Section 6

Development and Screening of Alternatives

6.1 Introduction

This section presents the development and preliminary screening of treatment process alternatives based on our understanding and evaluation of the condition and needs of the existing facilities and the Combined Sewer Overflow (CSO) Long Term Control Plan (LTCP) recommendations. The desire of the Water Pollution Control Authority (WPCA) to efficiently and effectively treat wastewater, reduce CSOs, and increase the resilience of the system to assure the value of investment in the facility is central to the evaluations presented herein. Although this section is broken down by unit process, CDM Smith acknowledges and has taken a holistic view of the treatment facility and understands how decisions made in one unit process can impact the performance and/or sizing of another process. For each unit process a discussion of the universe of alternatives is presented and screened to determine those most feasible options to carry forward for detailed evaluation and costing in Section 7.

Prior to examining each of the treatment technologies, the collection and treatment systems are holistically assessed. This includes a review of the hydraulic capacity of the collection system and the potential benefits of increasing wastewater treatment plants (WWTPs) capacity to accept additional flow at the two plants, as compared to providing off-line storage in the collection system to reduce combined sewer overflows. In addition, an evaluation of the benefits of consolidating the two treatment facilities for liquid treatment, solids treatment or both is assessed.

Graphically the evaluation process is presented in **Figure 6.1-1**. In this Section 6, the initial assessment and screening of alternatives and opportunities is undertaken for all treatment technologies, while concurrently considering opportunities for resource recovery, sustainable development, and community enhancement. Section 7 finalizes wastewater treatment plant concepts and builds alternative scenarios around various treatment capacities. These alternatives are integrated with system recommendations developed as a part of the Long-Term Control Plan, and ultimately results in a Recommended Plan further defined in Section 9.

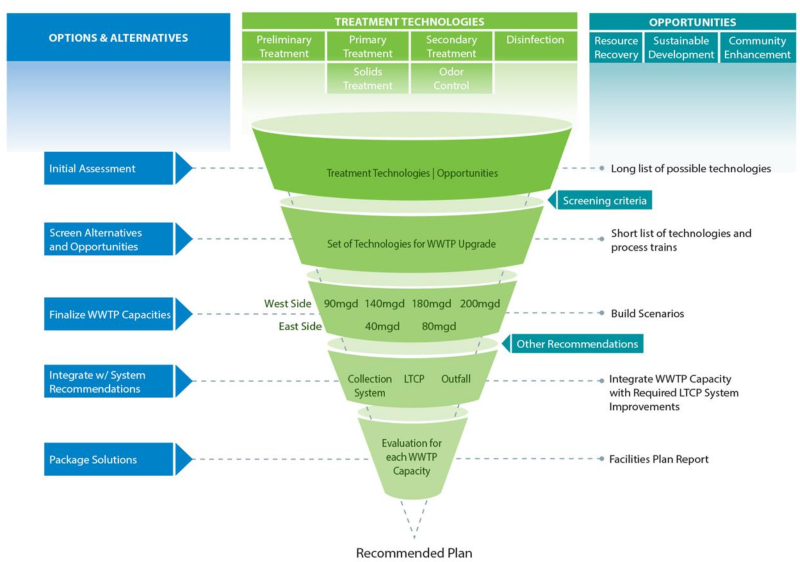


Figure 6.1-1
Evaluation Process

6.2 Collection System Alternatives Development

6.2.1 Introduction

This section explains the development of alternatives for the improvement of the wastewater collection system tributary to the West Side and East Side WWTPs. Improvements and alterations to the collection system were evaluated using the updated hydraulic model of the collection system. Alternatives to improve the conveyance of additional volume to the treatment plants and reduce system wide CSOs were a focus of the collection system alternatives development process. The alternatives under consideration could help the WPCA work towards their goal of cost-effective CSO control of the 1-year storm, 24-hour event described previously in Section 3.

6.2.2 Collection System Model Alternatives Analysis

An alternatives analysis was conducted to evaluate the benefit of expanded wet weather treatment capacity at the WWTPs on reducing surcharge in the collection system. In each alternative, WWTP wet weather capacity was increased and the resulting reduction of CSO volume was assessed. All alternatives (apart from the validation condition) assumed a "best case" maintenance scenario for the collection system through removal of all modeled sediment and reducing Manning's coefficient, *n*, to 0.013 (a measure of roughness). This was done to evaluate the CSO benefit from capacity changes at each WWTP while utilizing the maximum conveyance of the existing pipe network.

Design storm simulations were completed to assess the maximum system conveyance to each WWTP and to select the optimal wet weather capacities to evaluate further within this Facilities Plan. The flow limit to each WWTP was removed from the model and the "best case" maintenance scenario (as described above) was used. Under these conditions, 160 million gallons per day (mgd) was able to be conveyed to the West Side WWTP and 60 mgd was able to be conveyed to the East Side WWTP during the 1-year, 24-hour design storm. Capacity alternatives exceeding these rates must thus be paired with the addition of upstream conveyance improvements (construction of new piping) in order to deliver higher peak flow to each WWTP during the 1-year, 24-hour design storm. Note however, that higher flows could be conveyed under more extreme storm conditions.

Five alternatives were evaluated at the West Side WWTP and three at the East Side WWTP. Wet weather capacities of 90, 140, 160, 180, and 200 mgd were simulated at the West Side WWTP and capacities of 40, 60, and 80 mgd were simulated at the East Side WWTP. The 180 and 200 mgd alternatives at West Side WWTP and the 80 mgd East Side WWTP alternative included collection system pipe replacements to attain adequate conveyance to the WWTPs. A map of piping improvements in each alternative is shown in **Figure 6.2-1**. The alternatives simulated are outlined in **Table 6.2-1** and summarized further in Section 6.2.3 through Section 6.2.8.

DRAFT

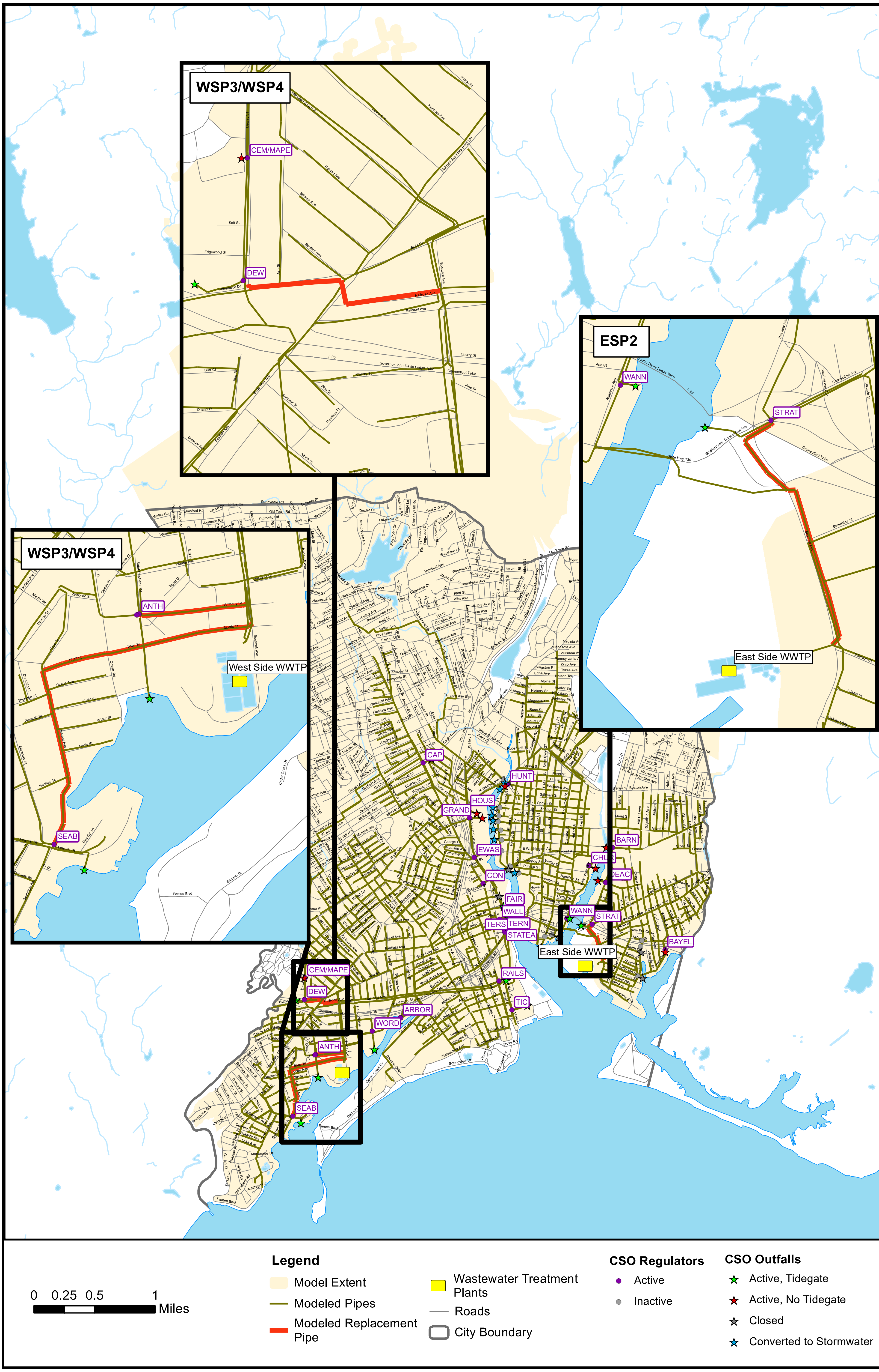


Figure 6.2-1
Pipe Replacement Alternatives

This page intentionally left blank.

Table 6.2-1 Simulated Alternatives

Scenario	West Side WWTP Capacity (mgd)	East Side WWTP Capacity (mgd)	Sediment	Pipe Replacement
Validation Condition[1]	80	35	Existing	None
Baseline[2]	90	40	None	None
WSP1[3]	140	40	None	None
WSP2	160	40	None	None
WSP3	180	40	None	▪ Upsize 4,300 ft of 24" to 42" from SEAB to interceptor ▪ Fix shallow slope in Ellsworth Park ▪ Upsize 1,400 ft of 12/15/18" downstream of ANTH to interceptor to 42" ▪ New 1,600 ft of 48-inch from DEW to interceptor
WSP4	200	40	None	Same as WSP3 pipe replacement
ESP1[4]	90	60	None	None
ESP2	90	80	None	▪ 750 ft of 30" to 48" STRAT to confluence with WANN ▪ Plug recombined WANN stormwater connection ▪ 1,700 ft of 48/54" to 60" from STRAT/WANN confluence to East Side WWTP

Notes:
1. Validation conditions reflect the flow limits in the updated model, which were based on observed maximum daily flow at each WWTP from 2017 to 2019. This scenario has lower capacities at each WWTP than their design capacities.
2. Baseline reflects the wet weather design capacity of each WWTP.
3. WSP – West Side Plant
4. ESP – East Side Plant

6.2.3 West and East Side WWTPs – Validation Condition (i.e. No Action)

The hydraulic model used to evaluate the WPCA's collection system capacity was updated to reflect the existing conditions of the collection system today. Record plans, including recent separation and lining work were incorporated into the model. The accumulated sediment in the pipes described in Section 3 was also included as part of the model update. A review of the existing WWTP influent flow data and discussion with plant operators revealed that both plants typically throttle the influent gates under high flow conditions to avoid either flooding the influent pump station (based on available pumping capacity) or exceeding treatment plant design capacity. As such, the model imposed a maximum plant capacity of 80 mgd at the West Side WWTP and 35 mgd at the East Side WWTP to reflect existing conditions.

6.2.4 West and East Side WWTPs - Baseline Condition (Cleaning, No Collection System Improvements)

Following model calibration based on the existing conditions, the model was used to evaluate the conveyance capacity of the collection system if all sediment was removed, and if the flow restrictions presented in the validation condition were removed to allow the wet weather design capacity of the WWTPs (90 mgd West Side plant, 40 mgd East Side plant) to be reached.

6.2.5 Alternatives WSP1, WSP2 and ESP1 (Increase WWTP Capacity, Cleaning, No Collection System Improvements)

The next few alternatives evaluated for each WWTP involved removing modeled sediment and removing all flow restrictions (resulting in a free discharge at the WWTPs). This results in an increase in overall collection system capacity capable of delivering a maximum of 160 mgd at the West Side WWTP, and 60 mgd at the East Side WWTP without any other physical collection system improvements. WSP1 evaluated increasing the capacity of the West Side WWTP to 140 mgd, WSP2 evaluated the West Side WWTP to 160 mgd, and ESP1 evaluated the East Side WWTP to 60 mgd.

The largest conveyance benefit comes from removing the flow restrictions at the two WWTPs. Removal of the modeled sediment increases the collection system conveyance during the 1-year, 24-hour storm modestly by 4 mgd and 9 mgd at the West Side plant and East Side plant, respectively (assuming free discharge).

In practice, these alternatives would involve upgrading and expanding the WWTPs beyond their current 80 and 35 mgd flow limits, and beyond their 90 and 40 mgd design wet weather capacities. The collection system in Bridgeport has the capability to deliver significantly more flow to the WWTPs by simply increasing the influent pumping and treatment capacity of the WWTPs. Additionally, completing the recommended pipe, siphon, and storage conduit cleaning outlined as baseline collection system recommendations in Section 3 would help increase collection system conveyance.

6.2.6 Alternatives WSP3 and WSP4 (West Side WWTP to 180 & 200 MGD)

Physical piping improvements that could be implemented in the collection system were evaluated in the model to increase the amount of flow delivered to the West Side WWTP. Hydraulic restrictions were identified and corrected in the model to determine if any cost-effective CSO solutions existed within the collection system. A peak influent flow of 160 mgd could be delivered with an increase in WWTP capacity and sediment removal alone (as described in Section 6.2.5 above), but a future capacity of 180 mgd was identified at the West Side WWTP as a break point at which multiple regulators would be controlled in a 1-year, 24-hour storm (see CSO results in Section 6.2.8).

The collection system improvements required in Alternatives WSP3 and WSP4 are the same, but these two alternatives vary based on WWTP peak capacity. After increasing the West Side WWTP capacity to either 180 mgd or 200 mgd, completing the cleaning outlined in Section 3, and constructing the physical alterations described below CSOs ANTH, CEM/MAPE, DEW, and SEAB regulators can be controlled to the 1-year level.

The WSP3 and WSP4 conveyance improvements include the following:

- An increase in treatment capacity to 180 mgd (WSP3) or 200 mgd (WSP4) at the West Side WWTP.
- All cleaning activities outlined in Section 3.
- Increasing 4,300 feet of 24-inch sewer to 42-inch sewer from the SEAB regulator to the main interceptor on Bostwick Avenue. This item also includes the adjustment of a shallow pipe slope in Ellsworth Park.
- Increasing 1,400 feet of 12 to 18-inch sewer to 42-inch sewer downstream of the ANTH regulator to the interceptor on Bostwick Avenue.
- New 1,600 ft of 48-inch sewer from DEW to interceptor

Maps of the WSP3/WSP4 improvements in the area of the West Side WWTP are included as **Figure 6.2-2 and Figure 6.2-3**. Pipeline improvements under these alternatives are highlighted in blue. These pipeline improvements represent a significant infrastructure improvement for the West Side tributary area and involve installing over a mile of large diameter pipe. These improvements would involve laying new pipeline through Ellsworth Park, as well as along Commerce Drive, Harbor Avenue, Railroad Avenue, Shell Street, St. Stephens Road, and Bostwick Avenue. Although two parallel pipelines are shown from St. Stephens Road to Bostwick Avenue in Figure 6.2-2 below, these two pipes could likely be consolidated into a single pipeline along the southern route to lower the overall cost-estimate for these improvements. Micro-tunneling is another potential alternative that could be considered for conveyance in the area of the SEAB regulator. Wet weather flow could potentially be conveyed from the area of Ellsworth Park across Brewster Cove to the West Side WWTP via a micro tunnel rather than adding new large diameter piping in the street along Harbor Avenue and Shell Street.

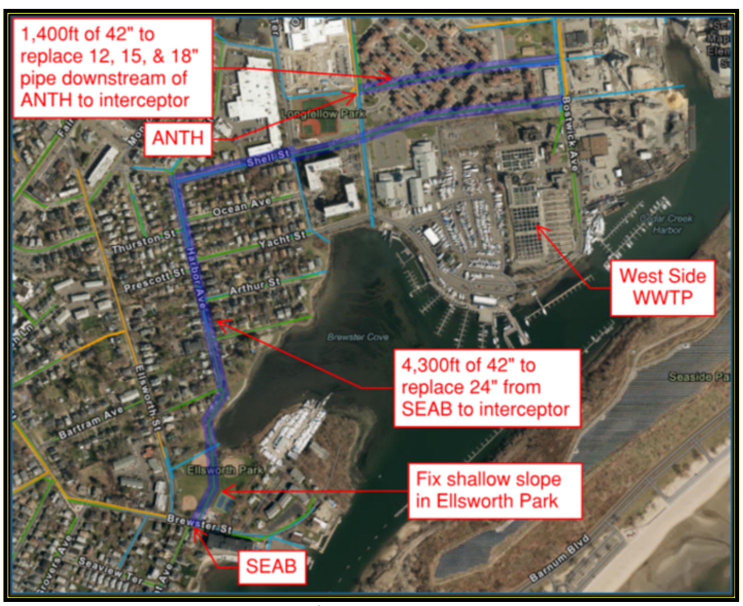


Figure 6.2-2
WSP3 and WSP4 Collection System Improvements (ANTH & SEAB)

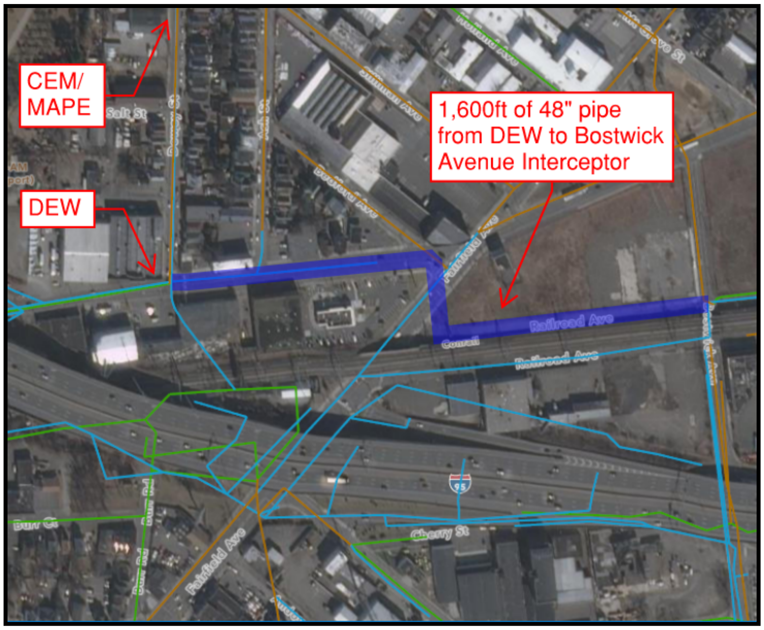


Figure 6.2-3
WSP3 and WSP4 Collection System Improvements (DEW & CEM/MAPE)

The collection system improvements shown in Figure 6.2-3 represents the current leading alternative in Ash Creek CSO study. This evaluation is being completed concurrently with this Facilities Plan. This is discussed further in **Section 6.2.12**.

While these collection system improvements would be significant, control of the ANTH, CEM/MAPE, DEW, and SEAB regulators in the 1-year, 24-hour storm is worth considering for the significant environmental benefit. These improvements would also eliminate the need for the Dewey Street and Ellsworth Park CSO storage tanks identified in the 2011 CSO LTCP.

6.2.7 Alternative ESP2 (East Side WWTP to 80 MGD)

In addition to the West Side WWTP, the capacity of the East Side collection system was also evaluated. It was determined that the East Side collection system could deliver a maximum of 60 mgd during a 1-year, 24-hour storm event, if the WWTP capacity is increased to 60 mgd (Alternative ESP1), and cleaning is completed, but no collection system improvements are constructed.

As with the West Side, the East Side tributary area was evaluated in the model in order to identify any hydraulic restrictions that may exist. Several areas were targeted for improvements that could increase the East Side collection system's delivery to 80 mgd. The East Side WWTP collection system capacity can deliver 80 mgd, if the improvements identified below are completed.

The improvements in Alternative ESP2 include the following:

- An increase of treatment capacity to 80 mgd at the East Side WWTP.
- All cleaning activities outlined in Section 3.

- Increasing 750 feet of 30-inch pipe to 36-inch pipe along Connecticut and Seaview Avenue from the STRAT regulator to the convergence with the WANN regulator dry weather flow.
- Increasing 1,700 feet of 48 and 54-inch pipe to 60-inch pipe along Seaview Avenue from the STRAT & WANN flow convergence to the East Side WWTP.
- Plugging the recombined WANN stormwater connection on Waterview Avenue.

The ESP2 collection system improvements are shown on **Figure 6.2-4.** The locations of piping improvements are highlighted in this figure in blue. These projects include significant pipe replacement along Connecticut Avenue (a state road), as well as replacing an interceptor to the East Side WWTP along Seaview Avenue. A recombined storm water connection upstream of the WANN regulator would also need to be plugged.

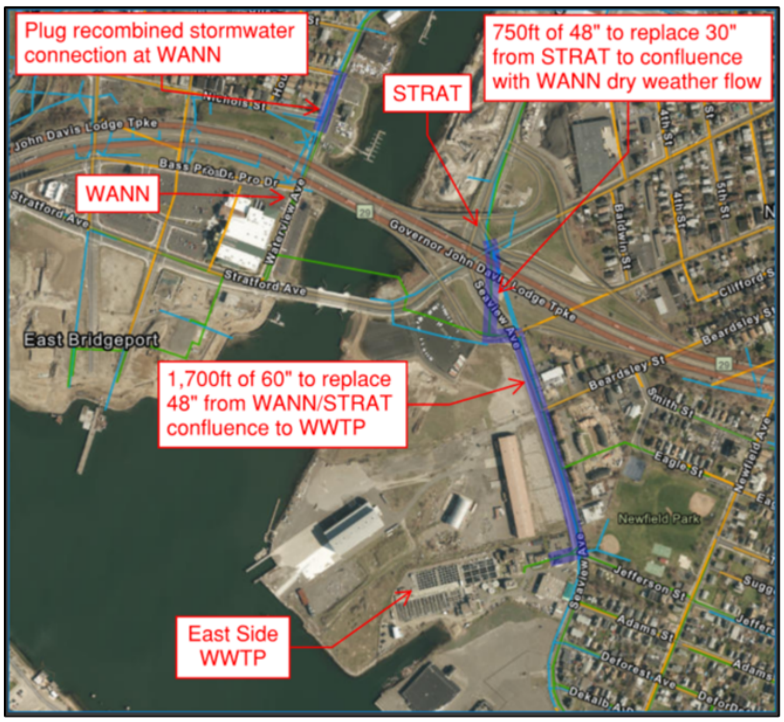


Figure 6.2-4
ESP2 Collection System Improvements

6.2.8 Model Results of Alternatives Analysis

Simulated CSO volumes decrease as flow to each WWTP is increased. West Side WWTP results are shown in **Figure 6.2-5.** This figure plots the West Side CSO versus WWTP capacity. CSO volume decreases as WWTP capacity is increased from 80 mgd to 200 mgd. Several key observations can be identified:

- Restoring design capacity of the West Side WWTP from 80 to 90 mgd results in a simulated reduction of 3.9 million gallons (MG) CSO.
- Increasing design capacity of the West Side WWTP from 90 to 140 mgd results in a simulated reduction of 9.4 MG CSO.

- CSO reduction plateaus between 140 and 160 mgd. Despite the 20 mgd increase in WWTP capacity, CSO only drops by 0.9 MG.
- Increasing design capacity of the West Side WWTP from 140 to 180 mgd results in a simulated reduction of 8.0 MG CSO.
- CSO volume reduction plateaus between 180 and 200 mgd. Despite the 20 mgd increase in WWTP capacity, CSO only drops by 1.5 MG.
- CSO WORD attains 1-year level of control in all modeled scenarios.
- CSOs RAILS and TIC achieve 1-year level of control when West Side WWTP wet weather capacity is 140 mgd and 160 mgd.
- CSOs CEM/MAPE, DEW, ANTH, and SEAB achieve 1-year level of control when West Side WWTP wet weather capacity is 180 mgd and higher.

