground surface to the seasonal high-water table in GB-classified areas.

The soil data collected from the subsurface investigation are compared to the RDEC, IDEC, and the GB PMC of the RSRs to provide an understanding of the magnitude of concentrations of constituents detected in soil to criteria established by the State of Connecticut as protective of human health and the environment, and protective to groundwater. When assessing general groundwater remediation requirements, multiple factors must be considered in conjunction with the major numeric components of the RSRs. The RSRs include the following criteria: Groundwater Protection Criteria (GWPC), Surface Water Protection Criteria (SWPC), and Groundwater Volatilization Criteria (VC) further classified by land use (i.e. residential or industrial/commercial). Because the site is located in a GB-classified area, the groundwater data collected from the subsurface investigation were compared to the SWPC and the Residential and Industrial/Commercial VC of the RSRs to provide an understanding of the magnitude of concentrations of constituents detected in groundwater to criteria established by the State of Connecticut as protective of groundwater and surface waters.

### 4.5.3.1 West Side Subsurface Investigation (Soils)

A total of seven borings were advanced, and soil samples were collected between 6 to 8 feet below ground surface (bgs) and 10-12.5 bgs. Major findings from the soil sampling effort are included below (excerpts from the West Side Subsurface Investigation Report).

**VOCs (Volatile Organic Compounds):**

- 3 low level detections of 1,2,4-trichlorobenzene, chloroform, and tetrachloroethene (PCE). None of the reported concentrations are above default, numeric RSRs criteria.

**SVOCs (Semi-volatile Organic Compounds):**

- Several SVOCs were reported above the laboratory detection limits in five of the seven soil samples submitted for analysis. Of the reported compounds, the concentration of benzo(a)anthracene (1,200 micrograms per kilogram (µg/kg)) in the 9-11.5 foot interval of location W-SB-006 was greater than the RDEC and GB PMC. Soil boring W-SB-006 was advanced adjacent to a suspect historical UST (AOC-1A).

**ETPH (Extractable Total Petroleum Hydrocarbons):**

- ETPH was reported above laboratory detection limits in three soil samples at concentrations ranging from 120 mg/kg in location W-SB-003 to 1,600 mg/kg in location W-SB-007. Only the concentration of ETPH reported in soil collected from the 7.5-10 foot interval from location W-SB-007 is greater than the RDEC.

**PCBs (Polychlorinated Biphenyls):**

- PCBs were detected above the laboratory detection limits in two of the seven samples ranging in concentration from 670 to 1,300 µg/kg. The concentration of PCBs in soil from location W-SB-007 (1,300 µg/kg) is above the RDEC.

**Pesticides and Herbicides:**

- Pesticides and herbicides were not reported above laboratory detection limits in the six soil samples and duplicate soil sample submitted for analysis.

**Metals:**

- One or more metals including arsenic, barium, cadmium, chromium, lead, mercury, and silver were reported above laboratory detection limits in the seven soil samples collected from the site, with the highest overall concentrations reported in soil collected from the 7.5-10 foot interval of location W-SB-007. The concentration of lead (650 mg/kg) in location W-SB-007 is greater than the RDEC for lead of 500 mg/kg. The concentrations of cadmium (0.1 mg/L) and lead (2.07 mg/L) are greater than their respective GB PMC.

### 4.5.3.2 West Side Subsurface Investigation (Groundwater)

Two monitoring wells were installed at the West Side WWTP for collection of groundwater samples. W-MW-001 was installed at the northern portion of the site adjacent to a former heating oil UST (AOC-1C) and W-MW-002 is located west of the control building near the loading dock and oil storage area (AOC-12). Laboratory analysis was completed on the groundwater samples, and a summary of the results are provided below (excerpts from the West Side Subsurface Investigation Report).

**VOCs:**

- No VOCs were detected above laboratory reporting limits in the two groundwater monitoring wells.

**SVOCs:**

- No SVOCs were detected at W-MW-001. Several SVOCs were reported above the laboratory detection limits in groundwater collected from W-MW-002 and the duplicate pair, with the concentrations of phenanthrene (1.2 µg/L and 1.7 µg/L, respectively), above the default, numeric SWPC of 0.077 µg/L.

**ETPH:**

- ETPH was below laboratory detection limits in the two groundwater samples; however, ETPH was reported at a concentration of 0.35 mg/L in the duplicate sample of W-MW-002. As an Additional Polluting Substances, the RSRs include a SWPC for ETPH of 0.25 mg/L.

**PCBs:**

- PCBs were not detected above laboratory reporting limits from either W-MW-001 or W-MW-002.

**Pesticides and Herbicides:**

- Pesticides and herbicides were not reported above laboratory detection limits in the two groundwater samples from West Side WWTP.