The CSO reduction benefits observed under alternative WSP3 (180 mgd) and WSP4 (200 mgd) are due in part to the collection system improvements described previously**.**

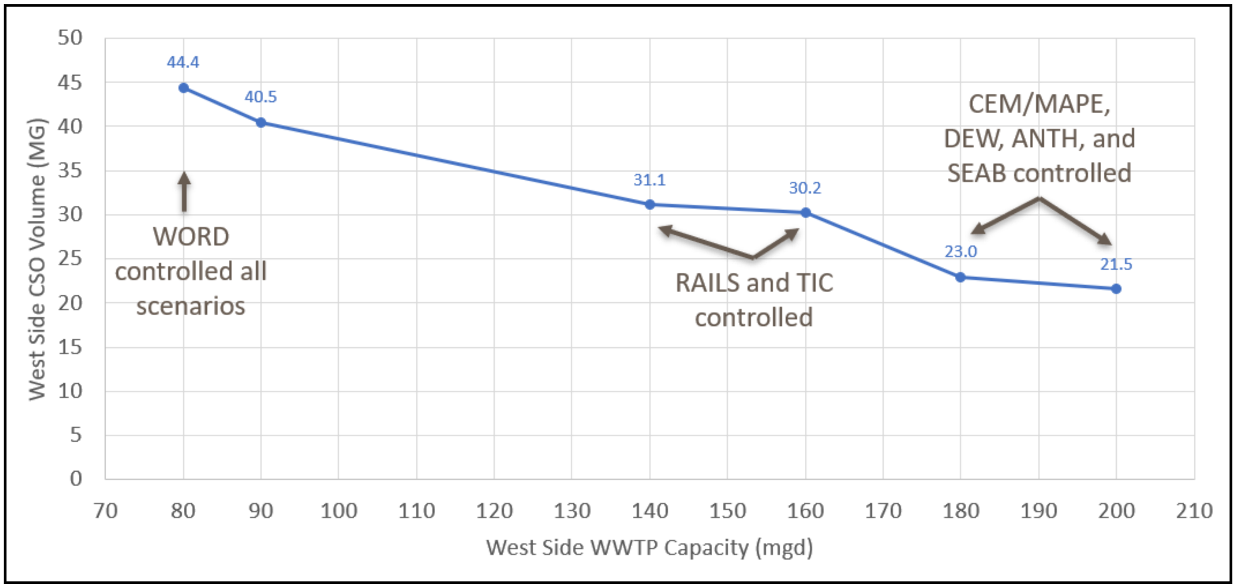


Figure 6.2-5
Simulated West Side WWTP Capacity vs CSO Volume during the 1-year, 24-hour Design Storm

Simulated results for the East Side WWTP are shown in **Figure 6.2-6**. This figure plots East Side CSO versus WWTP capacity. CSO volume steadily decreases as WWTP capacity increases from 35 mgd to 80 mgd. Three East Side CSOs attain 1-year level of control when capacity is increased to 80 mgd, including DEAC, WANN, and STRAT. Benefits observed under alternative ESP2 (80 mgd) are due in part to the collection system improvements described previously**.**

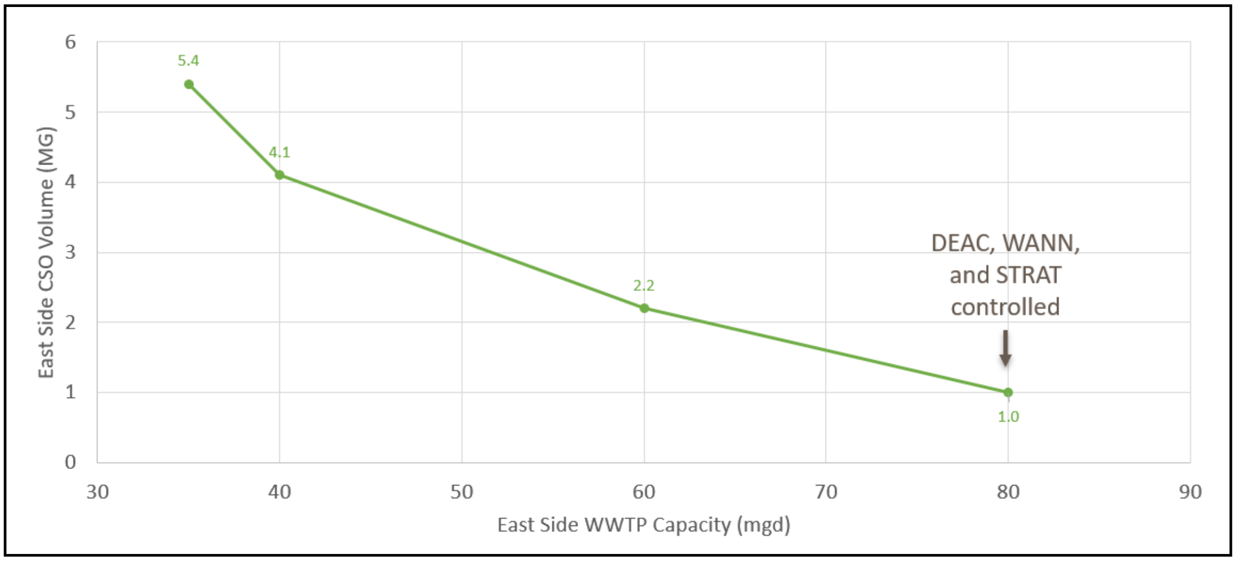


Figure 6.2-6
Simulated East Side WWTP Capacity vs CSO Volume during the 1-year, 24-hour Design Storm

6.2.9 Screening of Collection System Alternatives

The collection system alternatives presented previously were evaluated in the hydraulic model. This subsection serves to qualitatively screen the collection system alternatives to only those that will be considered in Section 7.

6.2.9.1 West and East Side WWTPs – Validation Condition (i.e. No Action)

This alternative represents a "no action" condition. No additional cleaning or maintenance would be completed. The accumulated sediment described in Section 3 would remain in the collection system. None of the baseline collection system recommendations (Cleaning; Capacity, Management, Operations, and Maintenance (CMOM); Tide Gate Repairs; etc.) would be completed. The WWTP practice of throttling the influent gates under high flow conditions to avoid either flooding the influent pump station (based on available pumping capacity) or exceeding treatment plant design capacity would continue. This limits the West Side WWTP to 80 mgd and East Side WWTP to 35 mgd.

This alternative would not require any additional operation and maintenance (O&M) or capital cost investment in the existing collection system. This alternative is not recommended because it does not take advantage of the collection system's capacity nor does it effectively advance the WPCA's goals for CSO reduction and WWTP upgrades. This alternative will not be considered further.

6.2.9.2 West and East Side WWTPs - Baseline Condition (Cleaning, No Collection System Improvements)

This alternative involves removal of sediment from the collection system. Completion of the baseline collection system recommendations (including routine, targeting, siphon cleaning, and storage conduit cleaning) is included in this alternative to restore the collection system to a "clean" condition. Treatment plant capacity would be restored to the existing peak flow capacities of 90 mgd at West Side WWTP and 40 mgd at East Side WWTP. This would be accomplished through reliability upgrades at the WWTPs to ensure that these peak flows could be accepted for reliable treatment, without the current influent gate throttling activities.

The baseline collection system recommendations represent a worthwhile benefit for a relatively moderate investment. Cleaning of storage conduits, siphons, and known problem areas would increase conveyance in the system and reduce CSO events. Repairing the tide gates will reduce tidal inflow, reducing the treatment of any extraneous tidal flow.

Completing the baseline collection system recommendations and restoring the treatment plants to their 90 mgd and 40 mgd peak capacities produces some environmental benefit within the collection system. These benefits are described below.

- <u>West Side Baseline (90 mgd):</u> Restoring the West Side WWTP to its design peak capacity (90 mgd) results in a CSO reduction of 3.9 MG (8.8%) in the West Side collection system during the 1-year, 24-hour storm. The WORD CSO regulator is also controlled to the 1-year level with this alternative.

- <u>East Side Baseline (40 mgd):</u> Restoring the East Side WWTP to its design peak capacity (40 mgd) results in a CSO reduction of 1.3 MG (24%) in the East Side collection system during the 1-year, 24-hour storm. No additional CSO regulators are controlled to the 1-year level as compared to existing conditions.

These two baseline alternatives address the WPCA's goals for both CSO reduction and WWTP improvements. These alternatives will be considered further within this facilities plan. Various alternatives for 90 mgd West Side WWTP and 40 mgd East Side WWTP upgrades are presented in **Section 7**.

6.2.9.3 Alternative WSP1, WSP2, and ESP1 (Increase WWTP Capacity, Cleaning, No Collection System Improvements)

Alternatives WSP1, WSP2, and ESP1 involve the completion of the baseline collection system recommendations in conjunction with WWTP upgrades beyond the design peak capacities. No additional alterations to the collection system would be needed in these three alternatives.

The West Side WWTP was evaluated in the model with a 140 mgd peak capacity (WSP1) and a 160 mgd peak capacity (WSP2). As described previously, 160 mgd is the maximum conveyance that can be obtained from the West Side collection system during the 1-year, 24-hour design storm event without requiring additional alterations to the piping network. Alternatives WSP1 and WSP2 provide the environmental benefits described below.

- <u>WSP1 (West Side, 140 mgd):</u> 13.3 MG (30%) of existing West Side CSO in the 1-year, 24-hour storm can be reduced by conveying 140 mgd to the treatment plant. CSO regulators WORD, RAILS, and TIC are controlled to the 1-year level.

- <u>WSP2 (West Side, 160 mgd):</u> 14.2 MG (32%) of existing West Side CSO in the 1-year, 24-hour storm can be reduced by increasing the capacity of West Side WWTP to 160 mgd. CSO regulators WORD, RAILS, and TIC are controlled to the 1-year level.

The East Side WWTP was evaluated in the model with a 60 mgd peak flow capacity (ESP1). As described previously, 60 mgd is the maximum conveyance of the East Side collection system during the 1-year, 24-hour design storm event, without additional physical piping improvements. Alternative ESP1 provides the following environmental benefits.

- ESP1 (East Side, 60 mgd): 3.2 MG (59%) of existing East Side CSO in the 1-year, 24-hour storm can be reduced by increasing the wet weather capacity of the East Side WWTP to 60 mgd. Although a large reduction in CSO volume is obtained, no additional CSO regulators are controlled.

Although all three of these alternatives address the WPCA's goals for both CSO reduction and WWTP improvements, the CSO benefits (in terms of both volume reduction and regulators controlled) are not as substantial as increasing the treatment plants' **capacities** to even higher peak flow rates (see Alternatives WSP3, WSP4, and ESP2).

WSP2 provides little CSO benefit over WSP1 and was not considered further. WSP1 was considered as a mid-point conveyance alternative between a baseline of 90 mgd and the higher conveyance alternatives (180 and 200 mgd). Therefore, WSP1 was carried forward to Section 7 for further consideration.

Alternative ESP2 at 80 mgd provides significantly more CSO benefit than Alternative ESP1 at 60 mgd, for a relatively small WWTP capacity increase. Therefore, ESP1 was not considered further in lieu of the greater benefits provided by ESP2.

6.2.9.4 Alternative WSP3 and WSP4 (West Side WWTP to 180 MGD and 200 MGD)

Alternatives WSP3 and WSP4 involve increasing the wet weather treatment capacity at the West Side WWTP to a peak of 180 mgd and 200 mgd respectively. As discussed previously, if the collection system cleaning is completed, and the West Side WWTP is treated as a free discharge in the hydraulic model, a maximum of 160 mgd can be conveyed through the existing pipes to the WWTP during the 1-year, 24-hour design storm event. In order to reach flow rates higher than 160 mgd during this storm event, physical alterations in the collection system must be constructed.

A summary of the conveyance improvements required in Alternatives WSP3 and WSP4 were described previously in **Section 6.2.6**. These two alternatives require the same collection system improvements but vary in West Side WWTP peak capacity. WSP3 and WSP4 provide the following environmental benefits:

- WSP3 (West Side, 180 mgd): 21.4 MG (48%) of existing West Side CSO in the 1-year, 24-hour storm can be reduced by conveying 180 mgd to the treatment plant. Seven CSO regulators (ANTH, CEM/MAPE, DEW, RAILS, SEAB, TIC, and WORD) are controlled to the 1-year level.

- WSP4 (West Side, 200 mgd): 22.9 MG (52%) of existing West Side CSO in the 1-year, 24-hour storm can be reduced by conveying 200 mgd to the treatment plant. Five CSO regulators (ANTH, CEM/MAPE, DEW, RAILS, SEAB, TIC, and WORD) are controlled to the 1-year level.

Alternatives WSP3 and WSP4 provide substantial CSO benefit during the 1-year, 24-hour design storm. This alternative meets the WPCA's goals of both upgrading the West Side WWTP and creates a considerable CSO benefit in the collection system. Both alternatives also control seven of the 19 CSO outfalls in the West Side WWTP tributary area to a 1-year level. Due to the benefits provided, alternatives WSP3 and WSP4 will be further evaluated within this facilities plan. Various West Side WWTP upgrade alternatives for 180 mgd and 200 mgd wet weather peak flow are presented in **Section 7**.

6.2.9.5 Alternative ESP2 (East Side WWTP to 80 MGD)

Alternative ESP2 relies on increasing the East Side WWTP to an 80 mgd peak wet weather capacity. This is above the available conveyance capacity of the East Side collection system. As described previously, if the East Side WWTP is evaluated as a free discharge in the hydraulic model, the maximum conveyance of the collection system is 60 mgd during the 1-year, 24-hour design storm event. In order to take the East Side WWTP to an 80 mgd peak flow, additional collection system improvements are required.

Section 6.2.7 describes the collection system alterations required in alternative ESP2. Completion of these ESP2 conveyance improvements and upgrade of the East Side WWTP to 80 mgd peak capacity provides the following environmental benefits:

- <u>ESP2 (East Side, 80 mgd)</u>: 4.4 MG (81%) of existing East Side CSO in the 1-year, 24-hour storm can be reduced by conveying 80 mgd to the treatment plant. Three additional CSO regulators (DEAC, WANN, and STRAT) are controlled to the 1-year level.

CSO volume tributary to the East Side WWTP is drastically reduced by Alternative ESP2. In addition to the volume reduction, ESP2 controls half of the CSO outfalls (3 of 6) in the East Side collection system. Due to the substantial environmental benefits provided, an 80 mgd East Side WWTP wet weather capacity was considered further within this facility plan. Various alternatives for 80 mgd peak flow upgrades at the East Side WWTP are presented in **Section 7** of this Facilities Plan.

6.2.9.6 Collection System Alternative Screening Summary

The collection system alternative analysis and evaluation examined different levels of conveyance and treatment at the East Side and West Side WWTPs to balance the WPCA's goals for treatment plant improvements, as well as cost effective mitigation for CSO in the collection system. Ultimately, four different levels of wet weather treatment were considered for the West Side WWTP, and two different wet weather treatment capacities were considered for the East Side WWTP. The treatment capacities carried forward for further evaluation in this facilities plan are listed below:

- <u>West Side WWTP:</u> 90 mgd, 140 mgd, 180 mgd, and 200 mgd
- <u>East Side WWTP:</u> 40 mgd and 80 mgd

The baseline WWTP upgrades of 90 mgd at West Side WWTP and 40 mgd at East Side WWTP represent a return to the design peak capacity of the existing plants. These alternatives would be focused primarily on restoring the reliability of the treatment plants, rather than focusing on CSO reduction in the collection system. The WWTP capacities of 140, 180, and 200 mgd at the West Side WWTP, as well as 80 mgd at East Side WWTP were carried forward in the evaluation process to take advantage of the existing conveyance capacity of the collection system and strive for the maximum environmental benefit that could be provided through these treatment plant upgrades.

Various alternative treatment technologies for the WWTP upgrades at the capacities carried forward from this section are evaluated in **Section 7**.

6.2.10 Benefits of Increasing WWTP Wet Weather Capacity

Increasing the wet weather treatment capacity at both the West Side and East Side WWTPs can significantly reduce the collection system CSO overflow as depicted previously in Figures 6.2.5 and 6.2.6. Bridgeport's collection system has available conveyance capacity that is not utilized due to the existing restrictions at the WWTPs.

The WPCA's 2011 CSO Long Term Control Plan (LTCP) recommended $385 million (2010 dollars) in collection system improvements to control CSO in the 1-year, 24-hour design storm. The LTCP recommended that CSO storage tanks, relief tunnels, and sewer separation be constructed in the collection system to control CSOs. Escalated to today's dollars, the cost of the LTCP program is $496 million (2020 dollars). Implementation of the highest conveyance alternatives at each plant (WSP4 and ESP2) conveys over half of the total system wide CSO volume to the WWTPs during the 1-year, 24-hour storm. Based on the 2011 LTCP program, a similar volume of CSO removed from the collection system would likely cost over $250 million (2020 dollars)

Utilizing the existing collection system conveyance and increasing the wet weather capacity at the WWTPs appears to be extremely cost effective for CSO control in Bridgeport. The costs of the necessary collection system alterations as well as the costs for different WWTP upgrade alternatives, and impact of sewer rates are explored further in **Section 7** and **Section 8** of this Facilities Plan.

6.2.11 Higher WWTP Capacity Without Conveyance Improvements

Additional model simulations were completed to quantify the impact of upgrading WWTP capacity without completing the pipe replacement in WSP3, WSP4, and ESP2. This was completed to evaluate how the collection system would perform if the West Side and East Side WWTPs were upgraded, but improvements to collection system conveyance were constructed at a later date.

In addition to the 1-year, 24-hour design storm, the 2-year and 5-year design storms described in the LTCP and a 10-year, 24-hour synthetic storm (SCS Type 3) were simulated with a maximum capacity of 200 mgd at the West Side WWTP, a maximum capacity of 80 mgd at the East Side WWTP, and clean pipes throughout the collection system. No other conveyance improvements or pipe replacement were included. The resulting peak flow received by both WWTPs and CSO volume are listed in **Table 6.2-2.**

- Key observations during the 1-year, 24-hour storm simulation include: Peak flow delivered to the West Side WWTP is 163 mgd, which is 37 mgd less than the modeled maximum capacity.

- West Side CSO volume is 30.1 MG, which is 14.3 MG (32 percent) less than the baseline CSO volume listed in Section 3 but 8.6 MG higher than alternative WSP4 (200 mgd) which includes pipe replacement.
- Peak flow delivered to the East Side WWTP is simulated to be 69 mgd, which is 11 mgd less than the modeled maximum capacity.
- East Side CSO volume is 1.9 MG, which is 3.4 MG (64 percent) less than the baseline CSO volume presented in Section 3 but 0.9 MG higher than alternative EPS2 (80 mgd) which includes pipe replacement.

While neither the East Side nor West Side WWTPs received the peak modeled design flows without pipe replacement during the 1-year, 24-hour design storm, each WWTP may receive flows of that magnitude in larger storm events. Simulated peak flow received at the West Side WWTP during the 2-year, 5-year, and 10-year events is 182, 167, and 200 mgd, respectively. Simulated peak flow received at the East Side WWTP during the 2-year, 5-year, and 10-year events is 78, 68, and 80 mgd, respectively. The 5-year design storm peak flows are low because this event occurs in January 1979 and the model simulates most of the event's precipitation as snow. These results suggest that each WWTP may receive flow as high as the maximum modeled capacity of 200 mgd and 80 mgd at the West Side and East Side WWTPs, respectively, even without pipe replacement during large storm events.

Table 6.2-2 Simulation Results – WWTP Upgrade without Pipe Replacement

Design Storm	Peak Flow to West Side WWTP[1] (mgd)	Peak Flow to East Side WWTP[2] (mgd)	1-year West Side CSO Volume (MG)	1-year East Side CSO Volume (MG)
1-year	163	69	30.1	1.9
2-year[3]	182	78	--	--
5-year[4]	167	68	--	--
10-year[5]	200	80	--	--

Notes:
1. Maximum capacity simulated is 200 mgd with clean pipes in the collection system.
2. Maximum capacity simulated is 80 mgd with clean pipes in the collection system.
3. Historic event observed at Sikorsky Airport on September 3, 1992. Listed in the LTCP (Arcadis, 2017).
4. Historic event observed at Sikorsky Airport on January 21, 1979. Listed in the LTCP (Arcadis, 2017). Simulated as a snow event due to cold temperatures, resulting in lower peak flow than storms with a lower return frequency.
5. Synthetic 24-hour event using a Soil Conservation Survey (SCS) Type 3 curve and 5.35 inches of rainfall (NOAA, 2020c).

6.2.12 Ash Creek (CEM/MAPE & DEW) and Ellsworth Park (SEAB)

The WPCA's CSO consent order currently has requirements for the completion of CSO controls in two areas of Bridgeport. The Ash Creek area is under order for CSO controls in that area to be constructed by April 2023, and the Ellsworth Park area is under order to have CSO controls constructed by March 2025. CSO regulator CEM/MAPE and DEW discharge to the Ash Creek, and the SEAB regulator discharges to Black Rock Harbor.

A storage tank was originally planned for the Ash Creek area, however, a study of other possible CSO controls is currently in progress, running in parallel with this Facilities Plan. Conveyance improvements from the Ash Creek to the West Side WWTP currently appear to be the most promising method of CSO control. These conveyance improvements in the Ash Creek area were included in Alternatives WSP3 and WSP4 as 1,600 feet of new 48-inch pipe from the DEW regulator to the 72-inch interceptor on Bostwick Avenue.

As with Ash Creek, a storage tank was recommended for the Ellsworth Park area in the WPCA's 2011 LTCP. Through this collection system alternatives evaluation, it was discovered that the SEAB regulator could be controlled to the 1-year level through conveyance alone, without a CSO storage tank. This conveyance alternative for SEAB was included in Alternatives WSP3 and WSP4. It is recommended that a more detailed study with metering and an evaluation of alternatives be completed in the SEAB area, similar to the one currently being completed for the Ash Creek area.

The environmental benefit provided by conveyance of this additional flow to the West Side WWTP and the possible elimination of CSO storage tanks in the collection system strengthens the case to increase the wet weather capacity of the West Side WWTP.

6.2.13 Other Recommended Collection System Improvements

In addition to the conveyance improvements outlined in the alternatives above, several other improvements to the collection system were identified. These additional improvements are described in this section.

6.2.13.1 Baseline Collection System Improvements

Baseline collection system recommendations were identified in Section 3 of this Facilities Plan. These recommendations included routine and targeted cleaning, siphon and storage conduit cleaning, CMOM activities, and repair of tide gates. It is recommended that these improvements be completed under any collection system alternative, as these activities represent "low hanging fruit" that will improve conveyance capacity and reduce CSO.

6.2.13.2 Pumping Station Improvements

As summarized in Section 3, the existing pumping stations are in overall good condition and do not require major upgrades. Regardless of the treatment plant capacity and collection system alternative selected, pumping stations will not need upgrades to convey additional flow. Therefore, only two minor improvements should be considered for the pumping stations which apply to all alternatives.

First, the gas detection systems at all stations should be restored to functional condition or replaced entirely. This is required under National Fire Protection Association (NFPA) 70 and ANSI/ISA standard for operator safety when preforming repair and maintenance activities. As, an alternative, additional signing could be installed to remind operation and maintenance staff that portable gas meters are required for entering the pumping stations.

The second improvement is in communication with the WWTPs. Currently, all stations communicate alarms through hard wired telephone auto dialers. This system could be maintained with WWTP upgrades and could be integrated with the supervisory control and data acquisition (SCADA) system. However, the programmable logic controllers (PLCs) at the dry well/wet well pump stations are obsolete and should be replaced. Additionally, the WWTPs will be upgraded to internet-based alarm dialing software and the auto dialers at all stations should be replaced to match this upgrade via cellular telemetry. Replacing PLCs and auto dialers would be a considerable effort requiring programming, testing, and bypass of the station during the transition. Therefore, all alternatives assumed integration of existing pumping station signals into the upgraded WWTPs. The need for additional communication and operation will be further evaluated during design.

6.3 Plant Consolidation

6.3.1 Introduction

This section presents alternatives considered to consolidate the East Side and West Side WWTPs into a single wastewater treatment plant, or to consolidate only residuals management at one of the sites. Consolidation of the two plants has the potential to offer economy of scale in the construction, operation and maintenance of one large facility versus two mid-sized facilities. This must be balanced against the capital and operation and maintenance cost to convey flow from one plant to another. In addition, the availability of land to construct the larger treatment processes must be assessed.

As presented in **Table 5-46**, the various flow conditions to be assessed in this section are summarized below in **Table 6.3-1**. For the peak flows, this analysis looked at the current flows of 90 mgd and 40 mgd respectively for West Side and East Side WWTP's, along with the highest potential future peak flows under consideration, 200 mgd and 80 mgd respectively, as they present the largest flows to accommodate. The estimated sludge production is based on industry standards and would be fine-tuned based on the ultimate unit processes selected, however, are appropriate for use in this assessment.

Table 6.3-1 – WWTP Design Flows

	Design Average Daily Flow (mgd)	Maximum Day Flow Thru Secondary Treatment (mgd)	Current Peak Hour Flow (mgd)	Ultimate Peak Hour Flow (mgd)	Design Average Sludge Production (lb/day)	Design Maximum Month Sludge Production (lb/day)
West Side WWTP	30	58	90	90 to 200	55,400	98,000
East Side WWTP	10	24	40	40 to 80	11,700	18,700
Consolidated WWTP	40	82	130	130 to 280	67,000	116,800

6.3.2 Alternatives Evaluation

The following alternatives for plant consolidation are assessed herein. A short description of the alternatives, major construction requirements, design considerations, and a conclusion/recommendation follows:

- Option No. 1 – Convey all flow from the East Side WWTP (up to 80 mgd) to the West Side WWTP for liquid and solids treatment
- Option No. 2 – Convey 24 mgd (secondary treatment capacity) from the East Side WWTP to the West Side WWTP for liquid and solids treatment
- Option No. 3 – Pump raw sludge generated at the East Side WWTP to the West Side WWTP for solids treatment
- Option No. 4 – Pump partially thickened sludge from the East Side WWTP to the West Side WWTP for further solids treatment
- Option No. 5 – Convey all flow from the West Side WWTP (up to 200 mgd) to the East Side WWTP for liquid and solids treatment
- Option No. 6 – Convey 58 mgd (secondary treatment capacity) from the West Side WWTP to the East Side WWTP for liquid and solids treatment
- Option No. 7 – Pump raw sludge generated at the West Side WWTP to the East Side WWTP for solids treatment
- Option No. 8 – Pump partially thickened sludge from the West Side WWTP to the East Side WWTP for further solids treatment

6.3.2.1 Option No. 1 – Convey All Flow (up to 80 MGD) from East Side WWTP to West Side WWTP

This alternative includes the construction of a conveyance tunnel to convey 40 to 80 mgd from the existing East Side WWTP site to the West Side WWTP for treatment. The tunnel would be a drilled tunnel under the Bridgeport Harbor in generally a straight line between the two plants. The benefit would be to eliminate the required upgrade to the East Side WWTP and subsequent operation and maintenance of a second wastewater treatment facility. The existing structures and infrastructure could be abandoned/decommissioned and or demolished leaving the waterfront site available for future redevelopment. To convey a peak flow of 40 to 80 mgd between the sites, a single 54-inch to 72-inch gravity pipe would be required (see estimated pipe velocities in **Table 6.3-2**). Due to the high variations in daily flow and the infrequency of high flow peak events, two gravity pipes of similar or different size could be installed within the main tunnel structure to provide more acceptable pipe velocities at both minimum and peak flows and to have the ability to take one pipe out of service for cleaning, maintenance, or repair. For planning purposes, a peak flow of 80 mgd was assumed along with a straight-line 72-inch inner diameter (96-inch outer diameter) tunnel between the two plants was assumed, with a length of about 10,900 linear feet. With anticipated soft soils below the harbor, a closed face pressurized face tunnel boring machine (TBM) was assumed for construction of the tunnel, at an invert depth of approximately 85 to 95 feet below mean low water level. Including the cost for launch and retrieval shafts, the cost for the tunnel is estimated to be in the range of $108 million to $179 million.

Once conveyed to the West Side WWTP site, a deep, high-head pumping station would be required to lift flow to the treatment process. The cost of the pumping station is not included in the cost presented above.

Table 6.3-2 Pipe Flows, Pipe Sizes and Velocities

	Pipe Velocity (feet/second)			
Pipe Size	10 mgd	24 mgd	40 mgd	80 mgd
24”	4.92	11.82	19.70	
30”	3.15	7.56	12.61	
36”	2.19	5.25	8.76	
42”	1.61	3.86	6.43	12.87
48”	1.23	2.95	4.92	9.85
54”	0.97	2.33	3.89	7.78
60”	0.79	1.89	3.15	6.30
72”	0.55	1.31	2.19	4.38

Considerations

Consolidating the East Side plant with the West Side plant for all anticipated peak flows would require the following:

- Increase of preliminary treatment (screening and grit) from 90-200 mgd up to 130-280 mgd
- Increase of primary dry and wet weather treatment capacity from 90-200 mgd to 130-280 mgd
- Increase in biological nutrient removal (BNR) treatment capacity from 58 mgd to 82 mgd
- Increase in disinfection and effluent pumping from 90-200 mgd to 130-280 mgd
- Increase in sludge processing facilities from 98,000 pounds per day (lbs/day) max month to 116,700 lbs/day max month.

It is likely feasible that a new plant headworks (screening and grit removal) could be designed to accommodate both an increase in West Side collection system flow (up to 200 mgd) along with additional peak flow from the East Side plant (40 to 80 mgd). The preliminary treatment processes would be sized accordingly with an appropriate number of units to effectively treat the expected range of plant influent flow (18 mgd minimum to 280 mgd peak). A space-savings primary treatment system (e.g. primary filtration or high rate clarification) would be required as it is likely space would not exist for a traditional primary clarifier system. The primary treatment system could also be sized to treat the expected range of flows, however with a commensurate increase in footprint required. Alternatively, a separate wet weather treatment system could be constructed for flow above 82 mgd.

The concern with additional flow at the West Side WWTP pertains to the secondary treatment process to achieve biological nitrogen removal. The existing aeration tanks and secondary settling tanks at the West Side plant are already undersized under current flows and loads to meet the required permit limits year-round. Plant consolidation would increase the required secondary treatment capacity by 24 mgd to 82 mgd. Additional bioreactor volume as well as clarification capacity would be required to accommodate the additional flow and load, further increasing the footprint required to treat the added flow.

Increasing the West Side plant total flow potentially to up to 130-280 mgd will increase the size of the required disinfection system and the effluent pump station that is needed to pump the discharge against the 100-year flood elevation. More importantly the integrity of the existing outfall pipe to accommodate the increased pipe velocities, up to 13.1 feet per second (ft/s) at 280 mgd would need to be assessed and could necessitate rehabilitation (i.e. lining), or replacement. The existing plant currently can discharge up to 90 mgd directly to Cedar Creek. Increasing the total plant flow may require a new outfall with an alternate discharge location beyond the extent of Cedar Creek.

Conclusions

Option No. 1 to convey all flow to the West Side plant is considered NOT FEASIBLE with the current available real estate at the West Side WWTP site. While increasing the new headworks and primary treatment processes to accommodate an additional 40 to 80 mgd of flow is likely feasible if additional site footprint were available to the north or west of the current site, adding up to 24 mgd to the secondary treatment system is not feasible due to limitations of the current West Side WWTP secondary system to treat even the current flow of 58 mgd. Adding up to 80 mgd will also likely affect the integrity of the existing the outfall, which could necessitate effluent outfall replacement.

This option is only potentially feasible if a significant amount of land could be acquired from the marina for an expanded WWTP, but the assessment would likely require the construction of a new effluent outfall.

6.3.2.2 Option No. 2 – Convey 24 MGD (secondary treatment capacity) from the East Side WWTP to the West Side WWTP

This alternative includes the construction of a 24 mgd pump station on the existing East Side WWTP site to convey the maximum day secondary treatment capacity to the West Side WWTP for treatment. The pumping station would benefit from mechanical screening ahead of the pumps to protect them from large debris common in a combined system and to reduce the likelihood of clogging in the 24-inch to 30-inch force main. To accommodate the excess flow during wet weather events, storage volume of approximately 10 MG would be constructed at the site to contain the excess flow volume associated with a 1-year storm event. Flow in excess of the 1-year storm event storage capacity would result in CSOs in the system. Although the operation of an East Side WWTP would be eliminated, infrastructure associated with the screens, pumping station, storage tank and odor control equipment would still require some level of operator attention. The remaining East Side WWTP equipment and infrastructure would be removed and abandoned/decommissioned and/or demolished.

The current influent sewers have inverts that are approximately 15 feet below grade and the intent would be for the existing sewers to connect and discharge directly to the new storage tank to avoid pumping. The storage tank would therefore be below grade with access points to the tank brought up above the 100-year flood elevation plus three feet (elevation 30.60), about 24 feet above the inverts of the incoming sewers. Submersible pumps (1 to 2 mgd capacity) would be installed in the tank to drain it following the wet weather event when the influent flows dissipate to below 24 mgd, discharging to the main pump station for eventual conveyance to the West Side WWTP.

As the storage facility would be storing raw wastewater, there would be a need to periodically, and perhaps after each use, clean debris that had settled in the tank(s). A means to collect, remove, and handle this debris would have to be incorporated in a final design effort.

Considerations

There are a number of considerations that need to be analyzed as part of consolidating the dry weather secondary flow from the East Side plant into the West Side plant. The main issue is the same as Option No. 1, the existing primary and secondary treatment systems are undersized and unable to properly treat even the current peak secondary flow of 58 mgd and would not be able to accept additional flow in their current configuration and footprint.

Another consideration is even if the secondary flow is pumped from the East Side plant site to the West Side WWTP, there will still be facilities at the East Side plant site that must be operated and maintained including pumps, screens, odor control, and the wet weather storage tank(s) that is part of this Option. There will also be a need to clean the storage tank after an event and waste streams (screenings and storage tank debris) that must be disposed of. While these are manageable, there is still an associated cost and labor requirement that must be considered.

Conclusions

Option No. 2 to convey the secondary flow (24 mgd) to the West Side plant is considered NOT FEASIBLE/NOT RECOMMENDED with the current available real estate at the West Side WWTP site. As is the case with Option No. 1, adding additional dry weather flow up to 24 mgd is not feasible at the West Side plant given the inability of the current system to treat even the current peak dry weather flow of 58 mgd along with the limited footprint available to expand the primary and secondary systems.

An objective of plant consolidations would be to eliminate processes, equipment, and labor requirements at the East Side plant site. It would be undesirable and not recommended to convey dry weather flow to the West Side plant and abandon the treatment system at the East Side WWTP, yet still have a wet weather storage facility at East Side plant that would require cleaning and another pumping system to dewater it, along with the larger 24 mgd pump station.

6.3.2.3 Option No. 3 – Pump Raw Sludge from the East Side WWTP to the West Side WWTP

In this alternative, the East Side WWTP liquid process train would be upgraded to treat the future flows and loads, but the solids generated (excluding screenings and grit) would be pumped to the West Side WWTP for processing and hauling off site. The most straightforward approach would be to consolidate the raw primary sludge and waste activated sludge (WAS) into a common storage tank that would provide storage, and also act as a mixing chamber and pumping system wet well. A transfer pumping system would then convey the blended sludge to the West Side plant for processing (thickening and potentially dewatering). Two 6-inch or 8-inch force main pipes would be installed to provide redundancy in the event one becomes plugged, requires cleaning, or requires repair or maintenance.

Considerations

The advantage with this alternative is that all solids processing is consolidated in one location at a single plant with a common set of sludge processing infrastructure, equipment, operations, and labor effort. Filling and hauling operations for offsite disposal would also be concentrated to one location in the City.

A disadvantage with this approach is the generation of hydrogen sulfide (H_2S) gas typically observed with the mixing of primary sludge and WAS. There would likely be a need to provide odor control at East Side plant for the storage tank and any adjacent enclosed areas along with the pump room. The generation of H_2S gas in the long force mains would also have to be mitigated, likely with the use of air release valves at high points in the line, or at regular intervals. Lastly, odor control would likely be required at the West Side plant process/tank where the force mains discharge.

A variation to this option would be to haul raw liquid sludge from the East Side plant to the West Side plant with trucks/tanker trailers in lieu of pumping through a force main. However, this may prove to be an unattractive variation due to the high number of daily tanker loads required to transfer the volume of un-thickened raw sludge. Current daily WAS volume alone is greater than 110,000 gallons per day (gpd), yielding more than 18 truckloads per day.

The largest cost item in this Option is associated with the sludge force main pipes between the two plants. For planning purposes, a route through the City streets was assumed as opposed to open trench along the bottom of the harbor or directional drilling under the harbor. A proposed route is shown in **Figure 6.3-1** (Route A) at the end of this Section. The planning level opinion of probable construction cost for this 3.4-mile route, assuming 6-inch piping, is in the range of $9.1 million to $12.1 million.

This page intentionally left blank.

DRAFT

Figure 6.3-1
Pipe Routes Between West & East WWTPs

This page intentionally left blank.

Conclusion

Option No. 3 to pump raw sludge from the East Side plant to the West Side plant and consolidate sludge processing operations at the West Side plant is FEASIBLE. There would be a benefit to have all sludge processing and hauling operations located at one plant with one set of equipment and infrastructure. All of the sludge processing equipment at the West Side plant already requires rehabilitation or replacement, adding the additional sludge flow from the East Side plant is just incrementally upsizing of the systems. There are concerns with potential odor and H_2S gas generations, but these are items that could be mitigated.

The main disadvantage with this option is cost, taking into the account the cost of larger sludge processing infrastructure and equipment at the West Side plant, the transfer pumping system at East Side plant, cost savings by eliminating sludge processing upgrade work at East Side plant, and the cost of a force main from the East Side plant to the West Side plant. Based on a planning level force main cost estimate of $9.1 million to $12.1 million, it is unlikely that the cost savings associated with consolidating sludge processing at the West Side plant and eliminating processes at the East Side plant will out weight these costs. However, the operational benefits of having consolidated sludge processing may offset any additional cost. The ultimate disposition of residuals processing will also play into the decision-making. If a more elaborate solids handling system is proposed now or in the future (e.g. digestion, heat drying, etc.) pumping sludge from the East Side WWTP to the West Side WWTP could become more attractive.

6.3.2.4 Option No. 4 – Pump Partially Thickened Sludge from the East Side WWTP to the West Side WWTP

In this alternative, sludge would be partially processed at the East Side plant if it is deemed beneficial to retain existing sludge processing infrastructure, e.g. if the existing Gravity Thickener (GT) tanks can be rehabilitated with equipment replaced and used to thicken primary sludge, WAS, or a combination thereof. The partially thickened sludge would then be conveyed to the West Side plant via a sludge pump transfer system and force main(s).

Considerations

The advantage with this is that it retains use of existing infrastructure, reduces the volume of sludge that has to be pumped to the West Side plant, and reduces the sizing/capacity of new sludge thickening infrastructure at the West Side plant. This alternative also becomes advantageous if it were recommended to dewater the sludge at both plants to cake for final hauling and disposal. This would consolidate the dewatering processes to one plant, reducing infrastructure and equipment.

A disadvantage with this option is the operational challenges of pumping thickened sludge over long distances which include increased pumping pressures and equipment sizes/horsepower (hp) and the potential for plugging of the force mains if they were to sit idle for an extended period of time. The option also becomes less advantageous if hauling and disposing of thickened sludge is the preferred disposal methodology. There is likely little benefit to partially thickening the sludge at the East Side plant just to pump it to West Side plant to be stored and pumped to a truck for hauling.

The largest cost item in this Option is associated with the sludge force main pipes between the two plants. As presented in Option No. 3, the planning level opinion of probable construction cost for this 3.4-mile route, assuming 6-inch piping, is in the range of $9.1 million to $12.1 million.

Conclusions

Options No. 4 to pump partially processed/thickened sludge to the West Side WWTP is NOT RECOMMENDED. There is a little benefit to partially thickening the sludge at the East Side plant and then pumping it to the West Side plant for only minimal additional processing, or perhaps only offloading and hauling offsite. This option would require sludge processing improvements (infrastructure, facilities, and equipment) at both plants as all sludge processing needs attention at both plants, eliminating the benefit of only performing sludge processing infrastructure and equipment upgrades at one plant and realizing those associated cost savings. Also, based on a planning level force main cost estimate of $9.1 million to $12.1 million, it is unlikely that the cost savings associated with partially eliminating sludge processing processes at the East Side plant will out weight improvement costs.

There would also be potential operational challenges associated with pumping partially thickened sludge over a long distance, approximately 3.4 miles depending on the exact route for the force main piping. At this distance with thickened sludge at potentially 2% to 4% solids, there would be higher pipeline friction and head loss, requiring larger and higher pressure transfer pumps.

6.3.2.5 Option No. 5 – Convey all Flow (up to 200 MGD) from West Side WWTP to East Side WWTP

This alternative includes the construction of a conveyance tunnel to convey 90 to 200 mgd from the existing West Side plant site to the East Side WWTP for treatment. The tunnel would be a drilled tunnel under the Bridgeport Harbor in a general straight line between the two plants. The intent would be to eliminate the operation of the existing West Side WWTP and the structures and infrastructure would be abandoned/decommissioned and equipment removed. To convey a peak flow of 90 to 200 mgd between the sites, a single 72-inchto 96-inch gravity pipe would be required (see estimated pipe velocities in **Table 6.3-3**). Due to the high variations in daily flow and the infrequency of high flow peak events, two gravity pipes of similar or different size could be installed within the main tunnel structure to provide more acceptable pipe velocities at both minimum and peak flows and to have the ability to take one pipe out of service for cleaning, maintenance, or repair.

Table 6.3-3 Pipe Flows, Sizes and Velocities

	Pipe Velocity (ft/s)				
Pipe Size	20 mgd	30 mgd	58 mgd	90 mgd	200 mgd
48”	2.46	3.69	7.14	11.08	24.62
60”	1.58	2.36	4.57	7.09	15.76
72”	1.09	1.64	3.17	4.92	10.95
84”	0.80	1.21	2.33	3.62	8.04
96”	0.62	0.92	1.19	2.77	6.16

Considerations

There are a number of considerations that need to be analyzed as part of consolidating the West Side plant into the East Side plant. The main item is if the additional West Side plant flow could be accommodated at the East Side WWTP along with an increased East Side plant flow itself. The plant secondary treatment system infrastructure would have to treat up to 82 mgd (24 mgd plus 58 mgd). And depending on the total collection system flow that is ultimately recommended to be conveyed to the West Side plant site (90 mgd to 200 mgd), the East Side plant's headworks, influent pump station, disinfection, and effluent pump station would have to be replaced with a new 130 mgd to 280 mgd system. Conveying all of the West Side plant flow to the East Side plant would necessitate a major expansion or more likely a full replacement of essentially all the existing plant infrastructure at the East Side plant (headworks, primary settling, aeration, secondary settling, solids processing, disinfection, electrical distribution, etc.).

Another disadvantage with this scenario is the construction of a large tunnel/pipeline, or multiple pipelines, between the two plants. A large diameter pipe is necessary to convey the flow from the West Side plant to the East Side plant via gravity. For planning purposes, a straight-line 72-inch inner diameter (96-inch outer diameter) tunnel between the two plants was assumed, a length of about 10,900 linear feet. With anticipated soft soils below the harbor, a closed face pressurized face TBM was assumed for construction of the tunnel, at a depth of approximately 85 to 95 feet below mean low water level. Including the cost for launch and retrieval shafts, the cost for the tunnel is estimated to be in the range of $108 million to $179 million.

With a tunnel 85 to 95 feet below mean low water level, a deep, high head pumping station would be required at the East Side plant to lift the West Side plant flow up into the new East Side headworks facility. This pumping station would be in addition to the pumping station that is required to lift the base East Side collection system flow into the liquid treatment process, as that pump station would be considerably shallower.

Increasing the East Side plant total flow to potentially 130 to 280 mgd (90 to 200 mgd plus 40 to 80 mgd) will increase the size of the required effluent pump station that is needed to pump the discharge against the 100-year flood elevation. Additionally, the existing plant currently discharges 40 mgd directly to Power House Channel. Increasing the total plant flow to a minimum of 130 mgd in this consolidation scenario will require a new outfall with an alternate discharge location beyond the extents of the Power House Channel. The permitting associated with a new outfall could delay the project, in addition to the time associated with constructing a new outfall likely further out into the harbor or even Long Island Sound.

Conclusion

Option No. 5 to convey all flow to the East Side plant is considered NOT FEASIBLE with the current infrastructure at the East Side site and with the available real estate at the East Side site. Adding 58 mgd of dry weather flow to the primary and secondary systems is impossible with the size of the current primary, aeration, and secondary tanks. There is also insufficient open land currently available at the site to double or over triple the size of these primary and secondary processes, depending on the potential treatment technologies that are under consideration (primary filters, integrated fixed film activated sludge (IFAS), membrane bioreactors, etc.).

While constructing new headworks and preliminary treatment systems at the East Side plant for 40 to 80 mgd to treat East Side collection system flow is likely feasible with open real estate along the northern site boundary, construction of facilities to treat 130 mgd to 280 mgd of consolidated flow would be a challenge within the existing property boundaries. Increasing the total plant flow to 130 mgd to 280 mgd will also require a new outfall with an alternate discharge location beyond the extents of the Power House Channel.

This option is only potentially feasible if a new location is selected for a completely new East Side WWTP, or construction of a completely new East Side WWTP could be accommodated at either of the adjoining parcels to the north or south. This increasing of total and secondary treatment flow would then just be an incremental increase in new system sizing. However, if a new site was secured for a new plant, the cost of the tunnel would have to be compared to the increased size of the East Side plant and the demolition and decommissioning costs at the West Side plant.

6.3.2.6 Option No. 6 – Convey 58 MGD (secondary treatment capacity) from the West Side WWTP to the East Side WWTP

This alternative includes the construction of a 58 mgd pump station on the existing West Side plant site to convey all dry weather flows that require secondary treatment to the East Side WWTP for treatment. The station could potentially include mechanical screening to protect the pumps from large debris that can be attributed to wet weather/CSO flow.

In contrast to East Side plant Option No. 2 to store excess flow, in this Option a standalone wet weather treatment system along with a wet weather pump station, new disinfection process, and potential odor control sized for 32 to 142 mgd would be constructed at the West Side plant site to accommodate the excess flow during wet weather events, the volume of excess flow is too large to capture and store. This facility would treat the wet weather flow to a primary level of treatment, typically required by the Connecticut Department of Energy and Environmental Protection (CT DEEP) for wet weather/CSO flow, and then disinfect it prior to discharge. Treatment alternatives could include traditional primary treatment, chemically enhanced primary treatment, primary filtration, or high rate clarification, potentially through repurposing of the existing primary and final settling tanks. Depending on which process was selected, additional pre-treatment processes would possibly have to be installed (e.g. high rate clarification and primary filtration would require ¼-inch fine screening along with grit removal). The new wet weather and disinfection processes would be constructed at a high enough elevation such that the wet weather could discharge by gravity without effluent pumping.

Considerations

The main disadvantage of this alternative is the operation of a 32 to 142 mgd satellite wet weather treatment facility/process along with the necessary wet weather pump station and screens. This is a system that could be highly automated, but it would require some level of regular inspection and oversight when in operation, or when being brought on line during a wet weather event. As wet weather events can be at times unpredictable and fast acting, it could be a challenge to have the necessary operators in the correct location at the appropriate time. A wet weather treatment system would also produce a solids stream that would have to be processed on site, pumped over to the East Side plant for processing, or hauled off site. The labor and expense necessary to handle these solids and also periodically clean and maintain this wet weather process and pump station must be considered.

There is also uncertainty as to whether or not the CT DEEP would approve a satellite/standalone CSO treatment system, as DEEP has typically not approved satellite CSO treatment systems in collection systems but instead required flow to be conveyed to wastewater treatment plants.

As a variation to this option, consideration was given to repurposing the existing primary tanks, aeration and secondary settling tanks, and the chlorine contact tanks for wet weather treatment and/or storage. With the need to discharge the flow against the recently revised 100-year flood elevation, the existing tanks are too low to permit a gravity discharge, triggering the need for an effluent pump station in addition to an influent pump station. Given this significant infrastructure requirement, repurposing existing tankage for wet weather treatment/storage is not a viable variation of this option.

If it were selected to abandon the West Side plant and pump dry weather flow to the East Side plant, another variation of this option would be to limit future flows to the site to the current 90 mgd, address wet weather CSO treatment/storage in the collection system, and only treat/store the excess volume of stormwater associated with a 30 mgd flow rate. Indications from WPCA is that there is a desire to avoid building CSO treatment/storage infrastructure in the collection system and instead handle the flows and volumes at the treatment plant(s), so this variation will not be investigated further.

Another major disadvantage associated with Option No. 5 is that the existing East Side plant primary and secondary treatment systems are only currently sized for 24 mgd and at times are not able to properly treat even these flows. These systems would have to be completely replaced to able to accept additional 58 mgd, and there is not sufficient available land at the current East Side plant site to construct primary and secondary systems for a total flow of 82 mgd.

Increasing the East Side plant total flow by 58 mgd will increase the size of the required effluent pump station that is needed to pump the discharge against the 100-year flood elevation. Additionally, the existing plant currently discharges 40 mgd directly to Power House Channel. Increasing the total plant flow by 58 mgd in this consolidation scenario will require a new outfall with an alternate discharge location beyond the extents of the Power House Channel, likely delaying the project for the permitting and construction.