**Total Metals:**

- Total metals including arsenic and barium were reported in groundwater collected from W-MW-001 at concentrations of 0.004 mg/L and 0.057 mg/L, respectively. Total metals including barium and chromium were reported in groundwater collected from W-MW-002 at concentrations of 0.35 mg/L and 0.005 mg/L, respectively. None of the concentrations of total metals reported in groundwater are above default, numeric SWPC.

**Dissolved Metals:**

- Dissolved metals including arsenic, barium, and lead were reported in groundwater collected from W-MW-001 at concentrations of 0.006 mg/L, 0.057 mg/L, and 0.002 mg/L, respectively. Dissolved metals including barium and chromium were reported in groundwater collected from W-MW-002 at concentrations of 0.348 mg/L and 0.004 mg/L, respectively. The concentrations of arsenic reported in groundwater from location W-MW-001 is above the SWPC of 0.004 mg/L.

### 4.5.3.3 East Side Subsurface Investigation (Soils)

Seven borings were advanced at the East Side WWTP, and soils were collected at each boring location. Major findings from the soil sampling effort are included below (excerpts from the East Side Subsurface Investigation Report).

**VOCs:**

- Soil samples collected from locations E-SB-001, E-SB-002, E-SB-003, and E-SB-005 exhibited low concentrations of VOCs including 1,2,4-trichlorobenzene (E-SB-005), carbon disulfide (E-SB-001, E-SB-002, and E-SB-003), chloroform (E-SB-005), and naphthalene (E-SB-003). None of the reported concentrations are above default, numeric RSRs criteria.

**SVOCs:**

- Several SVOCs were reported above the laboratory detection limits in all seven soil samples submitted for analysis. Of the reported compounds, the concentrations of benzo(a)pyrene (1,300 µg/kg), benzo(b)fluoranthene (1,200 µg/kg), and benzo(k)fluoranthene (1,400 µg/kg) in the 10-12.5 foot interval of location E-SB-001 are greater than the GB PMC and/or the RDEC.

**ETPH:**

- ETPH was reported above laboratory detection limits in two samples collected from locations E-SB-002 (270 mg/kg), and E-SB-004 (78 mg/kg), all below the default RDEC, IDEC and GB PMC.

**PCBs:**

- PCBs were reported above the laboratory detection limits in six of the seven soil sample submitted for analysis, with each concentration greater than the RDEC of 500 µg/kg . The concentration of PCBs in the 10-12.5-foot interval of location E-SB-002 (16,000 µg/kg) is also above the IDEC of 10,000 µg/kg.

**Pesticides and Herbicides:**

- The pesticide heptachlor epoxide was reported at a concentration of 73 µg/kg in the 9-11-foot interval of location E-SB-003; this concentration is above both the RDEC and GB PMC. Several pesticide compound reporting limits were greater the GB PMC and/or the RDEC; the elevated reporting limits are attributed to matrix interferences caused by the presence of PCBs in E-SB-002, E-SB-003, E-SB-004, E-SB-005 and E-SB-006.

**Metals:**

- One or more metals including arsenic, barium, cadmium, chromium, lead, mercury, selenium, and silver were reported above laboratory detection limits in the seven soil samples collected from the Site, with the highest overall concentrations reported in soil collected from the 10-12.5 foot interval of location E-SB-002 and the 7-9 foot interval of E-SB-004. The concentration of lead (561 mg/kg) in E-SB-002 is greater than the RDEC for lead. The concentration of arsenic (26.5 mg/kg) in E-SB-004 is greater than the RDEC and IDEC of 10 mg/kg. Relative to an evaluation of metals data to the GB PMC, the soil samples collected from locations E-SB-002, E-SB-003, E-SB-004, and E-SB-006 which exhibited the highest individual metals concentrations, were submitted for TCLP analysis for select metals. With the exception of the concentration of cadmium (0.092 mg/L) in the TCLP extract of soil from location E-SB-002, no other TCLP metals concentrations were reported above the GB PMC.

### 4.5.3.4 East Side Subsurface Investigation (Groundwater)

Two monitoring wells were established at the East Side WWTP for collection of groundwater samples. E-MW-001 was installed at the southwestern end of the site near the grit/scum storage building and fill piles (AOC-8) and grit leachate UST/chamber (AOC-9). E-MW-002 is located in the south-central part of the site, south of the control building, near former heating oil UST (AOC-2B) and exterior material storage/surface staining areas (AOC-12). Laboratory analysis was completed on the groundwater samples, and a summary of the results are provided below (excerpts from the East Side Subsurface Investigation Report).