Conclusion

Option No. 6 to convey the secondary flow (58 mgd) to the East Side plant is considered NOT FEASIBLE/NOT RECOMMENDED with the current available real estate at the East Side plant site. As is the case with Option No. 5, adding additional dry weather flow up to 58 mgd is not feasible at the East Side plant given that the current primary and secondary systems are only sized for 24 mgd, and there is insufficient footprint available to construct completely new primary and secondary systems.

An objective of plant consolidations would be to eliminate processes, equipment, and labor requirements at the West Side plant site. It would be undesirable and not recommended to convey dry weather flow to the East Side plant and abandon the treatment system at the West

Side plant, yet still have a CSO/wet weather primary treatment facility with influent pump station and screens and a separate 58 mgd dry weather pump station at West Side WWTP that would require oversight, cleaning, maintenance, etc. While these are manageable, there is an associated cost that must be considered.

6.3.2.7 Option No. 7 – Pump Raw Sludge from the West Side WWTP to the East Side WWTP

Similar to the approach discussed above in Option No. 3 to consolidate all sludge processing operations at the West Side plant, in this alternative, the West Side WWTP liquid process train would be upgraded to treat the future flows and loads, but the solids generated would be pumped to the East Side WWTP for processing and hauling off site. Raw primary sludge and WAS would combine in a common storage tank that would provide storage, and also act as a mixing chamber and pumping system wet well. A transfer pumping system would then convey the blended sludge over to the East Side plant for processing (thickening and potentially dewatering). Two 6-inch or 8-inch force main pipes would be installed in case one becomes plugged, requires cleaning, or requires repair or maintenance.

Considerations

The advantage with this alternative is that all solids processing is consolidated in one location at a single plant with a common set of thickening and dewatering infrastructure, equipment, operations, and labor effort. Filling and hauling operations for offsite disposal would also be concentrated to one location in the City.

Similar to consolidating sludge processing activities at the East Side plant, a disadvantage with this approach is the generation of H_2S gas typically observed with the mixing of primary sludge and WAS. There would likely be a need to provide odor control at West Side plant for the storage tank and any adjacent enclosed areas along with the pump room, mitigate H_2S gas in the long force main, and provide odor control measures at the East Side plant process/tank where the force mains discharge.

A variation to this option would be similar to the variation discussed in Option 3, haul raw sludge from the West Side plant to the East Side plant with trucks and tanker trailers in lieu of pumping through a force main. Again, this is an unattractive variation to the very high number of daily tanker loads required due to the volume of the un-thickened raw sludge.

The largest cost item in this Option is associated with the sludge force main pipes(s) between the two plants. For planning purposes, a route through the City streets was assumed as opposed to open trench along the bottom of the harbor or directional drilling under the harbor. A proposed route is shown in Figure 6.3-1 attached to the end of this memo. The planning level opinion of probable construction cost for this 3.4-mile route, assuming 6-inch piping, is in the range of $9.1 million to $12.1 million.

Conclusions

Option No. 7 to pump raw sludge from the West Side plant to the East Side plant and consolidate sludge processing operations at the East Side plant is FEASIBLE. There would be a benefit to have all sludge processing and hauling operations located at one plant with one set of equipment and infrastructure. All of the sludge processing equipment at the East Side plant already requires rehabilitation or replacement, adding the additional sludge flow from the West Side plant is just incrementally upsizing of the systems. There are concerns with potential odor and H_2S gas generations, but these are items that could be mitigated.

Similar to Option No. 3, the main factor with this option is cost, taking into the account the cost of larger sludge processing infrastructure and equipment at the East Side plant, the transfer pumping system at the West Side plant, cost savings by eliminating sludge processing upgrade work at West, and the cost of a force main(s) from the West Side plant to the East Side plant. Based on a planning level force main cost estimate of $9.1 million to $12.1 million, it is unlikely that the cost savings associated with consolidating sludge processing at the East Side plant and eliminating processes at the West Side plant will out weight these costs. However, the operational benefits of having consolidated sludge processing may offset any additional cost. The ultimate disposition of residuals processing will also play into the decision-making. If a more elaborate solids handling system is proposed now or in the future (e.g. digestion, heat drying, etc.) pumping sludge from the West Side plant to the East Side plant could become more attractive.

6.3.2.8 Option No. 8 – Pump Partially Processed Sludge from the West Side WWTP to the East Side WWTP

Similar to Option No. 4, in this alternative, sludge would be partially processed at the West Side plant if it is deemed beneficial to retain existing sludge processing infrastructure, e.g. if the existing Gravity Thickener tanks can be rehabilitated with equipment replaced or the new WAS rotary drum thickener equipment repurposed and used to partially thicken WAS, primary sludge, or a combination thereof. The partially thickened sludge would then be conveyed over to the East Side plant via a sludge pump transfer system and force main(s).

Considerations

The advantage with this Option is that it retains use of existing infrastructure, reduces the volume of sludge that has to be pumped to the East Side plant, and reduces the sizing/capacity of new sludge thickening infrastructure at the West Side plant. This alternative also becomes advantageous if it were recommended to dewater the sludge at both plants to cake for final hauling and disposal. This would consolidate the dewatering processes to one plant, reducing infrastructure and equipment.

A disadvantage with this option is the operational challenges of pumping thickened sludge over long distances including increased pumping pressures and equipment sizes/hp and the potential for plugging of the force mains if they were to sit idle for an extended period of time. The option also becomes less advantageous if hauling and disposing of liquid sludge is the preferred disposal methodology. There is likely little benefit to partially thickening the sludge at the West Side plant just to pump it to East Side plant to be pumped to a truck for hauling.

The largest cost item in this Option is associated with the sludge force main pipes between the two plants. As presented in previous options, the planning level opinion of probable construction cost for this 3.4-mile route, assuming 6-inch piping, is in the range of $9.1 million to $12.1 million.

Conclusions

Options No. 8 to pump partially processed/thickened sludge to the East Side WWTP is NOT RECOMMENDED. There is a little benefit to partially thickening the sludge at the West Side plant and then pumping it to the East Side plant for only minimal additional processing, or perhaps only offloading and hauling offsite. This option would require sludge processing improvements at both plants as all sludge processing needs attention at both plants, eliminating the benefit of only performing sludge processing infrastructure and equipment upgrades at one plant and realizing those associated cost savings. Also, based on a planning level force main cost estimate of $9.1 million to $12.1 million, it is unlikely that the cost savings associated with partially eliminating sludge processing processes at the West Side plant will out weight improvement costs.

6.3.3 Piping Routes

The West Side and East Side WWTP's are 2.15 miles apart in a straight-line distance. The potential routes described below proceed from the East Side plant to the West Side plant. It can be assumed that the conveyance of flow or sludge from the West Side plant to the East Side plant would follow a similar reversed path. These routes are also shown on Figure 6.3-1.

6.3.3.1 Route A – "Street" Route

This route would follow streets as much as possible. The pipe would generally follow the following route:

- Proceed North along Seaview Ave. and then west through open land toward Stratford Ave.
- Proceed west/northwest along Stratford Ave and under the two reaches of the harbor/riverway. Cannot run pipe along bridges as one is a drawbridge and the other is a vertical lift bridge.
- Proceed generally west/southwest along Fairfield Ave (Rt. 130).
- Proceed south along Bostwick Ave. to the West Side WWTP.

This route is approximately 3.4 miles long.

6.3.3.2 Direct Route

This route would be a more direct, shorter route between the two plants. The pipe would proceed west from the site and generally follow the following route:

- Proceed under the harbor and PSE&G Site (micro-tunnel).
- Proceed west/southwest, generally along Atlantic Street and Waldemere Avenue.
- Proceed west under Cedar Creek Harbor/Reach and O&G Bridgeport Asphalt Plant to the West Side WWTP (micro-tunnel).

This route is approximately 2.2 miles long.

6.3.3.3 Route C – "Harbor" Route

This route would generally avoid construction through/under existing roadways and properties and would primarily run under the harbor and Long Island Sound. The majority of the route would be tunneled, and the route would generally be as follows:

- Proceed southwest from the East Side WWTP under the harbor.
- Turn west around Seaside Park Point.
- Turn northwest and cross under the western portion of Seaside Park.
- Proceed west under Cedar Creek Harbor/Reach and O&G Bridgeport Asphalt Plant to the West Side WWTP (micro-tunnel).

This route is approximately 2.7 miles long.

6.3.3.4 Gravity Tunnel Route

This route would generally avoid construction through/under existing roadways and properties in the South End section of the City and would be a direct tunnel route between the two plants. The entirety of the route would be tunneled (via TBM) and the would generally be as follows:

- Launch shaft constructed at the East Side WWTP.
- Proceed west southwest from the East Side WWTP under the harbor.
- Retrieval shaft constructed at the West Side WWTP

This route is approximately 2.1 miles long.

6.3.4 Summary

The following **Table 6.3-4** is a summary of the options for plant consolidation.

Table 6.3-4 Plant Consolidation Summary

Option	Description	Conclusion
# 1	Convey all flow from the East Side WWTP (up to 40-80 mgd) to the West Side WWTP	Not Feasible
# 2	Convey 24 mgd (secondary treatment capacity) from the East Side WWTP to the West Side WWTP	Not Feasible / Not Recommended
# 3	Pump Raw Sludge from the East Side WWTP to the West Side WWTP	Feasible
# 4	Pump Partially Thickened Sludge from the East Side WWTP to the West Side WWTP	Not Recommended
# 5	Convey all flow from the West Side WWTP (up to 90-200 mgd) to East Side WWTP	Not Feasible
# 6	Convey 58 mgd (secondary treatment capacity) from the West Side WWTP to the East Side WWTP	Not Feasible / Not Recommended
# 7	Pump Raw Sludge from the West Side WWTP to the East Side WWTP	Feasible
# 8	Pump Partially Thickened Sludge from the West Side to the East Side WWTP	Not Recommended

Eight potential options to consolidate the East Side and West Side plants, ranging from consolidating the full flow to partial flows (dry weather) to just consolidating sludge processing.

Of the eight options, the only two that were deemed feasible were the ones to consolidate the processing of raw sludge at one plant, with the other plant pumping its raw sludge over. All of the other options were deemed not feasible or not recommended due to cost, impracticality, or minimal or no resulting benefit to the WPCA.

The two options to consolidate the sludge processing processes at one of the plants will be considered further as part of the overall plan for the two plants. However, the option to consolidate the sludge processing at the East Side WWTP may become not feasible if the projects to upgrade the two plants are staggered with the work at the East Side plant following the work at the West Side plant. If this were to happen, the upgraded West Side plant would not have a place to send/pump its sludge as the current East Side plant sludge processing infrastructure would not be able to accommodate the increased sludge flow. Therefore, the only feasible plant consolidation alternative would be to consolidate sludge processing at the West Side plant, with the East Side plant pumping its raw sludge after it's upgraded in the future.

6.4 Pumping and Preliminary Treatment

6.4.1 Introduction

Section 4 presents a summary of the existing influent pump stations and the preliminary treatment processes at the West Side and East Site WWTPs. Based on the system, infrastructure, and equipment deficiencies discussed, it is recommended that all new influent pumping and preliminary treatment processes be installed at both plants as part of the overall plant improvements. The intent is for new preliminary treatment to include coarse screening, grit removal, and a level of fine screening depending on downstream process requirements. Integrated within the preliminary treatment systems will be new influent pumping stations to lift the influent flow to allow for gravity flow through the downstream primary, secondary, and disinfection processes.

6.4.2 Influent Pumping

6.4.2.1 Design Approach/Criteria

The hydraulic elevation of the influent flow to the West Side WWTP and East Side WWTP is approximately 20 feet and 25 feet below the ground surface level respectively. As such, the flow must be lifted to increase the plants' hydraulic grade lines to allow for gravity flow through the existing downstream processes. It is also necessary/desirable to lift the influent flow to avoid having treatment processes well below grade level with excessively deep excavations and foundations. For these reasons, new influent pumping stations at both plants are required.

Station Location

Potential locations for the influent pump station within the plant process flow stream include upstream of all process, just downstream of coarse influent screens, downstream of fine screens, or downstream of grit removal. The advantage with the pump station upstream of all processes is that all of the downstream preliminary and primary treatment processes will be elevated above grade, reducing the amount of deep excavation, dewatering and foundations. The main disadvantage with this layout is that the pumps will be pumping the raw influent with no protection. Any large items that make their way into the sewer system and conveyed to the plant could end up in the wet well, potentially damaging the pumps. This is of increased concern in combined sewer systems like Bridgeport where debris can be collected in the stormwater systems.

A second alternative is to locate the pumps just downstream of a coarse screening process. The main advantage with this layout is that the coarse screens will remove the large, potential harmful debris, providing a reasonable level of protection for the influent pumps and limiting damage to them. The disadvantage with this layout is that is puts the coarse screen process below grade, resulting increased structure size/depth and construction costs and the need to convey screenings removed up to grade.

Another alternative would be to locate the pump station downstream of a coarse and fine screen process, and potentially even downstream of the grit removal process. The advantage with this configuration is that it provides maximum protection for the pumps. It also could allow for the coarse and fine screen systems to be located back to back, or even in the same channels, reducing the overall length of the preliminary treatment process/structure. In addition, this provides a lower profile for the new structures. The main disadvantage with this alternative is that it puts all, or nearly all, of the preliminary treatment process well below grade. With grit tanks that could be over 20 25 feet deep, the bottom of these tanks could very well be over 50 feet below grade resulting in increased construction costs related to excavation, support of excavation, concrete, anti-flotation ballast, etc. Again, screenings and grit removed from these processes would have to be conveyed up to grade.

Pump Quantity

The West Side and East Side WWTPs will both have wide ranges of flow in the future that increases as the plant capacity increases. The West Side plant flow could range from a minimum flow of 14.6 mgd up to 90 mgd or 200 mgd peak. The East Side plant flow could range from a minimum flow of 3.3 mgd up 40 mgd or 80 mgd peak. With such a wide range of flows for both stations, multiple pumps will be required at each station to accommodate the range of flows.

One alternative for the pump quantity/size would be to provide pumps that are of the same size. For example, this could be four 30 mgd pumps at the West Side WWTP in a three duty plus one standby arrangement, similar to the existing setup, or perhaps four or five smaller duty pumps to better handle low flow conditions and prevent pumps from cycling on/off. A peak flow of 200 mgd at the West Side WWTP would likely be a minimum of seven 33.3 mgd pumps in a six duty plus one standby arrangement. These pumps would be driven by motors sized on the order of 400 hp to 475 hp based on conceptual estimate for anticipated discharge head/pressure.

For East Side plant, this could be five 10-mgd pumps for the 40 mgd flow in a four duty plus one standby pump arrangement. For an 80-mgd peak flow, this could be six 16-mgd pumps in a five duty plus one standby arrangement. These pumps would be driven by motors sized on the order of 100 hp to 175 hp based on conceptual estimate for anticipated discharge head/pressure.

The second alternative for pump quantity and size would be to provide pumps of different capacities/sizes. The driver for this approach would be to provide a pump(s) sized specifically to accommodate the typical daily flows to improve daily pump operation and efficiency. The balance of the required pumps would then be sized to accommodate the less frequent high/peak flows. Using a 200 mgd peak flow at the West Side WWTP as an example, three 20 mgd pumps could be installed to handle the more frequent daily average and peak dry weather/secondary flows up to 58 mgd, with the pumps potentially having sufficient turndown to avoid pump cycling at nighttime and low flow conditions. To convey the remaining balance of up to 142 mgd of wet weather flow, three to four pumps sized for 47.3 or 35.5 mgd could be installed to accommodate these more infrequent flows. The station standby pump would be the size of the larger pump so that the station would not lose any capacity if any one pump was out of service. **Table 6.4-1** below represents summary of potential pump scenarios for the East Side and West Side WWTPs.

Table 6.4-1 Pump Size/Quantity Example Scenarios

Parameter	West Side WWTP 90 mgd	West Side WWTP 200 mgd	East Side WWTP 40 mgd	East Side WWTP 80 mgd
Equal Size Pumps	3+1 @ 30 mgd	6+1 @ 33.33 mgd	4+1 @ 10 mgd	5+1 @ 16 mgd
Varying Size Pumps	3 @ 20 mgd 1+1 @ 30 mgd	3 @ 20 mgd 3+1 @ 47.5	N/A	2@20 mgd 3+1 @ 20 mgd

The size and quantity of the pumps at each of the two new pump stations will be determined during final design when final facility layout is set, and the final plant hydraulics are calculated. At that time, the required discharge head of the pump station can be accurately determined. The design team will also review historic and projected flow data in more detail to better understand the frequency of average, minimum, and peak flows. The design team will then work with pump vendors to determine a pump quantity/size arrangement that can best address average flows, minimum flows, and peak flows taking into account pump efficiencies, best operating points, and turndown.

6.4.2.2 Pump Alternatives

A summary of the types of pumps considered for the East Side and West Side WWTPs pump stations are discussed below and summarized in **Table 6.4-2**. The pumps considered include:

- Dry-Pit Direct Coupled Centrifugal
- Dry-Pit Extended Shaft Centrifugal
- Dry-Pit Submersible Centrifugal
- Wet-Pit Submersible Centrifugal
- Axial Flow
- Archimedes Screw

Dry-Pit Direct Coupled Centrifugal Pump

Dry-pit direct coupled centrifugal pumps are rotodynamic pumps with a circular volute housing and vane-type impeller. They are designed to achieve wide ranges of pumping rate capacities at wide ranges of discharge heads when compared to similarly sized axial flow propeller pumps, or Archimedes screw pumps, with the ability to provide rated net lifts well in excess of 100 feet

Table 6.4-2 Pump Alternatives Analysis

Pump Type	Motor Arrangement	Advantages	Disadvantages
Archimedes Screw Pump (Open or Closed)	Geared motor	• No variable-frequency drives (VFDs) required, easiest to control • Can run dry, no NPSH requirements • Pumps are less affected by changes in wet well (inlet) water surface level	• Large pump footprint • Large and heavy assemblies requiring specialized maintenance • Maintenance accessibility issues • Not as efficient as other pumps, always lifting to high elevation • Aboveground discharge channel required • Cannot discharge to a header assembly • Can have maintenance issues with bearings
Axial Flow	Line Shaft Motor	• Pump operation is less affected by discharge conditions when set up in an open discharge • High capacity per pump - efficient with reduced power/motor size • Motor is accessible without pulling pump • Easier maintenance than submerged options • Can meet the required hydraulic conditions	• Limited discharge pressures and greater submergence required • Additional access required for motor maintenance • More sensitive to static head changes than centrifugal pumps when installed in hard-piped application • VFDs required
	Submersible	• Pump operation is less affected by discharge conditions when set up in an open discharge • High capacity per pump - efficient with reduced power/motor size • Reduced structure footprint	• Limited discharge pressures and greater submergence required • Pump must be removed for motor maintenance and inspection • More sensitive to static head changes than centrifugal pumps when installed in hard-piped application • Pump/motor and seal repair and maintenance is labor intensive and may require work at specialty shop • VFDs required
Centrifugal Volute Pump	Wet-Pit Submersible	• Reduced structure footprint - elimination of pump dry pit • Lower power costs • Higher discharge heads compared to axial flow and screw pumps	• Maintenance requires pump removal from wet pit • Seals can be more difficult to maintain • Unable to visually inspect pumps and discharge piping • Pump/motor repair and maintenance is labor intensive and may require work at specialty shop • Reapir to discharge piping requires half the staiton to be offline • Permanently submerged and susceptible to water intrusion • Seal failure could lead to severe damage to motor and/or cooling system • VFDs required
	Dry-Pit Submersible	• Easier accessibility for inspection • Lower power costs • Higher discharge heads compared to axial flow and screw pumps • Submersible capability protects the pumps and station against flooding of the dry-pit	• Motor cooling required (open or closed loop cooling system) • Requires additional subsurface structure for dry pit • Seals and leak sensors can be more difficult to maintain • Pump, motor, seal repair and maintenance is labor intensive and may require work at specialty shop • Seal failure could lead to severe damage to motor and/or cooling system • VFDs required
	Dry-Pit Vertical Direct Coupled	• Pumps and motor located in dry gallery - easier inspection • Standard electrical motors, no internal cooling system • Easier maintenance than wet-pit and dry-pit submersible options • Lower power costs • Higher discharge heads compared to mixed/axial flow pumps • Motor is accessible without pulling pump • N long extended shafts	• Requires additional sub surface structure for dry pit, larger footprint • Susceptible to catestrophic damage if dry-pit floods from pump, piping, or wet well wall failure • VFDs required
	Dry-Pit Vertical Extended Shaft	• Pumps and motor located in dry gallery - easier inspection • Less susceptible to damage from partial dry-pit flood - motor located above grade and above flood elevation • Easier maintenance than wet-pit and dry-pit submersible options • Lower power costs • Higher discharge heads compared to mixed and axial flow pumps • Motor is accessible at grade without pulling pump	• Requires additional sub surface structure for dry pit • Requires additional level in structure for motor mounting • Proper support of shaft is required • Issues with shaft alignment or vibration could lead to premature failure of pump, shaft, seals, or bearings leading to leaks • VFDs required

This page intentionally left blank.

depending on the pump performance characteristics. Centrifugal pumps are suited to discharge to a gravity system or to a closed header system due to their ability to handle variable discharge heads.

With direct coupled pumps, the motor is directly coupled to the impeller shaft at the pump volute. Direct coupled pumps can be installed in a horizontal or vertical orientation, with the vertical setup being the most common in wastewater pump station applications as it greatly reduces pump footprint as the motor sits on top of the pump volute. In the dry-pit arrangement, the pump is typically located in a pump gallery adjacent to the station wet well.

Advantages and disadvantage of the dry-pit direct coupled pumps are discussed in Table. 6.4-2. The main issue with this pump and station style is that a catastrophic failure of the pump, the pump suction or discharge piping, or the pipe connections at the wall separating the gallery from the wet well could lead to flooding of the dry-pit, taking the entire station out of service with difficult repairs.

Dry-Pit Extended Shaft Centrifugal Pump

Dry-pit extended shaft pumps are very similar to dry-pit direct coupled pumps, except the motor is connected to the pump volute/impeller with an extended vertical shaft. In this arrangement, the pump itself is typically located a lower level of the dry pit gallery and the motor is located on an upper level. The motor is then connected to the pump with a metal drive shaft that is typically supported laterally to prevent vibration of the shaft. Bearings are provided in the lateral supports to reduce friction. The number of supports depends on the length of the extended shaft.

Advantages and disadvantage of the dry-pit extended shaft pumps are discussed in Table. 6.4-2. A distinct aspect of this pump is that the motor is located above the dry-pit pump gallery, greatly reducing the potential for damage from flooding of the dry-pit. Further protection of the motor can be attained when the motor is also located above the 100-year flood elevation plus three feet. Additionally, the motor is located at an upper operating level, allowing for even greater access for maintenance, repair, and removal/replacement.

Dry-Pit Submersible Centrifugal Pump

Dry-pit submersible pumps are similar to the dry-pit direct coupled pumps in that the pump motor is directly coupled to the pump impeller shaft and volute. However, with these pumps, the motor is sealed to the pump and also sealed itself, with the entire unit capable of operating in a submerged condition. The pump is designed to operating in a dry environment, but it is protected from damage and failure should the dry-pit flood.

As these pumps have motors that are sealed to outside air, they very often (especially in the case of large hp pumps) require a cooling system jacket around the motor to prevent it from overheating. The cooling system design in one of two ways. The first design has the pumped media surround the motor housing to cool it off. A small impeller on the pump shaft circulates a small volume or the pumped fluid around the motor housing for cooling. The potential concern with this is for debris in the pumped flow to clog the cooling system. The alternative design is for a closed loop system around the motor housing. The coolant is a glycol cooling liquid that is circulated by a small impeller connected to the pump shaft.

As the pump and motor are sealed together to allow for uninterrupted submerged operation, dry pit submersible pumps have to utilize a series of mechanical seals to prevent cooling liquid from getting into the motor housing. And in the case of pumps with closed loop cooling systems, there are also seals to prevent the pumped fluid from entering the closed cooling system. As the intrusion of water into the cooling jacket or motor could cause serious harm to the pump, dry-pit pumps are typically supplied with a series of sensors to detect the presence of water in the cooling system or motor housing.

Detailed advantages and disadvantage of the dry-pit submersible pumps are discussed in Table. 6.4-2. The main advantage with dry-pit pumps is the ability for them to be installed in a pump gallery adjacent to the wet well without the concern of dry-pit flooding due to catastrophic failure of the pumps or piping or wet well walls, however these pumps are more difficult to maintain and repair.

Wet-Pit Submersible Centrifugal Pump

Wet-pit submersible pumps are very similar in construction to the dry-pit submersible pumps except that they are installed directly in the wet well. They are a fully sealed pump designed and required to operate in a submerged condition. They do not require any suction piping as the pump sits in the wet well. The pump discharge piping is permanently installed in the wet well. The pump engages with the discharge piping as it is lowered into the wet well on a rail system. The weight of the pump provides the force required to seal the pump to the discharge pipe.

Detailed advantages and disadvantage of the wet-pit submersible pumps are discussed in Table. 6.4-2. The main advantage with the wet-pit submersible pumps is footprint reduction. However, as they are located in the wet well, they are not visible for inspection, they must be removed for any kind of maintenance or repairs, and the nature of the sealed motor housing and cooling system also makes it difficult to perform maintenance and repairs on the motor, cooling system, and seals in the field. Significant repairs or maintenance generally requires the motor or complete pump assembly to be shipped to a specialty shop for work.

Operating submerged, the pump is also susceptible to water intrusion at all times. While these pumps are specifically designed for these conditions, there is always the potential for water entering the pump/motor housing. Therefore, these pumps are reliant on the mechanical seals to protect the motors and the integrated pump moisture sensors to detect any leaks. If there is damage to these seals or sensor, the repairs can be labor intensive and likely require the work to be performed a specialty shop.

Axial Flow

Axial flow pumps are rotodynamic pumps with propeller-type impellers. They are designed to achieve high pumping rate capacities at low discharge heads when compared to similarly sized centrifugal impeller pumps, providing a rated net lift of up to 40 feet depending on the pump performance characteristics. Axial flow pumps are suited to discharge to a gravity system, however they can connect and discharge to closed header system through the use of an automatic discharge valve that aids during pump start-up.

Submersible axial flow pumps are configured with the motor directly connected to the pump impeller, with the entire pump assembly mounted in a vertical riser column, submerged below the water surface of the wet well. Motor and impeller are contained within a common housing. The motor is of special construction with seals to keep the motor interior dry. The motor depends upon contact with the pumped medium for cooling. The vertical column in which the pump is installed can be accessed from the top via a removable cover. Dry motor construction locates the motor above the water surface, driving an extended line shaft arrangement that transmits the motor drive to the pump impeller located at the base of an extended column, submerged in the wet well. Because of the low operating speed typical of axial flow pumps, the motor may be paired with a gear reducer to provide a cost-effective drive system.

The pumping efficiency of axial flow pumps varies depending upon the specific operating conditions. Axial flow pumps must be selected with care to match the pump operating characteristics to the system hydraulic conditions, to result in pump operation as close as possible to best efficiency point (BEP).

If axial pumps connect to a common closed-pipe discharge header instead of individual free discharges (i.e. to a common open channel), the discharge flow rate of axial flow pumps can be sensitive to variations of operating head caused by variations in the receiving water and variation of pipeline friction losses caused by flow rate and pipeline velocity variation. Increased discharge pressures will lead to a reduction in pump throughput at a constant source water elevation. This is a design element that would have to be considered in a final design effort when determining required pump performance specifications.

Variable frequency drives (VFDs) will be also be required to control pump rotational speed to match pumping capacity to the incoming flow along with instrumentation to control the number of pumps running and their operating speed to ensure the pumps respond promptly and properly to changes in flow and water surface elevation in the wet well.

Detailed advantages and disadvantage of the axial flow pumps are discussed in Table. 6.4-2. The main advantages are the vertical arrangement with reduced structure footprint and the potential high efficiency. But submersible motors tend to have more labor-intensive repairs and maintenance.

Archimedes Screw Pump

Archimedes screw pumps are constructed with open helices mounted on a central tube to drive fluid up an inclined trough or with helices assembled within an enclosed tube to convey flow up the tube. The screw arrangement is rotated by a constant speed motor and gear reducer typically mounted at the top of the screw drive shaft. Fluid conveyance by an Archimedes screw pump is not pressurized like a rotodynamic pump, but rather the service fluid is 'pushed' up the inclined trough or tube at atmospheric pressure. Fluid is pumped to the higher elevation of the gravity tank to provide the required static head to convey the design flow rate under all downstream flow and head conditions to the outfall.

The open screw pumps are typically arranged to standard inclination angles of 30 or 38 degrees above parallel. Enclosed screws can have an angle of incline up to 45 degrees. The pumps draw their water from an open well and then discharge to an open tank or channel. They do not have the ability to have a pressure discharge to a closed piping system.

The advantage with Archimedes screw pumps is their ease of operation and reduced operational and control constraints. These pumps do not require a minimum submergence level which allows them to run dry without concerns for long-term damage to the pump. Archimedes screw pumps operate at a constant speed and vary their pumping capacity based upon the submergence at the pump inlet. As the water surface elevation in the wet well lowers, the pumps automatically draw less flow. Therefore, variable speed controls are not required to adjust the speed of the pumps to match pumping capacity to influent flow rates. The number of pumps in operation is also dictated by the water surface level at the pump inlet. At the level increases, additionally pumps are brought online. Similarly, pumps turn off as the pump inlet level decreases.

Although Archimedes screw pumps are a simplistic option for lifting large quantities of water for short lifts, there are drawbacks to the units. Flow through the inclined trough is generally slower than for a rotodynamic pump at the same rated pumping capacity. To achieve the same volumetric flow rate, a greater cross-sectional area is required. Therefore, Archimedes screw pumps become large and heavy when compared to rotodynamic alternatives. These results in pump station size and footprint that are significantly larger that comparably sized rotodynamic pump stations.

Archimedes screw pumps are also limited in the lifting head that they are capable of providing. With install angles of 30-45 degrees, a long laying length is required as the lift height increases, further adding the already large size of these stations. Therefore, Archimedes screw pumps are generally applied to high flow, low lift applications (less than 30 feet).

Another disadvantage of Archimedes screw pumps is the discharge channel/structure required to generate enough static head to drive flow through the downstream process as these pumps are not capable of discharging to a pressurized pipe system. The discharge channel has to be high enough to provide sufficient hydraulic gradient. The drive system of the screw pump will then extend at least an additional 10 feet above the top elevation of the structure.

A disadvantage related specifically to enclosed screw pumps is if the inlet water level rises above the top of the tube, air cannot enter the tube and a vacuum is created. If a vacuum occurs, the pumping capacity may decrease by up to 25 percent. Excessive intake water surface elevation can flood the lower rollers, bearings, and the lower forged ring leading to accelerated corrosion and deterioration of these components and potentially catastrophic failure of the lower bearings and rollers. Enclosed Archimedes screw pumps are also generally twice as expensive as open screws when comparing only equipment capital costs.

The advantages and disadvantage of the Archimedes screw pumps are summarized in Table. 6.4-2.

6.4.2.3 Wet Well Alternatives

There are two main styles of pump station wet wells that are applicable for the size stations required at the West Side and East Side WWTPs. They are long trench style wet wells and rectangular wet wells.

Trench Style

Trench style wet wells are long narrow wet wells suitable for use with dry pit pumps and also wet pit submersible pumps. They typically lie in line with the upstream flow with the pump suctions oriented perpendicular to the flow (in the side wall of the wet well). They are generally narrow in width with very little wet well storage volume, therefore they are suitable for us only with pumps controlled through VFDs where a target wet well elevation is maintained. They are typically not set up to operate in fill/draw scenarios with pumps cycling on/off. Hydraulic Institute (HI) standards have specific design guidelines for the station configuration.

In large stations where a split wet well is desired to allow for half the wet well and pumps to be taken off-line for pump or wet well maintenance, two parallel trench wet wells can be installed. The pumps can then be split evenly between the two wet wells. The wet well floor is also usually sloped down toward the last pump, with one or more steep spillways. This allow for flushing of the wet well floor by drawing down the wet well water surface level through the use of the most downstream pump.

A main advantage with the trench style wet well is its reduced footprint. Also, the sloped wet well floor with spillway(s) allows for them to be flushed periodically, beneficial in CSO systems where there can be significant amounts of grit and other solids being conveyed to the plant.

A disadvantage with the trench style wet well is its small storage volume. This requires pumps to operate off of VFDs to maintain set water surface levels. There is not a real opportunity to operate in a fill/draw operation. This requires the pumps to be sized appropriately with sufficient turndown to allow them to remain in operation at the minimum low flows.

Rectangular

Rectangular wet wells are generally larger in size and volume. Flow typically enters along one of the long sides of the well, with the pumps/suctions located along the opposite side. The rectangular shape is not always conducive to proper hydraulic flow within the wet well and into the pump suctions. Individual pump compartments, baffles, formed suction openings, etc. are typically recommended by the HI to prevent undesirable hydraulic conditions that could negatively affect pump operations, reduce pump capacity, or reduce pump efficiency.

To assist with pump, piping, and wet well maintenance and repairs, rectangular wet wells are typically of a split wet well design with a concrete separation wall. Pumps are evenly split between the two separate wells. A gate is typically incorporated into the separation wall to allow for the wells to operate as two individual wells with the gate closed, or as one large well with the gate opened (more common). The gate can used to isolate a well when one needs to be taken out of service.

6.4.2.4 Pump Evaluation

While Archimedes screw pumps are efficient and relatively “simple” with respect to operation and control, they are limited in pump discharge height/head and require much larger footprints when compared to similar capacity rotodynamic pump stations. Therefore, Archimedes screw pumps stations are not applicable for either the East Side or West Side WWTP influent pump station based on the preliminary estimate of pump station discharge needs (approximately 45 to 60 feet).

Axial flow pumps are efficient at pumping large quantities of flow, but only at lower discharge heads/pressure, less than 40 feet. Based on the preliminary estimate of pump station discharge needs (approximately 45 to 60 feet), axial flow pumps are not applicable for either the East Side or West Side WWTP influent pump station.

With their wide range of flow capacities and discharge head capabilities, all of the dry-pit pump options are applicable for consideration at the East Side and West Side WWTPs. Preliminary inquiries with pump vendors indicate that there are available pump selections that make these types of pumps viable options.

Similar to the dry-pit centrifugal pumps, the wet-pit submersible pumps offer a wide range of flows and discharge head capabilities and there are available pump selections for the range of proposed flows at the East Side and West Side WWTPs. They also offer a pump station with a reduced footprint as there is no station dry-pit. However, the wet-pit submersible pumps do have a number of negative aspects including fully submerged operation, inability to visually inspect the pumps and immediate discharge piping, need to lift a pump out of the wet well for any form of inspection and/or maintenance, reliance on sensors to determine any leaks of the sealing system, etc. With East Side WWTP pumps potentially ranging from 100 to 200 hp, and the West Side WWTP pumps being as large as 500 hp, removing these pumps from the wet well for any kind of inspection or service would be a labor-intensive activity. For these reasons, and considering that wet pit submersible pumps would be permanently submerged, wet pit submersible pumps are not recommended.

6.4.2.5 Summary and Recommendation

As discussed in the previous sections, the various variations of the dry pit centrifugal pumps are the viable pumps for the influent pumping stations at both the West Side and East Side WWTPs. Each variation of the dry pit pumps has its own advantages and disadvantages, but the main thing they have in common is that they are all located within a pump gallery and easily accessible for visual inspection, which is an advantage over the wet pit submersible pumps considering the potential size of the pumps for the West Side and East Side WWTPs (up to 450-500 hp).

Each of the dry-pit pumps has its own merit and drawbacks. The dry-pit direct coupled pump is the most straightforward option, as it involves standard motors mounted directly on the pump with no extended shaft to accommodate. However, with the influent pumps located well below grade in a gallery due to the hydraulic grade line of the influent interceptors, there is concern about them becoming flooded from either a pump or piping failure, leak in the wet well separate wall, or from a catastrophic flooding of the site. Given that the influent pump station is necessary for the plant to operate and to prevent backups of the collection system, having motors in a location where they could possibly (however remote) become damaged is not advisable from a risk standpoint. For this reason, dry pit closed coupled pumps are not recommended.

The main advantage of the dry pit submersible pumps is that they remove the flooding risk associated with direct coupled pumps as their motors are fully sealed and are capable of operating in a submerged environment, allowing for uninterrupted station operation should there be a catastrophic event that floods the below grade pump room. The drawbacks of these pumps is generally associated with the submersible capability of the motor; motor cooling system to monitor and maintain, mechanical seals and motor/seal chamber leak sensors in the pump housing can be difficult to maintain, and motor maintenance and repair may require off-site work at a specialty shop. With the additional complexities of the nature and design of the dry pit submersible pumps and motor, these pumps are not recommended for either the West Side WWTP or East Side WWTP.

The recommended pumps for the West Side and East Side WWTPs are the dry pit extended shaft pumps, similar to the pumps that are currently in use at both plants. These pumps provide flood protection by allowing the motor to be kept out of the below grade pump room and also above the 100-year plus 3 feet flood elevation mandated by CT DEEP for critical plant infrastructure. This design also allows for the installation of standard electrical and eliminating the complexities associated with submersible style motors (internal mechanical seals, seal leakage sensors, motor cooling systems). The disadvantage with these pumps is the extended shaft and the potential for vibration and unbalance and the effect on the pump seals. But this can be mitigated with sufficient shaft supports and routine inspection and maintenance of the pumps, the shaft connections, any support bearings, and the seals.

Wet Well – West Side WWTP

The influent pump station at the West Side WWTP will be a very large pump station ranging from 90 mgd to 200 mgd depending on the recommended future plant flow, with anywhere from 3 to 6 operating pumps with an additional standby unit equal to the capacity of the largest pump (if station contains more than one sized pump). The wet well for the station will be designed in accordance with industry guidelines, namely the HI standards. While not as predominant as more traditional rectangular wet wells, trench style wet wells do offer some distinct advantages over rectangular wet well, namely being their reduced footprint, a large advantage given the very limited availability of open space at the site for new processes and the size of the new station. The trench style wet wells can also self-clean by using the last pump to quickly draw down the water surface level to flush the sloped floor, an advantage with a combined collection system like Bridgeport's that could convey large quantities of grit during a wet weather event. For these reasons, a trench style wet well is recommended for the West Side WWTP. The wet well should be a split wet well to allow for half the station to be taken offline for any required inspection, maintenance, or repair to the wet well, pumps, or suction piping and valves. However, this will be reviewed during final design when the detailed layout of building interiors and their placement on the site are further advanced to confirm that the trench style design is the optimal solution.

Wet Well – East Side WWTP

Compared to the West Side WWTP, the East Side WWTP influent pump station will be considerably smaller, 40 to 80 mgd. While this station still will likely have 3 to perhaps 5 or 6 operating pumps, plus a standby pump equal to the capacity of the largest pump, it will have smaller wet well requirements regardless of a trench or rectangular layout. For this station, a trench style wet well sized and dimensioned in accordance with HI standards is recommended for commonality of design and operation between the two plants. The wet well should be a split wet well to allow for half the station to be taken offline for any required inspection, maintenance, or repair to the wet well, pumps, or suction piping and valves. Again, this proposed approach will be reviewed during final design when the detailed layout of building interiors and their placement on the site are further advanced to confirm that the trench is the optimal solution for the East Side WWTP.

6.4.3 Influent Screening

6.4.3.1 Approach

Based on the deficiencies of the existing screenings system, infrastructure, and equipment, new screenings facilities are recommended for both the West Side and East Side WWTPs. The approach will be to provide initial screening of all the flow (dry and wet weather) at the head end of the plant upstream of all processes similar to the existing process to provide protection of the main influent pumps. To prevent blinding of these screens and to accommodate the high flows associated with CSO and wet weather events, the intent would be for these screens to be a coarse bar screen with a bar spacing between 1-inch and 2-inch to remove only the large harmful debris.

A number of the dry weather and wet weather processes being evaluated as part of the Facility Planning effort (certain grit processes, primary filters, IFAS, membrane bioreactors, high rate clarification) require additional fine screening of the flow for protection of the treatment process. For this reason, a fine screen process is also recommended for installation as part of the upgrades to the plant headworks, and a screen opening of ¼-inch was assumed for the planning effort. Given the flood elevation at the site, the below grade elevation of the influent flow, and to avoid having a deep process and screen structure, it's recommended that the fine screening process be located downstream of the influent pumping process. This will put the process above grade and above the 100 year plus three feet flood elevation and also keep the screens short in length, reducing their cost. Similar to the coarse screen process, the fine screen system would also be sized for the full influent flow to protect both the dry and wet weather treatment equipment.

West Side WWTP

With a peak plant flow of up to 200 mgd and minimum flows down to 10 mgd, multiple screens should be provided to maintain acceptable channels velocities over the range of flows. Working with screen vendors, using the 200 mgd peak flow as an example, three duty screens and one standby screen each sized for 67 mgd are recommended for the coarse screening process. Normal operation may consist of only one screen in service as a screen capable of passing 67 mgd would be sufficient to handle peak dry weather flows, however it may be more straightforward to have two screens in operation to reduce having to bring a second screen on online whenever flows go above 67 mgd. The third screen would be brought into service during the higher flow events. The fourth screen provides system redundancy during all flows should a screen be out of service for maintenance or repair.

For the fine screen process, four or five total screens each sized for 50 or 67 mgd (depending on the screen technology) is likely necessary, three or four duty and 1 standby, as the headloss through these screens will be higher and they will not be able to pass as much flow as the coarse screens. Normal operation will likely have two screens in operation to handle peak dry weather flows. The third and fourth screens would be brought into service during the higher flow events. The fifth screen provides system redundancy during all flows should a screen be out of service for maintenance or repair.

East Side WWTP

With a peak plant flow of 40 to 80 mgd and minimum flows of less than 5 mgd, multiple screens should be provided to maintain acceptable channels velocities over the range of flows. Working with screen vendors, two duty screens and one standby screen for both the coarse and fine screening processes are recommended. Normal operation would likely only require one screen to be in service as a screen capable of passing 20 to 40 mgd would be sufficient to handle peak dry weather flows along with a fair percentage of the smaller wet weather high flow events. The additional screen(s) would be brought into service during the higher flow events. The third screen provides system redundancy during all flows should a screen be out of service for maintenance or repair.

6.4.3.2 Screening Technology Alternatives

The following screen technologies that could be applicable at the West Side and East Side WWTPs are summarized herein.

- Climber Type Screen
- Multiple Rake Bar Screen
- Perforated Plate Belt/Band Screen
- Center Flow Screen
- Step Style Screen

While some technologies are applicable to coarse and fine screening, others may only be applicable to one of the processes.

Climber Type Screens

The West Side and East Side WWTP has been operating old climber type screens with ½-inch to ¾-inch bar spacing for over 20 years. There are minor operational concerns with these screens given their age, and the WPCA reports that the performance of the equipment has generally been problematic.

The climber screen is a mechanically cleaned bar screen with a single rake assembly. The physical setting of the bar screen rests at an angle in the influent channel while the large single rake arm removes debris. Each cycle of the large rake arm can remove a large volume of screenings during high flows with a lot of debris in the flow. The rake carries the debris from the low point of the bar screen to a discharge point above the channel. At the discharge point, a wiper assembly helps removes all debris from the rake arm. The rake teeth on many of the units fully penetrate the bar screens to avoid clogging. The drive motor and all other mechanical parts are located above the water elevation. The bar rack, rake assembly and side frame are typically all made of 304 or 316 stainless steel to mitigate potential corrosion concerns. Some manufacturers utilize a two-speed operation, allowing the screen to operate at a faster cleaning rate when the screen becomes blinded and the water level differential across the bars increases.

The screen utilizes guides tracks on the side frames for the rake assembly shaft rollers. This design eliminates the need for a lower sprocket and drive chain and thus the need for permanently submerged moving parts. However, this design is not low profile. Eliminating the need for permanently submerged parts increases the overall height of the unit above the operating floor and discharge point. A climber style screen is shown in **Figure 6.4-1**.

Figure 6.4-1
Climber Screen (source: Vulcan Industries)

Climber type screens are generally considered to be coarse screens with effective bar spacings of ½-inch and larger. In recent years, the trend has been pushing towards smaller openings and several 3/8-inch and even a few ¼-inch applications have been installed on separated systems with sanitary flow only. However, for combined systems, it's recommended that this type of screen only be considered for coarse screening scenarios with bar spacings of ½-inch or larger.

Multiple Rake Bar Screens

The multiple rake type screen is a mechanically cleaned bar screen that contains multiple rake assembles/bars attached to side mounted drive chains to permit quick cleaning of the bars and to reduce the amount of screen blinding.

The bar screen rests at an angle in the influent channel while the multiple rakes or flights rotate in/out of the channel to remove debris. The flights carry the debris from the low point of the bar screen to the upper discharge point. Each flight is smaller and carries less material than the single rake arm on a climber screen, but the multiple flights and varying operational speed create a comparable or even greater removal rate. At the discharge point, a scraper mechanism helps removes all debris from the rakes. The rake teeth fully penetrate the bar screens to avoid clogging.

The drive motor, upper drive shaft and upper chain sprocket are located above the water elevation mounted to the screen frame. The lower chain rotational point varies between manufacturers and can consist of a lower shaft, sprockets, and bearings; a stainless steel or UHMWPE semi-circular guide for the chain; or may utilize drains chains with large links that create a self-supporting "sprocket" turning point, eliminating the need for any form of chain guide. Manufacturers of screens with lower sprockets and bearings indicate that the systems are a self-lubricating design and grease lines are not required.

The design of the multiple rake screens allows for these units to have very low headroom requirements with only a minimal amount of equipment located above the screen discharge point. Some units are also capable of being pivoted out of the channel to allow for easy access for inspection and maintenance. The screen drive motor is typically controlled by a VFD to moderate the rate at which the rakes revolve, and some designs also include a self-reversing feature should the screen become blocked. A multi-rake screen is shown in **Figure 6.4-2**.

An advantage with the multiple rake screens is the wide range of bar spacings available. Screens can be provided with ¼-in bar spacing, satisfying fine screen requirements for many primary and secondary processes, and up through 1-inch and 2-inch bar spacing. This will allow for the same screen, just with different bar spacing, to be used for both the coarse and fine screening processes.

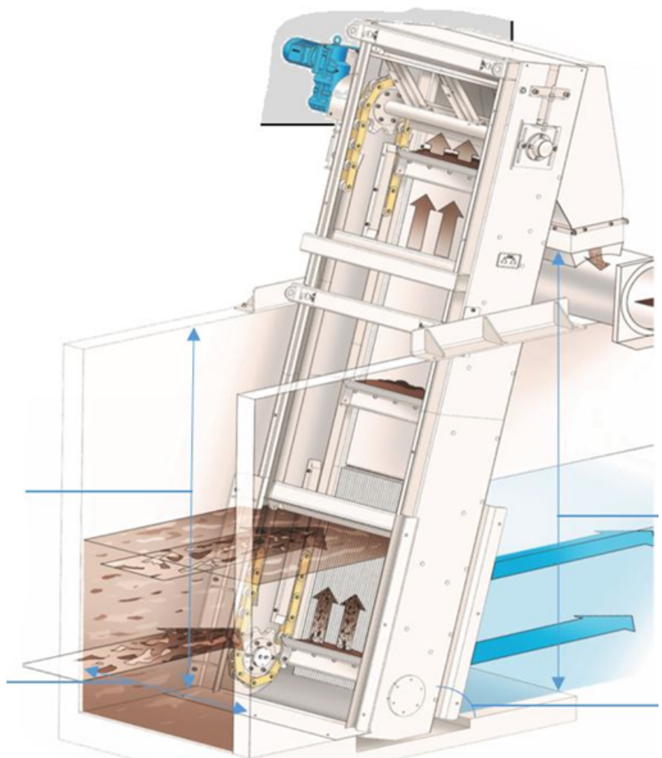


Figure 6.4-2
Multi-Rake Screen (source: Huber Technology)

There are a number of wastewater equipment companies that manufacture multiple rake screens. While each may have its own unique features, they are generally similar in fabrication and operation. Examples of multiple rake screens include the Headworks Inc. Mahr screen, the Huber RakeMax, the Chain & Rake Monster Bar Screen by JWC Environmental, and the FlexRake by Duperon.

Perforated Plate Belt/Band Screens

Perforated plate belt/band screens are continuous screens that rotate through the influent stream. The screens remove debris from the stream and convey it out of the channel and up to the operating floor for discharge. The belt/band screens can be connected perforated metal or UHMW plates sections or continuous plastic belts and the screen openings can be circular, rectangular, or square in shape, with openings as small as 1/8-inch (3 mm). The accumulated debris is then removed by water spray, sometimes with a counter- rotating brush. A perforated plate screen is shown in **Figure 6.4-3**.