**VOCs:**

- No VOCs were detected in E-MW-001. VOCs including 1,2,4-trimethylbenzene, methyl tert-butyl ether (MTBE), and naphthalene, common fuel oil constituents, were reported above laboratory detection limits in groundwater collected from E-MW-002 and the duplicate pair. None of the reported concentrations were above SWPC or above the Residential Volatilization Criteria (RVC).

**SVOCs:**

- No SVOCs were detected in E-MW-001. Several SVOCs were reported above the laboratory detection limits in groundwater collected from E-MW-002 and the duplicate pair. With the exception of the concentration of phenanthrene reported in groundwater from E-MW-002 and its duplicate (0.44 µg/L and 0.48 µg/L, respectively), none of the reported SVOC concentrations were above the default, numeric SWPC.

**ETPH:**

- ETPH was not detected in E-MW-001. ETPH was reported above laboratory detection limits in the groundwater sample E-MW-002 and its duplicate at concentrations of 0.31 mg/L and 0.19 mg/L, respectively. As an Additional Polluting Substances, the RSRs include a SWPC for ETPH of 0.25 mg/L.

**PCBs:**

- No PCBs were detected above laboratory reporting limits from E-MW-001. At E-MW-002 and it's duplicate, PCB-1242 was detected at a concentration of 1.7 µg/L and 1.8 µg/L, respectively, which is above the SWPC of 0.5 µg/L.

**Pesticides and Herbicides:**

- Pesticides and herbicides were not reported above laboratory detection limits in the two groundwater samples from the East Side WWTP.

**Total Metals:**

- Total metals including arsenic, barium, and chromium were reported above laboratory detection limits in E-MW-001 and E-MW-002, and lead was reported above laboratory detection limits in E-MW-002. The concentrations of arsenic reported E-MW-001 (0.007 mg/L) and E-MW-002 (0.006 mg/L) are above the default, numeric SWPC (0.004 mg/L). None of the remaining reported concentrations of metals are above SWPC.

**Dissolved Metals:**

- Dissolved metals including barium and chromium were reported in groundwater collected from E-MW-001and E-MW-002, below RSR criteria.  In addition, dissolved arsenic and lead were detected at E-MW-002. The concentration of arsenic from well E-MW-002 is above the, SWPC.

### 4.5.4 Summary of Environmental Investigations

The environmental conditions of the West Side and East Side WWTP sites were evaluated through a Phase I environmental site assessment and subsequent subsurface soil and groundwater investigations as well as a hazardous building material survey. Several areas of concerns were identified and evaluated at each WWTP site. PCBs are prevalent throughout the buildings at both the West Side and East Side WWTPs. Various contaminants including a few isolated metals (arsenic, cadmium, chromium and lead), a few isolated low-level VOCs, pesticides, and PAHs were identified in the soils and groundwater at both sites. At the East Side WWTP, PCBs were found in a majority of the boring locations at levels above RSR criteria.

In addition to the environmental investigations completed as a part of this facility planning process, previous environmental findings must also be considered. In particular, the previous investigations at the West Side site and the adjacent 2.2-acre northern parcel, which is being considered as a location for future facilities, revealed PCBs, metals, PAHs, and VOCs in the soil, with PCB concentrations higher than 50 mg/kg detected in some locations.

These contaminants can be expected to increase the cost of demolition of buildings and disposal of soils during construction. PCBs, asbestos, and lead containing materials within the buildings will increase building demolition costs. Soils with PCB concentrations greater than 2 mg/kg (as detected at East Side WWTP) will require disposal at a commercial disposal facility at increased cost. The elevated PCB concentrations at the West Side site adjacent northern parcel (greater than 50 mg/kg), may require disposal at a Resource Conservation and Recovery Act (RCRA) hazardous waste disposal facility. These requirements further increase excavate disposal costs.

During construction, it is anticipated that it will be necessary to dewater and manage groundwater in several excavation areas. Groundwater from the dewatering process may be managed by containment and off-site disposal to a treatment facility or discharged to an adjacent surface water. In order to discharge groundwater from the dewatering process as a wastewater to the adjacent surface water during site redevelopment under the Connecticut *General Permit for the Discharge of Stormwater and Dewatering Wastewaters from Construction Activities*, the discharge shall not cause nor contribute to an exceedance of water quality standards in the receiving surface water body. Based on the results of the groundwater sampling, it is expected that some treatment of impacted groundwater prior to discharge will likely be necessary and may require discharge under DEEP's *General Permit for the Discharge of Groundwater Remediation Wastewater*.

The expected cost increases associated with building demolition, soil disposal, and groundwater disposal have been considered in the alternative evaluation within this report. It is expected that additional investigation and characterization of these contaminants will be completed (particularly in the West Side WWTP northern parcel) prior to construction to further refine the prevalence, extents, removal and disposal options, and the cost associated with these substances.