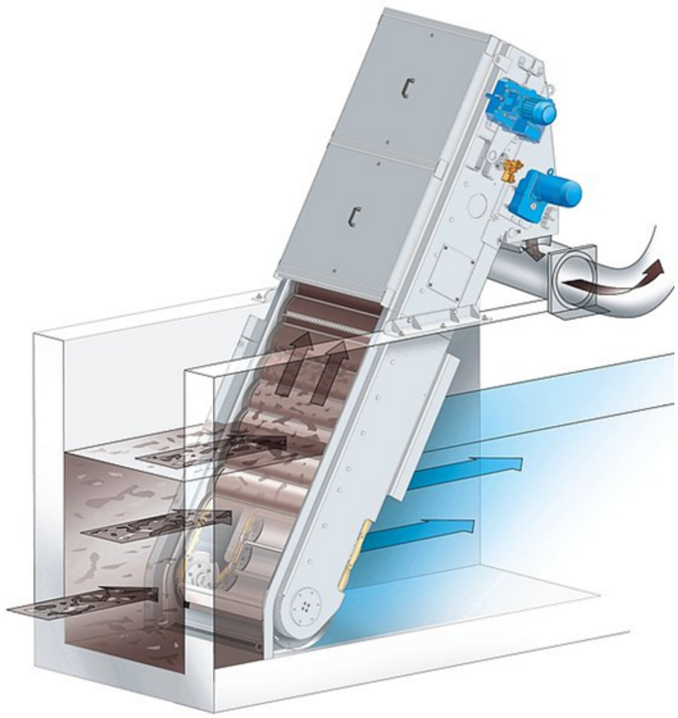


Figure 6.4-3
Perforated Plate Screen (source: Huber Technology)

The main advantage with the perforated plate screens is their high capture rate, and the circular/square openings collect stringy and other narrow material that could pass through fine screens that only consist of closely spaced screening bars. There are distinct disadvantages to the design of belt/band screens. The cleaning brush or cleaning spray water at times may be unable to remove stringy material from the screen openings. There is also the chance for screening carryover as the screen returns down into the channel behind the front face of the screen. This also creates higher headloss. An O&M concern is the submerged lower rotating assembly for the screen, but some manufacturers have addressed this with maintenance free bearings.

Center Flow Screens

Center-flow screens are designed as an elongated drum screen with an inside-out flow through the screen, utilizing openings in perforated plates. The screens are installed parallel to the flow path. A steel plate covers the back of the inside of the drum, perpendicular to the path of flow, and forces the flow out through both sides of the screen. The screenings are captured in the perforated drum with plates/“shelves” serving to aid in lifting material within screen drum. Material is washed from screen surface using a pressure water spray system at the high point of the drum and is deposited onto an internal trough that carries material away from the screen. A center flow is shown in **Figure 6.4-4**.

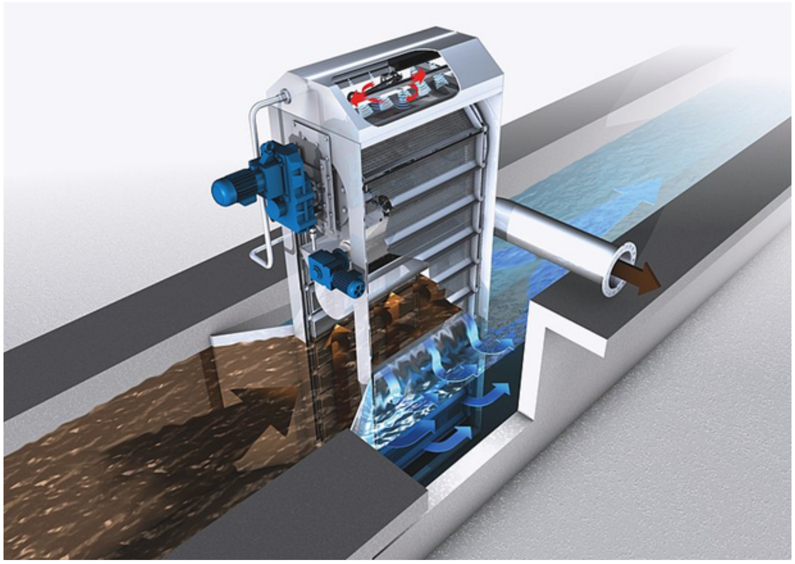


Figure 6.4-4
Center Flow Screen (source: Huber Technology)

The design of the center-flow screen provides many advantages. Because the screens are installed parallel to the flow, they provide twice as much screening area in the same footprint as other designs. Therefore, the headlosses are less when compared to similar size screen openings using other technologies such as perforated plates. The capture rate is very effective because the screened material is trapped within the screen, preventing contact with clean stream and allowing less material to carryover and continue downstream.

Step-Style Screens

The step-style screen design resembles an escalator. Equally spaced stainless-steel lamella plates move the screened material up and out of the channel one step at a time. Half of the vertical lamella plates remain stationary while half move up/down in an elliptical motion. A clean step is exposed at the bottom while the collected material is carried over the top and placed onto a discharge chute.

An advantage with the step-style screen is that the screened material remains on the steps above the channel water line, providing dewatering of the screenings material prior to discharge. Step-style screens are also a low-profile design, requiring minimal equipment above the discharge point. Another advantage is that cleaning brushes and spray water streams are not required and the drive system is completely located above the water line.

The screening procedure of a single step movement is also advantageous. It allows the material collected on the steps to form a mat that acts as a filter to increase the screening removal efficiency. The screened material is then carried up and over to the discharge, carryover and contamination of the downstream flow is not a concern. A step screen is shown in **Figure 6.4-5.**

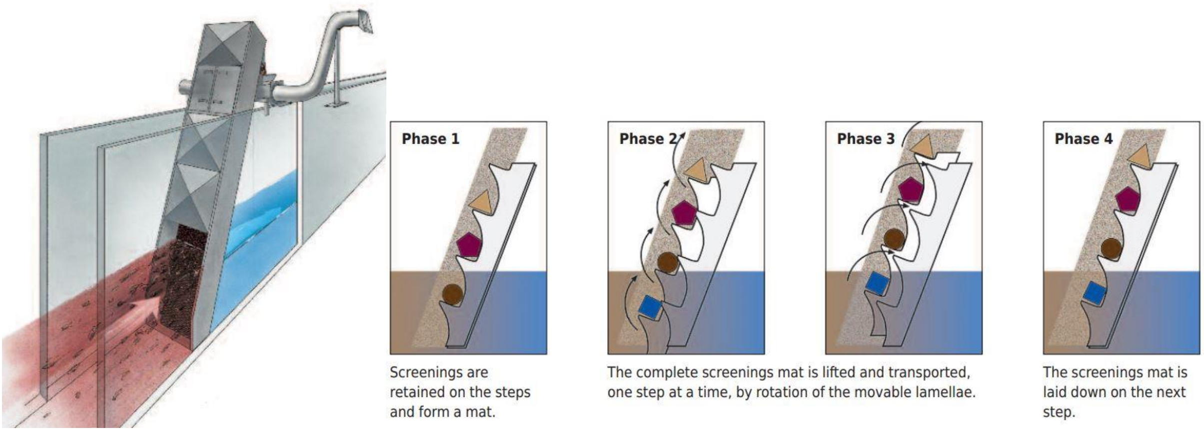


Figure 6.4-5
Step Screen (source: Huber Technology)

A disadvantage with this screen is that heavy material (e.g. grit, heavy solids) can collect on the channel floor below and in front of the first step. Manufacturers have claimed, however, that this is minimal with newer designs. Another disadvantage is that material can become jammed between the lamella bars, causing them to flex if the plastic lamella spacers have worn down. Excessive flex can cause the lamellas to rub against each other or possibly jam together. In addition, if the unit continues to try to move stuck plates, damage can occur, however, step screens can be installed with sensor systems to prevent this.

The screening capture ratio is lower for the step-style screen when compared to screens with similarly sized perforated openings as stringy and narrow material can pass through the parallel bars.

6.4.3.3 Screen Technology Evaluation/Assessment

A summary of the potential screens and their advantages and disadvantage is presented in **Table 6.4-3** with additional discussion below.

Coarse Screening

For the coarse screening process, climber type bar screens and multiple rake type bar screens are applicable technologies due to their robust construction (thick stainless-steel screen bars), wide range of bar spacing (1/2-inch to 3-inch), ability to remove large quantities of screenings material, ability to remove large sized debris from the channels. They will be considered for installation at both plants.

Perforated plate belt/band screens will not be considered for the coarse screening process as they are typically utilized for fine screening operations. They do not offer the large opening spacing required for the coarse screening process (greater than 1-inch), are not as robust in construction due to the perforated or mesh style screening surface.

Table 6.4-3
Influent Screen Alternatives Summary

Screen Type	Advantages	Disadvantages
Climber Style Bar Screen	• Applicable for large channels and high flows • No submerged moving parts • Wide range of bar spacings	• Long rake cycle • Tall screen height above operating floor • Unable to provide fine screening (i.e. 2-mm, 1/4-in)
Multiple Rake Bar Screen	• High capacity material removal with multiple rakes • Applicable for large channels and high flows • Wide range of bar spacings • Ability to serve as coarse and fine screen (1/4-in) • Low head room requirements • Lower cost compared to climber screens	• Potential for submerged parts/bearings • Potentially requires entry into channel for maintenance
Perforated Plate Belt/Band Screen	• Perforated openings for high capture ratio • Openings down to 1-mm • Low head room	• Potential for screenings carry over • Typically require spray water or brush for material removal and cleaning of the plates • Headloss increases as blinding increases
Center Flow Band Screen	• Perforated openings for high capture ratio • Openings down to 1-mm • High flow throughput • Low headloss • No potential for screenings carryover	• Requires wider channels • Requires spray water for cleaning of the screen and conveyance of material away from the screen (sluice launder)
Step Style Screen	• Bar spacing down to 1/4-in • Screenings "mat" that develops increases capture ratio • Low headloss	• Potential for major damage is the lamella bars jam • Potential for material to settle in channel before the first step • Limited overall screen height

This page intentionally left blank.

Similar to the perforated plate belt/band screens, center flow screens will not be considered for the coarse screening process. They are typically utilized for fine screen operations and are only offered with mesh or perforated type screens at openings less than 1-inch.

Step style screens are a bar style screen and could be appropriate for coarse screening operations. They will not remove excessive debris as is the concern for perforated type screens. However, the influent coarse screens will be tall, over 25 to 30 feet from the channel inverts to the discharge points. For this reason, step style screens will not be considered for coarse screening as there is concern of excessive flexing of the bars and potential jamming.

Fine Screening

While climber style and multiple rake bar screens are typically considered more of a coarse or general use influent screen, they can be utilized in fine screen operations, however there are limitations as to how close the bars can be spaced and how "fine" of a process they can provide. Due to the nature of the way the rake arms engage the screening bars on climber screens, they are only able to provide a bar spacing down to ½-in. Therefore, climber screens are not being considered for the fine screen process as many of the potential downstream processes (cloth filters, IFAS, etc.) require screening of ¼-in or finer. Multiple rake bar screens will be considered for the fine screening process as they have to ability to screen down to ¼-inch bar spacing. This could also allow for the coarse and fine screens to be of the same manufacturer and even model, just with finer bar spacing. The negative aspect of multiple rake bar screen is that it has slotted bar openings, not mesh or perforated openings, reducing the screens capture rate.

The perforated plate belt/band screens are generally purposely built for fine screening operations, mainly related to the design of the mesh or perforated screen surfaces that can provide screening openings down to 1 or 2-millimeter (mm) to provide a very high screening capture ratio and a high level of protection of the downstream equipment. The majority of the installations of these types of screens are specifically for fine screening operations. These screens are therefore being considered for fine screening processes.

The center flow screens are similar to the perforated plate screens, outside of the screen design and flow through the screen, in that they provide screening surfaces with very fine (1 to 2-mm) mesh or perforated openings. They are also generally purposely built for fine screening operations with the majority of the installation of these types of screens specifically for fine screening operations. These screens are therefore being considered for fine screening processes.

Step screens can be utilized for fine screen processes as they can be supplied with bar spacings down to 1/8-inch. The concern with step style screens for fine screening at either of the two Bridgeport plants is that the process will likely be upstream of the grit removal process. There is the potential for grit to settle out before the bottom screen step and become lodged between the fixed and moving lamella bars, creating a concern for the bars to jam. While screen manufacturers have improved designs to improve on this, the CSO flow in both collections systems combined with an unknown expectation for grit quantity and gradation provides a level of apprehension with the use of these screens. For these reasons, step style screens will not be considered for fine screening.

Table 6.4-4 below represents summary of potential screen scenarios for the East Side and West Side WWTPs.

Table 6.4-4 Influent Screening Options

Process	Screen Type	West Side WWTP 90 mgd	West Side WWTP 200 mgd	East Side WWTP 40 mgd	East Side WWTP 80 mgd
Coarse Screen (1-in spacing	Multi-Rake	2+1 @ 45 mgd 60 to 72-in channel	3+1 @ 67 mgd 60 to 72-in channel	2+1 @ 20 mgd 36 to 48-in channel	2+1 @40 mgd 60 to 72-in channel
	Climber Type	2+1 @ 45 mgd 72-in channel	3+1 @ 67 mgd 60 to 72-in channel	2+1 @ 20 mgd 48-in channel	2+1 @ 40 mgd 72-in channel
Fine Screen (1/4-in or 6-mm)	Multi-Rake	2+1 @ 45 mgd 60 to 72-in channel	3+1 @ 67 mgd 60 to 72-in channel	2+1 @ 20 mgd 36 to 48-in channel	2+1 @ 40 mgd 60-in channel
	Perforated Plate Belt/Band	2+1 @ 45 mgd 72-in channel	4+1 @ 50 mgd 72-in channel	2+1 @ 20 mgd 36 to 48-in channel	2+1 @ 40 mgd 72-in channel
	Center Flow	2+1 @ 45 mgd 78-in channel	3+1 @ 67 mgd 78-in channel	2+1 @ 20 mgd 78-in channel	2+1 @ 45 mgd 78-in channel

6.4.3.4 Screenings Processing

Screenings washer/compactor equipment is very common in conjunction with influent screens to reduce the disposal of organics and reduce odors, water, and volume prior to disposal. Screening compactors are compact units that generally consist of an enclosed spiral auger screw(s) that washes, dewaters, and compacts the screening material. Spray water systems within the compaction unit and sometimes at the inlet hopper work to first strip the organic material from the screenings materials. A perforated plate or other filter/screen at the bottom of the unit allows for the equipment to drain and for the water and organics to be returned to the liquid process train. The auger screw washes, compacts, dewaters, and conveys the dry solids material through the discharge chute to a bin for disposal.

Grinder units can be also be integrated into the compactor units, installed between the receiving hopper and the auger housing. The function of the grinder is to reduce the size of the screenings material prior to washing and compacting to enhance the washing and also help to make the discharged product look more indistinguishable. The advantage with this is that grinding of the screenings prior to compaction and rendering it indistinguishable allows for it to be disposed of as a municipal solid waste per CT DEEP guidance, as opposed to a special waste. The disadvantage with the grinder is that CDM Smith has seen installations where the grinder can be a source of increased maintenance and repair. A screenings wash compactor with grinder is shown in **Figure 6.4-6**.

The screenings compactors can be set up in various orientations. One option is to dedicate a screening compactor to each screen, mounted directly behind the screen discharge point. This provides a clean and space saving installation and avoids the need to convey the screens to a separate location for processing. The main disadvantage is that if a compactor were to be out of service for maintenance or repair, then the associated screen cannot operate. A solution to this disadvantage that has been utilized in the past is to mount the compactor on a frame with castor wheels so that it could be moved out of the way if it were out of service. When moved, a bin could be placed at the screen discharge to allow the screen to stay in operation.

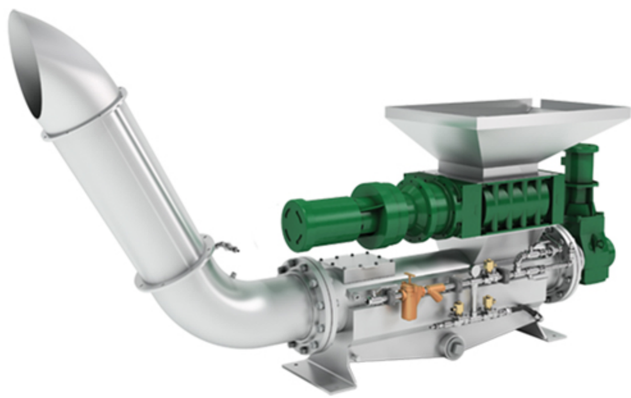


Figure 6.4-6
Screenings Wash Compactor (w/Grinder) (source: JWC Environmental)

Another orientation is to group the compactors together and locate them separate from the screens so that they can function as common units, not directly tied to a screen. A screw conveyor, conveyor belt, sluicing trough, or other means of material handling would be installed at the discharge of the screens to convey the collected screenings to the common compactors. Diversion gates would then direct the screenings to one or multiple units. The advantages with this is that multiple screens could discharge to a single compactor and an out of service compactor does not affect the operation of the screens. The disadvantage is the conveyance of the screenings from the screens to the compactors. If this is a mechanical piece of equipment such as a screen or belt, any extended interruption of its operation could affect the entire screening process. A sluicing trough is a more passive conveyance method but could be prone to plugging and also requires a large amount of flushing water, typically plant water.

Screenings Processing Assessment

The use of screenings compactors on the material removed from the coarse screens will be determined by the spacing and design approach for the coarse screens. If the coarse screens are provided with very large bar openings with the intent to just remove very large debris for pump protection (i.e. greater than 2-inch to 3-inch), then compactors are likely not warranted. There would be very little material removed that would be effectively processed by the compactors. Most of the rag, stringy material, and larger solids material would simply pass through these screens and be captured by the fine screens. The large material removed from these very coarse screens could just be deposited directly into disposal containers/roll-off containers.

If the coarse screens have a smaller opening, approximately 1-inch, to provide both basic pump protection and also a limited level of initial flow screening, then there would be a benefit to installing compactors for this process. Screens with this size opening would remove some of the rags, stringy material, and other solids material. Processing this material would reduce the water content, return organic material back to the process flow, reduce the screenings volume/weight, and also help to limit odors.

The fine screen process, regardless of the technology and ultimate size and shape of the openings, will remove large quantities of rags, stringy, and solid material. There is a very tangible benefit to washing and compacting this material to strip/wash away the organic material and return it to the process flow, to reduce the water content and weight of the material requiring disposal, and to reduce the potential for odors in the material discharged to the disposal containers. Screenings compactors are recommended for the fine screen process.

The use of grinders incorporated into the inlet hopper of a screenings compactor is viable and provides a greater level of material washing, compacting, reduction of size, reduction of organic content and potential odors, and disposal location options due to recharacterization of the material. The main concern with adding the grinder to the compactor is that the grinder can be a source of increased maintenance and repair. The use of grinders tends to vary from plant to plant depending on the material disposal options available. They will be considered further as part of this study.

6.4.3.5 Summary and Recommendations

Coarse Screening

Coarse influent screening is recommended upstream of the influent pumps at both the West Side and East Side WWTPs to provide pump protection. Based on the evaluation of alternative screens, multiple rake bar screens are recommended due their robust construction, ability to more rapidly clean the bar face due to the multiple rake setup, low headspace requirements, and lower cost when compared to equivalently sized climber style screens. Also, many manufacturers of multiple rake bar screens also manufacture fine screens, both bar screen style and perforated plate style screens and some also manufacturer center flow screens. The benefit of this is that a single vendor/manufacturer can be responsible for both sets of screens, aiding in service, maintenance, and repair. Additionally, the controls setup effort would be reduced and streamlined as the contractor would only have to coordinate with one manufacturer for both systems, each of which would have similar control panels and signals along with similar programming in the new plantwide SCADA system. Plant staff would only have to coordinate with a single entity for any screen issues.

To provide for protection of the influent pumps while also providing a modest level of material removal upstream of the remaining preliminary processes, screens with a 1-inch bar spacing are proposed. For all the various East Side and West Side WWTPs proposed flow scenarios, multiple parallel operating screens are required and recommended as shown in the influent screening options table above to address minimum and peak flows. For each scenario at each plant, a standby screen will be provided to allow for one screen to be out of service without affecting peak flow capacity.

Each screen will be installed in its own influent channel with upstream and downstream slide gates to provide isolation for maintenance, repair, or removal. The screens should be manufactured from 316 stainless steel construction due to the salt water influence in the influent wastewater and the H_2S in the influent.

Screenings washer/compactors are recommended for the coarse screen process. Multiple units should be provided, either one compactor dedicated to each screen mounted at the discharge, or multiple common compactors that can receive screenings from one or multiple screens via a common conveyor or sluice trough under the screen discharges. The final orientation of the compactors will be determined during the final design effort when the layout of the new facility is furthered advanced and defined.

Should the bar spacing of the coarse screens be revised as part of the final design phase to a much wider spacing, greater than 2-inch for example, the screenings compactor would likely no longer be beneficial. Very coarse screens with a large bar spacing would only be removing large solid debris, and not capturing the rags and other smaller solids. There would be little material that could be compacted and drained. In this scenario, the removed material would be transferred directly to a disposal container.

Fine Screening

Fine screening of all the influent flow is also recommended, either downstream of the new main influent pumping station or directly downstream of the coarse screening process. Fine screening, with ¼-inch bar spacing or ¼-inch circular or square perforated openings, is required for a number of the treatment processes considered in this facility planning study including Headcell grit removal, primary cloth filters, IFAS, membrane bioreactors, and high rate clarification. Additionally, as the technology has advanced, ¼-inch screens are becoming industry standard due to the overall benefit to downstream systems with the enhanced removal of additional solids, floatables, rags, and stringy material.

Perforated plate screens with ¼-inch openings are recommended for the fine screen process due to their high removal efficiency when compared to ¼-inch bar screens or step-style screens. This increased material removal will aid the performance of the downstream processes and help to prevent potential fouling. Fine screens with parallel bars could still allow long, thin material such as rags and stringy material to pass through them, potentially fouling downstream advanced treatment processes.

Center flow perforated screens will also be assessed further in the final design phase as the size, orientation, and layout of the fine screen process area is developed in conjunction with the overall preliminary treatment system. They have an advantage over the perforated screens in that they eliminate potential carry over, however they do require wider channels. This design decision will be easier to assess and resolve when the fine screen facility process area becomes more defined.

Screenings washer/compactors are recommended for the fine screen process. Multiple units should be provided, either one compactor dedicated to each screen mounted at the discharge, or multiple common compactors that can receive screenings from one or multiple screens via conveyor or sluice trough under the screen discharges. The final orientation of the compactors will be determined during the final design effort when the layout of the new facility is furthered advanced and defined.

6.4.4 Grit Removal

6.4.4.1 Approach

The West Side and East Side plants currently lack grit removal and instead rely on degritting the primary sludge. The issue with the sludge degritting approach is that grit is allowed to settle out in the primary tanks increasing stress and wear on the sludge removal equipment. There is also increased wear on the primary sludge pumps and piping along with increased handling of the primary sludge. In addition, CDM Smith's experience is that many communities that practice this approach have had continued difficulties in consistently removing grit from the primary sludge, negatively affecting downstream processes.

Based on the performance limitations associated with the existing primary sludge degritting operations, CDM Smith recommends that new appropriately sized grit removal facilities be installed at the two plants to remove the grit as part of the preliminary treatment process. This approach will remove grit early in the treatment process before it has the opportunity to negatively affect downstream processes, provide a means to remove and process wet weather grit loads which will continue to arrive at the plant due to the nature of the existing upstream combined conveyance system, eliminate pumping grit into the primary clarifiers and the associated primary sludge degritting problems, and eliminate grit carryover into the aeration tanks and other downstream processes.

The grit systems will be sized to treat the full combined dry and wet weather flows that are projected to be conveyed to the plants for protection of the full dry and wet weather processes. Limited information was available regarding the quantities of grit captured at the plants and no information is available about its physical properties. For preliminary planning, system manufacturers assumed grit quantities and characterizations that are typical for the New England regions and CSO wastewater flows.

The target of a new grit removal system would be to remove about 95 percent of all grit particles with a specific gravity of 2.65 that are greater than or equal to 150 microns (100 mesh) in size at peak flows. However, this design point may be refined when a grit study/characterization is performed as part of the preliminary design work to estimate influent grit quantities and gradation.

6.4.4.2 Grit Removal Technology Alternatives

There are various grit removal technologies/strategies on the market to remove grit from combined dry and wet weather wastewater flows. The following three selected technologies are discussed below taking into account capital and operating costs, space requirements, grit removal capability, ease of operation, and feasibility for use in Bridgeport.

- Vortex
- Stacked Tray
- Aerated

Vortex Grit Removal

Vortex type separators include low energy, medium energy and high-energy variations. The difference in variation is the method used to induce and maintain a rotary vortex motion within the chamber over the range of design flows for the unit. The typical turndown ratio for an individual low and medium energy induced vortex type unit is about 5:1 up to 10:1 depending on the manufacturer. The turndown ratio is the ratio of peak flow to minimum acceptable flow for the unit. For example, a unit with a turndown ratio of 5:1 could be operated from 100 percent down to 20 percent of the peak unit flow.

High-energy units rely solely on velocity to maintain the induced vortex and have limited flexibility in design flow rates. High-energy units such as cyclone separators are typically used as secondary classification devices to separate water from collected grit and are not applicable as a primary grit removal system for Bridgeport.

Low energy vortex grit collectors use a mechanical paddle in the center of the circular chamber to maintain rotary motion of the wastewater flow and the induced vortex action for grit settling. Vortex units are widely known by the proprietary name of PISTA Grit by Smith & Loveless, but other systems include the SpiraGrit system by Lakeside Equipment Corporation, VORMAX Grit Chamber by Huber Technology Inc., J+A Jeta Grit Trap by Ovivo, Vortex Grit Chamber by WesTech Engineering Inc, and others. These units have low head loss across the chambers, typically 12-in or less at peak flows and 6-in or less at design flows. Given that these systems have limits on flow turndown (10:1) that affects removal performance, multiple smaller units are required to provide adequate treatment at low and peak flows at plants that have highly variable flows.

Medium energy units rely on system flow velocity or a velocity control device instead of a mechanical paddle to maintain rotary wastewater motion over a more restrictive 4:1 turndown flow range. Headloss for these units is similar to other vortex units, less than 12-inch at peak flows. These units have a smaller footprint that the vortex units with mechanical vortex inducing equipment, and similar to the low energy systems, multiple smaller units are required to provide adequate treatment at low and peak flows give the turndown limits. A difference with these systems is that the influent to the chambers is typically through a piped connection versus a channel entry common to the other vortex systems. The GritKing by Hydro International is an example of a medium energy system.

In the vortex grit systems, the settled grit is typically collected in a lower hopper section below the main circular tank/chamber and removed with pumps suitable for handling a grit-water slurry. The pumps may be top-mounted suction-lift style, submersible style located directly in the lower hopper, or dry pit centrifugal pumps with a flooded suction if space is available to construct a below-grade pump gallery or basement. The grit is then pumped to downstream processing equipment.

Figure 6.4-7 shows a typical section through a vortex grit chamber. **Table 6.4-5** contains sample model information and planning level sizing for an 80 mgd East Side WWTP Smith & Loveless PISTA grit system and also for a 200 mgd West Side plant system for conceptual level planning.

Most vortex units require a pre-screening size no larger than ¾ inches. As part of the upgrades to the East Side WWTP, screens of this spacing, or smaller, will be likely be installed and will therefore protect a vortex style grit system.

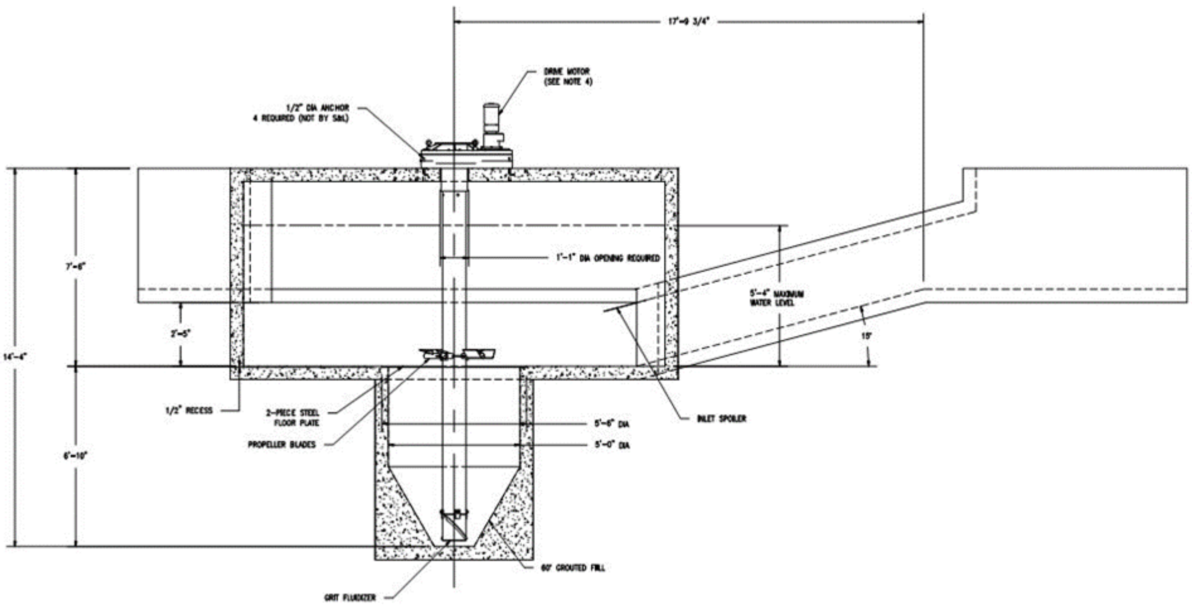


Figure 6.4-7
Typical Section Through Vortex Unit (Source: Smith & Loveless)

Table 6.4-5 PISTA Vortex Unit Data

Parameter	West Side WWTP Value	East Side WWTP Value
Total System Flow	200 mgd	80 mgd
Flow per Unit (S & L Model 20.0B 360 PISTA)	50 mgd	20 mgd
Number of Units Required for 80 mgd	4	4
Upper Chamber Diameter	20'-0"	16'-0"
Upper Chamber Depth	11'-6"	7'-6"
Lower Chamber Diameter (min)	5'-0"	5'-0"
Lower Chamber Depth (min)	8'-0"	6'-10"
Inlet/Outlet Channel Width	5'-0"	4'-0"

Because of the highly variable flows during a storm or other wet weather event at the East Side WWTP, and for operational flexibility, the design approach for a vortex grit removal system would be to use multiple units to meet the targeted removal efficiency at minimum dry weather flows and at peak wet weather flows. Each unit must have the appropriate turn down ratio to meet low nighttime flow requirements. In addition, multiple units provide flexibility to maximize redundancy and to allow for system maintenance and/or repair. The disadvantage of multiple units is that space requirements increases for the additional units, connecting piping, and workspace/clearances, and units would have to be brought online and taken offline as flows fluctuate.

The conceptual footprint for a Wet Side WWTP 200 mgd system (based on a four-unit PISTA system) including support equipment and grit processing equipment is approximately 100-ft by 120-ft. And the conceptual footprint for an East Side WWTP 80 mgd vortex grit removal system (based on four-unit PISTA system) including support equipment and grit processing equipment is approximately 80-ft by 100-ft.

Stacked Tray System

The stacked tray system (Headcell Settleable Solids Separator as manufactured by Hydro International) is a type of vortex unit that operates under differential pressure (as shown in **Figure 6.4-8**). The modular, multi-tray concentrator has a small footprint designed to fit into small concrete tanks. Flow feeds into the concentrator tangentially, establishing the vortex flow pattern. Solids settle into the boundary layer of each tray and are drawn down the center to the collection chamber. The Headcell provides fine grit removal down to 50 microns. The headloss in a 45 mgd unit is 12 inches. The Headcell requires a considerable amount of service water, however if plant service water system is replaced and upgraded as part of the project, water demand could be incorporated into the new system.

In the Headcell stacked tray systems, the settled grit is collected in a smaller hopper section below the bottom tray and removed with pumps suitable for handling a grit-water slurry. The pumps are typically dry-pit centrifugal pumps with a flooded suction located in a below-grade pump gallery or basement adjacent to the tanks. The grit is then pumped to downstream processing equipment. Two pumps should be provided per unit to provide for redundancy at each tank given the wear and repairs associated with grit slurry pumps.

The Headcell system has the advantages of efficient grit removal, a small footprint, low energy requirements, and a modular design. The disadvantages include higher capital cost and this is also a newer technology to the market with limited installations and no true “or equal,” which can make bidding a challenge or require pre-purchasing or pre-selecting the equipment.

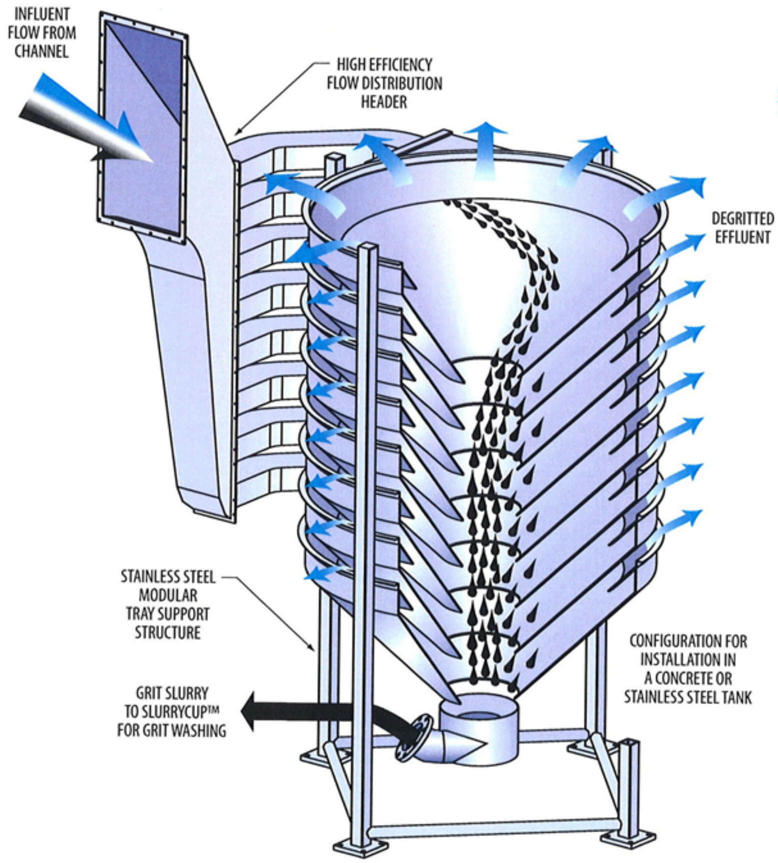


Figure 6.4-8
Headcell Concentrator Diagram (source: Hydro International brochure)

Typical design parameters are shown in **Tables 6.4-6 and 6.4-7** for 200 mgd West Side plant flow scenario and 80 mgd East Side plant flow scenario for planning purposes. The grit washing and dewatering units are typically installed paired with each Headcell Concentrator.

Table 6.4-6 Headcell Data – West Side WWTP 200 mgd

Parameter	Headcell	SlurryCup Grit Washing Units	Grit Snail Dewatering Units
Flow per Unit	40 mgd	550 gpm	4.0 cy/hr
Number of Units Required for 200 mgd	5	5	2
Number of trays per unit	12 trays @12-ft dia.		
Footprint (tankage)	18' x 18' each	5 washers, 2 dewaterer: 15' x 45'	
Height	25' (tank depth)	13'	
Headloss	12"	10.33'	-

Table 6.4-7 Headcell Data – East Side WWTP 80 mgd

Parameter	Headcell	SlurryCup Grit Washing Units	Grit Snail Dewatering Units
Flow per Unit	40 mgd	550 gpm	6.0 cy/hr
Number of Units Required for 80 mgd	2	2	1
Number of trays per unit	12 trays @12-ft dia.		
Footprint (tankage)	18' x 18' each	2 washers, 1 dewaterer: 12'x17'	
Height	25' (tank depth)	13'	
Headloss	12"	10.33'	-

As discussed with the vortex grit removal technology, multiple smaller units would be recommended to meet the variable flows and to maximize operational flexibility and redundancy. However, the disadvantage of multiple units is that space requirements increase for the additional units, connecting piping, and workspace/clearances. Space is also required for various accessory equipment associated with grit removal technologies. These include grit pumps, odor control measures, and grit treatment systems such as cyclones, grit classifiers, grit washers, and disposal containers.

The conceptual footprint for a Headcell system including support equipment and grit processing equipment is approximately 120-ft by 80-ft for a West Side plant system and approximately 40-ft by 80-ft for an East Side plant system.

Aerated Grit Removal

Aerated grit removal has a long history of use at major wastewater treatment facilities with well-established design criteria. Properly designed tanks are typically long and narrow with a depth approximately equal to the width. The aerated grit process creates a spiral rolling motion along the length of the tank through the addition of air via coarse diffusers located along one side of the chamber. The diffused air addition is used as a method of controlling particle velocities within the tanks, which in turn controls of the size of grit particle allowed to settle and be removed. Because of the spiral air flow pattern, grit particles make several passes across the sloped tank bottom thus improving the chances for removal. The diffused air also keeps the lighter organic particles in suspension, and they are discharged over the effluent weir and onto the downstream biological processes.

A typical section through an aerated grit chamber is shown in **Figure 6.4-9**.

An advantage with the aerated grit process is the volume associated with the multiple tanks. This tank volume can provide storage, equalization, and flow buffering within the overall treatment process, which enables further flexibility for varied flow and load conditions as experienced in a combined sewer system.

A disadvantage with aerated grit systems is that they tend to have a larger footprint when incorporating the influent distribution and effluent boxes/channels, a gallery adjacent to the tanks for the removal screw drives, grit slurry pumps, and air blowers, and the processing area for the grit classifiers/washers and disposal containers. Aerated grit chambers also create odors and these tanks would have to be covered and have odor control/mitigation.

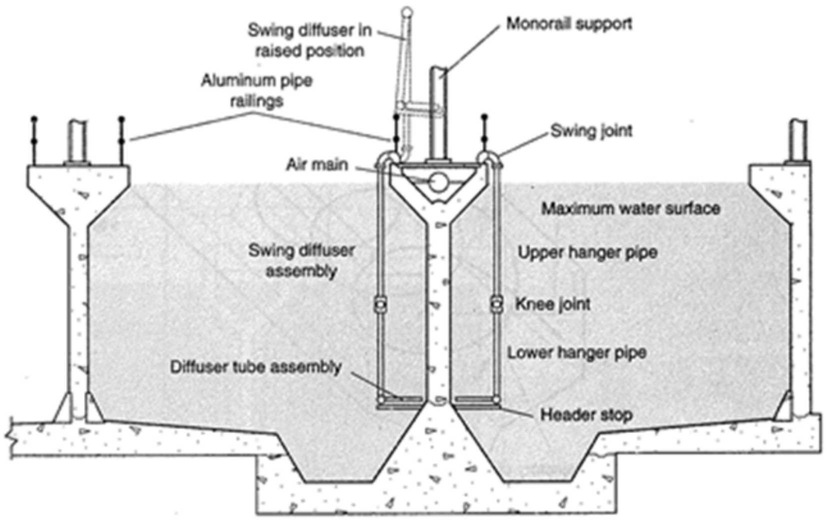


Figure 6.4-9
Typical Section Through Aerated Grit Chamber (Source: M&E 2003)

Typical aerated grit tanks process design criteria for an East Side plant and West Side plant scenario are listed in **Table 6.4-8** below along with potential dimensions and arrangement for an aerated grit system at the West Side and East Side WWTPs.

Table 6.4-8 Aerated Grit Design Criteria

Parameter	Typical Criteria	Proposed for West Side WWTP – 200 mgd	Proposed for East Side WWTP – 80 mgd
Number of Tanks	As Required - Min. of 2	3	2
Hydraulic detention time	3 to 10 minutes at peak flow	4.2 minutes	4.1 minutes
Overflow Rate for 100 Mesh	42,000 gal/day/sq ft	42,000 gal/day/sq ft	42,000 gal/day/sq ft
Side Water Depth	7 to 16 ft	17 ft	16 ft
Width (each tank)	Varies to Suit	19 ft	16 ft
Width to Depth Ratio	0.8:1 to 1:1	1.1:1	1:1
Length (each tank)	Varies to Suit	80	60 ft
Length to Width Ratio	3:1 to 8:1	4.2:1	3.75:1
Air Supply	3 to 8 cfm/ft of length	6 cfm/ft	6 cfm/ft

Historically, grit has typically been removed from aerated grit chambers using grit screws at the tank invert in conjunction with bucket elevators or in conjunction with recessed impeller centrifugal pumps in a drywell adjacent to the grit tank. A small number of plants utilize a clamshell bucket mounted on a two-way bridge crane to lift grit from the bottom of the tanks. The advantage to this is that the grit is somewhat dewatered as the clamshell is lifted out of the flow stream, however this is a more labor-intensive process requiring direct manpower to clean out the grit tanks.

The traveling bridge grit removal system is another removal technology on the market that is suited to handle the high peaks associated with wet weather flow and is suitable for use in aerated grit chambers. Submersible pumps are mounted on a bridge system that moves along the chamber in both directions and continuously removes grit collecting in the troughs at the bottom of the tanks. A typical section through an aerated grit chamber with traveling bridge pumps is shown in **Figure 6.4-10**.

The conceptual footprint for an aerated grit system including support equipment, pump and blower gallery, influent distribution box, effluent channel, and grit processing area is approximately 115-ft by 70-ft for an East Side plant system, and approximately 140-ft by 90-ft for a West Side plant system.

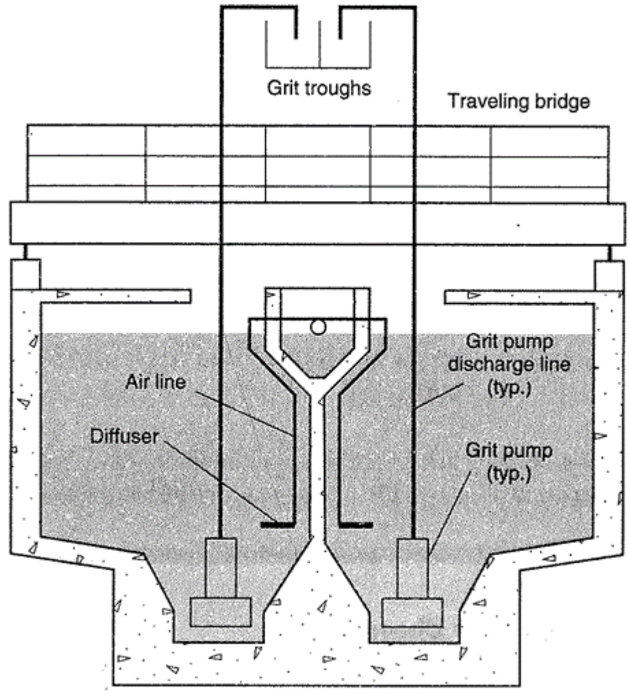


Figure 6.4-10
Typical Section Through Aerated Grit Chamber with a Traveling-Bridge-Type Grit Removal System (Source: M&E 2003)

6.4.4.3 Grit Removal Evaluation/Assessment

The following **Table 6.4-9** describes a brief description of the advantages and disadvantages of the various grit removal alternatives considered.

Table 6.4-9 Grit Technology Evaluation

Grit Technology	Advantages	Disadvantages
Vortex Type Separators	• Low headloss through the system • Low energy requirements • Modular to handle range of flows • Similar large flow installations	• Limited turndown ratios • Multiple parallel units present potential siting, arrangement, and flow splitting challenges
Headcell Settleable Solids Separator	• Compact unit footprint • Modular design • Fine grit removal (50 microns per manufacturer's claim) • Low headloss	• Exclusive technology • Requires higher service water flow • Fewer installations
Aerated Grit Tanks	• Adapts to variable flows and grit loads • Similar successful CSO installations in New England • Provides level of high-flow equalization/buffering	• Odor generation from the aeration process • Higher energy requirements • Multiple system components to maintain (grit removal screw, air blowers, grit pumps) • Larger footprint

Table 6.4-10 provides a more detailed non-economic evaluation of the grit removal processes.

While not as prominent as vortex type and aerated grit removal systems, the Headcell settleable solids separator system is a proven grit removal technology with systems of comparable size, and even larger, to the highest peak flows proposed for the West Side and East Side WWTPs (200 mgd and 80 mgd respectively) throughout North America. The largest system is a 110 mgd average flow, 368 mgd peak flow system in Calgary, AB consisting of 10 units. CDM Smith has also designed multiple Headcell systems in the U.S. The Headcell system is similar to vortex systems in that it is multi-unit design with units on/off-line depending on the plant influent flow. It is a high performing, low headloss system with a compact footprint. For these reasons, the Headcell settleable solids separator grit system is a viable option for grit removal at both the East Side and West Side WWTPs and will be considered further.

The aerated grit tank grit removal process is a decades old process that can be found at plants of all sizes throughout the state and country. While there are installations where aerated grit systems have proven to be ineffective, the reason can usually be attributed to non-ideal tanks dimensions and dimension ratios. When designed in conformance with industry recommended guidelines, aerated grit tanks can be an effective process for grit removal. The large tanks also provide a level of flow buffering/equalization when plants experience large increases in flow. The drawback to this system is its larger footprint which is greater than the vortex and Headcell systems when taking into account all the support and processing equipment/areas. However, an aerated grit system is a viable option for grit removal at both the East Side and West Side WWTPs and will be considered further.

Table 6.4-10 Grit Removal Alternative Non-Economic Evaluation

Alternative Process Description	Vortex Type Separators	Settleable Solids Separator	Aerated Grit Tanks
Non-Economic Criteria	**Smith & Loveless PISTA**	**Hydro International Headcell**	**N/A**
Success at Other Installations/ Reliability	Common process used at small and large-scale facilities to for grit removal. Successful installations throughout New England and the U.S. with sizes comparable and larger than East & West Side WWTPS.	Less process, newer technology, used at small and large-scale facilities to for grit removal. Successful installations (more limited) throughout North America with sizes comparable and larger than East & West Side WWTPs.	Common process used at small and large-scale facilities to for grit removal throughout New England and the U.S. with sizes comparable and larger than East & West Side WWTPS. Successful installations when the system tanks are sized according to recommended industry guidelines.
Site Utilization	Compact process footprint with compact tank area and inlet/outlet velocity channels. Systems for East and West Side WWTPs would vary from 2-4 circular tanks with separate grit pump gallery and at-grade grit processing/load-out area.	Smallest process footprint with smallest tank area. Systems for East and West Side WWTPs would vary from 2-5 square tanks with separate grit pump gallery and at-grade grit processing/load-out area.	Largest process footprint with largest tank area. Systems for East and West Side WWTPs would vary from 2-4 long narrow tanks with separate gallery for grit pumps, blowers, and grit removal drives and at-grade grit processing/load-out area.
Maintenance of Plant Operations	A vortex type grit system would be constructed as a completely new process in a currently open parcel of land at the site (open parcel north of the existing sludge building and chlorine contact tanks). MOPO would be included in the startup and integration of the overall new preliminary treatment facility (screening, influent pumping, grit removal).	A Headcell settleable solids separator grit system would be constructed as a completely new process in a currently open parcel of land at the site (open parcel north of the existing sludge building and chlorine contact tanks). MOPO would be included in the startup and integration of the overall new preliminary treatment facility (screening, influent pumping, grit removal).	A aerated grit system would be constructed as a completely new process in a currently open parcel of land at the site (open parcel north of the existing sludge building and chlorine contact tanks). MOPO would be included in the startup and integration of the overall new preliminary treatment facility (screening, influent pumping, grit removal).
Ease of Operations	Less complex - less equipment. Removal may be hydraulic, but likely to include mechanical mixer. Given the wide range of flows, the number of operating units will depend on plant flow and may require operator input taking tanks on/off line depending on level of system automation. Grit pump operation will be automated and flow or time based, but will require monitoring and adjustment to ensure they do not plug or run too excessively. Tank mixer will be constant when a tank is on operation. The grit washer/classifier operation will be automated but will be require routine monitoring, and is similar to the other alternatives. All equipment controlled by manufacturer supplied control panel(s).	Less complex - minimal equipment. Removal process is all hydraulic. Given the wide range of flows, the number of operating units will depend on plant flow and may require operator input taking tanks on/off line depending on level of system automation. Grit pump operation will be automated and flow or time based, but will require monitoring and adjustment to ensure they do not plug or run too excessively. The grit washer/classifier operation will be automated but will be require routine monitoring, and is similar to the other alternatives. All equipment controlled by manufacturer supplied control panel(s).	More complex - more system equipment. The major components are the grit removal screw at tank invert, the grit transfer pumps, and air blowers. These systems need to operate in conjunction with one another to optimize grit removal (appropriate air rate, screw speed and operating time, and pump operating time/frequency. Systems will be automated, but will require monitoring and adjustment. With wide range of flows, the number of operating units will depend on plant flow and may require operator input taking tanks on/off line depending on level of system automation. The grit washer/classifier operation will be automated but will be require routine monitoring, and is similar to the other alternatives. All equipment controlled by manufacturer supplied control panel(s).
Ease of Maintenance	Less complex - less equipment. Some systems are all-hydraulic with no operating equipment in the tanks, relying on flow velocities only for settling, while most systems employ mechanical mixers in the tanks to create vortex motion. The mixers will require maintenance along with the grit pumps that remove settled grit from the tanks and transfer it to the processing equipment. The grit washer/classifier maintenance is similar to the other alternatives.	Less complex - minimal equipment. The Headcell system is an all-hydraulic grit removal system with no operating equipment in the tanks. The main maintenance item is the grit pumps that remove settled grit from the tanks and transfer it to the processing equipment. The grit washer/classifier maintenance is similar to the other alternatives.	More complex - more system equipment. The major components are the grit removal screw at the tank invert, the grit transfer pumps, and the air blowers. These will require routine maintenance. The removal screw drives, blowers and pumps will be located in the gallery adjacent to the tanks and will be readily accessible. Issues with a removal screw, or plugging of the grit hopper by the pump suction, will require a tank to be taken offline. The grit washer/classifier maintenance is similar to the other alternatives.
Neighborhood Impacts	The process will likely be located at the northern end of the site property. The system will include tanks with raw wastewater. To reduce odors and the sight of wastewater, the tanks will be covered or housed within a building with odor control. Load-out containers will be contained in building.	The process will likely be located at the northern end of the site property. The system will include tanks with raw wastewater. To reduce odors and the sight of wastewater, the tanks will be covered or housed within a building with odor control. Load-out containers will be contained in building.	The process will likely be located at the northern end of the site property. The system will include tanks with raw wastewater. To reduce odors and the sight of wastewater, the tanks will be covered or housed within a building with odor control. Load-out containers will be contained in building.
Ability to Phase Implementation	Multi-tank system that could be expanded if flows were increased. Provisions would be included in initial design to accommodate expansion.	Multi-tank system that could be expanded if flows were increased. Provisions would be included in initial design to accommodate expansion.	Multi-tank system that could be expanded if flows were increased. Provisions would be included in initial design to accommodate expansion.
Sludge Impacts	Will improve sludge handling by removing grit upstream of primary and secondary setting and reducing the grit content in the generated sludge.	Will improve sludge handling by removing grit upstream of primary and secondary setting and reducing the grit content in the generated sludge. An efficient Headcell system would be expected to remove more grit than the aerated grit process.	Will improve sludge handling by removing grit upstream of primary and secondary setting and reducing the grit content in the generated sludge.
Energy Efficiency	Low energy requirement - tank mixer, grit pumps, grit washers/classifiers.	Low energy requirement - grit pumps, grit washers/classifiers.	Highest energy requirement due to the full-time operation of one or multiple air blowers along with grit pumps and removal screws.
Chemical Handling/Hazards	N/A - system does not require any chemicals for operation.	N/A - system does not require any chemicals for operation.	N/A - system does not require any chemicals for operation.

This page intentionally left blank.

6.4.4.4 Grit Treatment

Removed grit can contain a large amount of attached organic material and conveyance water. If removed from the grit system and put immediately into a disposal container, the organic material can become a major source of odors and an attraction for insects and rodents if not immediately removed from the site. To address these issues, the grit can be passed through an additional treatment process to remove organic material that settled with the grit and reintroduce it to plant flow for full treatment, and also allow for excess water to be removed, partially drying the grit. Grit treatment alternatives are generally limited to grit classifiers and grit washers, however various manufacturers have unique variations on this type of equipment with respect to design and operation.

The grit treatment system should be designed such that it has capacity for the maximum flow of grit/water that is removed from the system. The system should include multiple units and interconnections to allow for system flexibility and for the system to be maintained and repaired. Processed grit should be discharged to a roll-off container or other suitable container for offsite disposal.

Grit Classifiers

Grit classifiers typically contain two main components, a sedimentation tank and an inclined removal screw. Flow enters the tank either from above or from a flanged connection along the side of the tank. The agitation caused by the flow entering the tank causes organic material to separate from the grit. In pumped grit slurry applications with higher flow rates, cyclones are typically installed on the influent to the classifier to help remove excess water and organics being conveyed with the grit. Without the cyclone, the classifier would need to be larger to provide a reasonable detention time in the tank for grit to settle out.

The heavy grit settles to the bottom of the sedimentation tank where the lower end of the inclined removal screw is located. At the same time, the buoyant organic material floats to the surface and exits the tank over a weir or baffle along with the excess flow. The drain from the weir or baffle is typically connected back to the plant influent flow or to the grit removal process. The advantage with this is that any grit that passed through the classifier has another opportunity of being recaptured by the process. The disadvantage is that the organics are also returned back to the grit process, where they can potentially accumulate, as opposed to being discharged further downstream closer to the primary or secondary systems where they can be of beneficial use and removed more effectively. A grit classifier is shown in Figure **6.4-11**.

The removal screw operates intermittently to remove grit that has settled to the bottom of the tank. The screw and trough typically extend a few feet above the water in the tank, allowing the grit to dewater as it is moved along by the screw. Spray bars can be mounted along the dewatering section to help remove additional organic material, but this can also lead to excess water content in the cleaned grit. The grit is then discharge at the high point of the removal screw. Although cleaner than when the grit was removed from the tank, the grit can still contain some raw organic matter. Because of this, grit that passes through a grit classifier cannot be reused as granular material and still needs to be disposed of in a landfill.

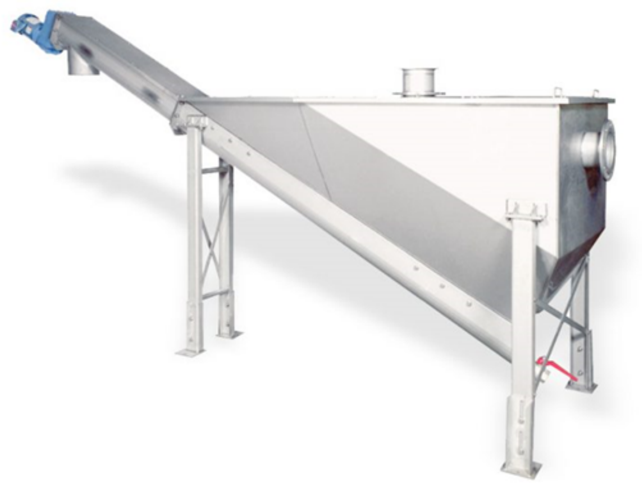


Figure 6.4-11
Grit Classifier (source: Vulcan Industries)

Grit classifiers are available in various sizes to accommodate a range of influent flow, from 150 gallons per minute (gpm) to over 550 gpm.

Grit Washers

Grit washers are similar in construction to grit classifiers in that they contain two main components: a sedimentation tank and an inclined removal screw. A grit slurry flow is conveyed to the tank where the heavy grit particles settle out and the organic material is returned to the process flow. The grit removal screw then rotates to remove the settled grit and dewater it prior to discharge. A grit washer is shown in Figure **6.4-12**.

The main difference with the grit washers is that they agitate the grit slurry considerably more than the grit classifiers prior to settling. With diffused air or mechanical mixers, a fluidized bed of grit is kept in motion in the tank. This agitation and shearing action allows much more of the organic matter to be stripped from the grit particles. The grit settles to the bottom of the tank and a wash water connection typically flushes the organic material to the top of the tank and over an overflow weir to the tank drain for introduction back into the process stream.

While suitable for continuous flow from a grit pump, the washer can also be operated in a batch mode. After a grit pump is operated for a predetermined length of time to bring grit to the tank, the washer can then be run without additional flow entering the unit. The washing action can be completed, the organic material flushed out, and the clean grit removed and dewatered with the screw.

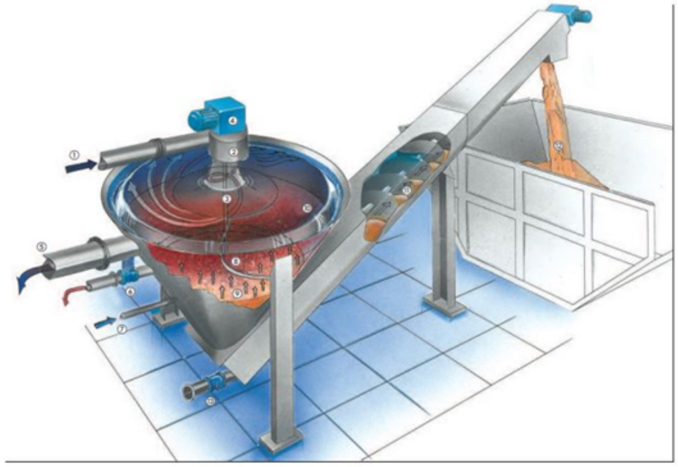


Figure 6.4-12
Grit Washer (source: Huber Technology)

The main advantage with the grit washer is that it produces a clean grit that is virtually free of organic matter - typically 3 percent or less. By stripping the buoyant organic matter from the grit, the washers are also able to settle out more of the finer grit particles. The potential then exists for reuse of the grit as aggregate material. The discharged grit also tends to be less odorous due to the lower organic content.

One disadvantage to the grit washer is that they are more expensive than the grit classifiers. There are also fewer manufacturers of grit washers compared to the classifiers, potentially reducing competitive pricing. However, washers are becoming more common in the wastewater industry due to the cleaner final product, the reduced odor generation, and the potential reuse of the material.

6.4.4.5 Grit Treatment Evaluation

Both the grit classifiers and grit washers are applicable treatment solutions for the grit removed from the influent flow. The grit classifiers are more straightforward pieces of equipment in that they provide a more passive and less intensive cleaning of the grit with minimal moving parts and system. The grit essentially settles out in the tank and the inclined screen removes the settled grit. They are lower in price, have a higher flow capacity per unit (when cyclones are not incorporated), and require simpler controls and operation. They are, however, not as effective as removing material attached to the grit particles, resulting in a grit product that tends to be more odorous, higher volume, and less dry.

Grit washers are a more sophisticated piece of grit processing equipment compared to the classifier units. While designs vary amongst manufacturers, they generally include mixers or diffused air in the tanks to provide agitation and enhanced cleaning and a wash-water system with actuated valve(s) to flush the organic material from the tank. These additional components result in a more complicated system, although they are usually integrated into the manufacturer's control. However, the washers do typically require more setup and usage time to fully optimize the washing and backwashing systems. The result of this is cleaning, drier grit with less organic material attached to the particles, less odor, and less volume to dispose of.

Both of these technologies are viable for grit processing at either plant and will be further considered.

6.4.4.6 Summary and Recommendation

All three of the grit removal processes presented in this section would be viable alternatives at the West Side and East Side WWTPs, each with its own distinct advantages and disadvantages. The vortex systems are a low headloss, low energy system with successful installations similar in size to those proposed for West Side and East Side WWTPs. But the systems have somewhat limited turndown to address minimum flows and the circular tanks and inlet/outlet channel requirements can present siting challenges.

The Hydro International Headcell system is a compact, high performing, low headloss and low energy system will installations in North America of comparable size to the majority of the West Side plant and East Side plant proposed sizes. However, there are limited installations comparable to the largest 180 mgd and 200 mgd West Side plant flow scenarios. There are no other systems similar to the Headcell system that could be equal bidders. Use of this system may require pre-selection or pre-negotiation of the equipment during the initial phases of design.

The aerated grit process is an older technology that, when sized according to industry guidelines, does operate effectively. The larger tanks can also provide a small level of flow buffering/equalization during times of significant increases in influent flow rate. However, the aerated grit system has the largest footprint of the three systems discussed in this study, a disadvantage for both if the constricted East Side and West Side plant sites. It also is a more complex system with more operating equipment (removal screws, air blowers).

While all three of the grit systems would be viable for either the East Side or West Side WWTPs, the Headcell settleable solids separator is recommended for the implementation at both plants for the following aspects:

- Smallest footprint
- All hydraulic system with minimal mechanical equipment
- Low energy system
- Increased grit removal performance, fine grit removal (50 microns per manufacturer)
- Modular design
- Ease of operation
- Low headloss

Grit Processing

Grit from the removal process will be processed prior to discharge to reduce water content, return organics to process stream, and reduce odors associated with the discharged product. Both the grit classifier and the grit washers will provide this treatment, with the grit washer providing cleaner and dryer grit with less organic content, but carrying higher equipment costs and more complex controls, setup, and optimization. The grit washer is the recommended alternative for both the East Side and West Side WWTPs. With their higher performance and enhanced cleaning, they will return more organics to the process stream, remove more organic from the discharged grit, reduce the volume of the grit to be disposed of, provided a drier material, and provide a discharged material with reduced odors. At the West Side plant in particular, the reduced odors will be advantageous given that the new grit facility will likely be located at the northern edge of the property, in close proximity to residential neighborhood. Any ability to reduce odor sources is beneficial to the operation of the plant.

6.4.5 Septage Receiving

Septage can be a significant source of solid and organic loadings at WWTPs depending on the volume received in comparison with the total plant flow. Septage received at municipal WWTPs should therefore generally be treated before being incorporated into either liquid or solids stream unit processes, per TR-16 guidelines. As discussed in Section 4, the West Side WWTP does not have special facilities for receiving septage waste.

Package septage acceptance plants are commonly available which include debris and inorganic solids removal, fine screening, and material washing, dewatering, and compaction prior to discharge of the organic material to the downstream processes. Rock traps can also be included for protection of the septage plant's integral operating components. Package systems can be unreliable, prone to failure and require extensive maintenance. The alternative to a package septage acceptance plant would be to introduce septage at the head of the plant for co-treatment with the main process flow provided that the relative volume of septage received would not cause upsets in the process treatment train.

Accomplishing flow measurement on incoming septage can be a challenge due to variances in offloading operations from the trucks, piping that may not flow full or at consistent velocities, variances in the makeup and consistency of the septage flow, etc. Many large facilities therefore bill septage haulers by truckload, sometimes dependent upon truck size. Septage receiving provisions and methodologies will be further evaluated during preliminary design.

The remainder of this facilities plan assumes that septage will continue to be accepted at the West Side WWTP. However, further evaluation to determine whether the West Plant WWTP should continue to accept septage, or whether septage deliveries should be moved to the East Side WWTP should be considered during preliminary design. The West Side WWTP site is constrained and will become even more constrained should the plant's capacity be increased to accept higher peak flows. The circulation of traffic around the future West Side WWTP will need to be able to accommodate septage truck traffic.

The West Side WWTP's headworks facilities are located along the northern edge of the property, which is directly adjacent to a residential housing complex. The East Side WWTP is located within an industrial area, with no immediate neighboring residences. If septage receiving is moved to the East Side WWTP, the neighborhood adjacent to the West Side WWTP could benefit from reduced truck traffic and potential odors associated with septage receipt.

Potential septage impacts to the East Side plant would need to be further evaluated using process modeling during preliminary design to assess whether this operational change would be feasible.

6.5 Primary Treatment and High Flow Management

6.5.1 Introduction

As presented, the existing primary treatment systems at the West Side and East Side WWTPs consist of traditional settling tanks which are undersized for all design and observed flow conditions and at the end of their useful life.

In addition, the combined sewer system can deliver more flow to the West Side and East Side WWTPs during wet weather events, than the capacity of the secondary system and the capacity of the influent pumping stations. When the secondary system capacity is exceeded, primary effluent is discharged to the receiving water after disinfection, when the influent pumping capacity is exceeded the influent gates are closed and the collection system is surcharged resulting in CSOs.

As such, this section assesses alternative primary treatment and high flow management treatment technologies that could be employed to upgrade and expand the primary treatment system to improve plant performance and effluent quality. **Table 6.5-1** presents the flow capacities analyzed.

Table 6.5-1 Primary Treatment/High Flow Management Capacity Analyzed

	Plant Capacity (mgd)	Secondary Capacity (mgd)	Wet Weather Bypass (mgd)
West Side WWTP			
	90	58	32
	140	58	82
	180	58	122
	200	58	144
East Side WWTP			
	40	24	16
	80	24	56

The objectives of primary and wet weather treatment are similar, so the technologies can be used in both applications. One approach to high flow management is to pass full plant flow through one large dual-use primary/wet weather treatment facility at each plant. Another approach is to employ separate dry and wet weather treatment trains with two different treatment technologies. In this case, one technology would be designed to provide primary treatment for flow up to the secondary treatment system capacity, with flow in excess of this capacity, up to the plant capacity, treated with a separate wet weather facility.

The dual-use approach is beneficial because systems do not have to be brought online and shut down, before and after a storm event, and the equipment does not sit idle for the majority of time. However, the separate dry/wet trains approach offers greater flexibility with the overall footprint, and the treatment and operation may be more reliable with separate trains dedicated to each condition. Both approaches are assessed herein. The treatment technologies evaluated are as follows:

- Traditional rectangular primary settling tanks
- Traditional rectangular primary settling tanks with chemically enhanced primary treatment (CEPT)
- Primary Filtration
- High Rate Clarification

Because of site constraints, circular primary clarifiers were not assessed.

6.5.2 Traditional Primary Settling Tanks

One option for improving the primary treatment system is to rebuild and expand the traditional rectangular primary settling tanks appropriately sized for design year flows and loads. New concrete tanks would be constructed with new non-metallic chain and flight sludge collection mechanisms.

The TR-16 recommended surface overflow rates (SOR) for average daily flow and peak hourly flow are 1,200 and 3,000 gpd/sf, respectively. The TR-16 recommended minimum length to width ratio and tank depth were also used to size new primary clarifiers. The dimensions of the current primary tanks at both plants are acceptable according to the TR-16 recommendation. For this analysis CDM Smith has assumed tank dimensions of 136-ft by 36-ft by 10.7-ft deep for each primary clarifier at the West Side WWTP, and 99.5-ft by 26-ft by 10.7-ft deep for each primary clarifier at the East Side WWTP. It is also assumed that all tanks would be operational under peak flow scenarios. The number of new primary settling tanks required for to pass each peak design flow option and approximate footprint of the potential facilities are shown in **Table 6.5-2**.

Table 6.5-2 Traditional Primary Settling Tanks Required and Approximate Footprint

Peak Flow	Number of Tanks Required	Approximate Footprint
▪ West Side WWTP		
▪ 58 mgd	6	160-ft x 220-ft
▪ 90 mgd	9	160-ft x 330-ft
▪ 140 mgd	11	160-ft x 410-ft
▪ 180 mgd	14	160-ft x 520-ft
▪ 200 mgd	15	160-ft x 560-ft
▪ East Side WWTP		
▪ 24 mgd	3*	130-ft x 90-ft
▪ 40 mgd	6	130-ft x 160-ft
▪ 80 mgd	12	130-ft x 320-ft

*Rehabilitate existing tanks

Because of the age and condition of the existing tanks, the hydraulic grade line, and maintenance of plant operation considerations, all design flow options for the West Side WWTP assume the construction of all new concrete tanks. At the East Side WWTP, the current primary settling tanks are sufficiently sized for the 24 mgd peak flow option, and the existing tanks could be maintained and rehabilitated. In this case, influent exceeding 24 mgd would be treated through a dedicated wet weather train.

As presented in Section 7, one option for the West Side plant is to repurpose the existing final settling tanks as primary settling tanks for wet weather treatment. This is only a viable option if membrane filtration is implemented in lieu of secondary clarification.

The advantages and disadvantages of traditional primary settling tanks are presented in **Table 6.5-3.**

Table 6.5-3 Traditional Primary Settling Tanks Advantages and Disadvantages

Advantages	Disadvantages
▪ Proven Technology with well-established design criteria ▪ Similar to existing system ▪ Ease of operation and maintenance	▪ Large footprint

6.5.3 Primary Settling Tanks with Chemically-Enhanced Primary Treatment (CEPT)

Another somewhat conventional option for primary treatment is chemically-enhanced primary treatment (CEPT). Existing or new primary sedimentation tanks can be retrofitted to accommodate chemical injection to enhance primary clarification either year-round or only during high flow events. CEPT involves the application of metal salts and/or polymers in primary settling tanks to aid in the formation of flocs via coagulation and flocculation. As the particles form, their specific gravity increases which leads to higher rates of settling. The faster particle settling rates allow for increased SORs and primary treatment capacities, with increased five-day biochemical oxygen demand (BOD_5) and total suspended solids (TSS) removal rates, reducing the overall footprint of the system. There is no TR-16-recommended overflow rate for average and peak flow using CEPT. Pilot and/or jar testing is recommended by TR-16 to develop design criteria, including target SOR. The CEPT peak hour SOR may increase to nearly double the recommended peak hour SOR for traditional settling.

The advantages and disadvantages of CEPT are presented in **Table 6.5-4.**

Table 6.5-4 CEPT Advantages and Disadvantages

Advantages	Disadvantages
▪ Smaller footprint than traditional primary settling ▪ Proven Technology, but design criteria is not well established ▪ Relative ease of operation and maintenance when not in CEPT mode	▪ Increases sludge production and therefore cost to manage sludge ▪ CEPT sludge more difficult to thicken and dewater ▪ Requires chemical delivery, storage and feed facilities ▪ CEPT may impact biological nitrogen removal ▪ Intermittent use complicates operations ▪ Overdosing of chemical and polymer could adversely impact receiving waters

6.5.4 Primary Filtration

The cloth media disc filter is a filtration system that uses pile cloth media in a disk configuration to filter primary solids without the use of chemicals. The AquaPrime® cloth filter by Aqua-Aerobic Systems, Inc. was designed and is being marketed to serve as a replacement for primary clarifiers. Although the technology is new for use in primary treatment, cloth disk filters have been employed for tertiary filtration since the 1990s. A schematic of the AquaPrime® cloth media filter is presented in **Figure 6.5-1**. Inlet water enters the tank and completely submerges the cloth media. Liquid passes through the media by gravity and enters the internal portion of the disk where it is discharged through the center shaft. The suspended solids are captured on the outside surface of the cloth disk filters, and as the solids accumulate, the water level in the tank rises due to increased pressure drop across the filter. When the water level reaches a predetermined level, a backwash cycle is initiated. The disks rotate during the backwash cycle with two being cleaned at a time; filtration is not interrupted during the cleaning. The backwash water is pulled from the filtered effluent and is directed to solids processing. Solids that settle to the bottom of the tank are periodically pumped out of the tanks. Floatable scum collects on the water surface and as the water level increases it is removed by flowing over the scum removal weir.

AquaPrime requires a minimum of ¼-inch (or finer) screening in preliminary treatment upstream of the filters. Each filter unit can contain as many as 24 disks with a filter surface area of 108 sf/disk. During dry weather conditions, the design overflow rate is 4.0 gpd/sf. Typical rates of TSS and BOD_5 removal are 80% and 40-50% respectively. During wet weather events, the allowable overflow rate increases to 6.5 gpd/sf. The assumption is that during wet weather events, the system will be treating dilute influent with lower solids concentrations and less efficient solids removal is acceptable. The number of filter units required and the approximate footprint of the proposed facilities for each design flow option are presented in **Table 6.5-5**.

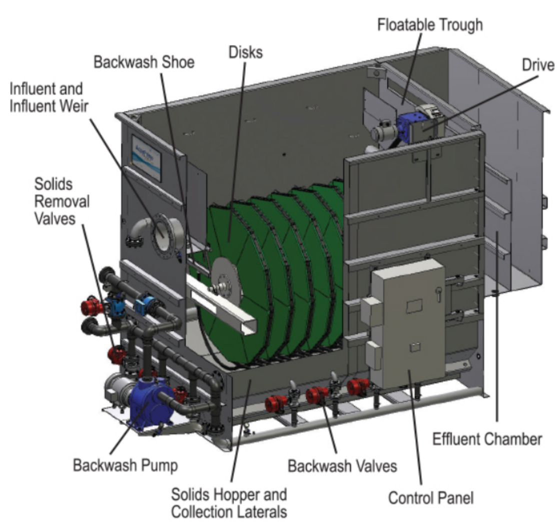


Figure 6.5-1
Schematic of AquaPrime Cloth Media Filter

Table 6.5-5 AquaPrime Primary Cloth Filter Units Required and Approximate Footprint

Peak Flow	Number of Units	Approximate Footprint
▪ West Side WWTP		
▪ 58 mgd	7*	70-ft x 195-ft
▪ 90 mgd	7	70-ft x 195-ft
▪ 140 mgd	8	140-ft x 110-ft
▪ 180 mgd	10	140-ft x 135-ft
▪ 200 mgd	11	140-ft x 160-ft
▪ East Side WWTP		
▪ 24 mgd	3*	70-ft x 95-ft
▪ 40 mgd	4	70-ft x 120-ft
▪ 80 mgd	5	70-ft x 145-ft

*1 redundant filter. Other alternatives assume all units operational at peak flow.

The advantages and disadvantages of Primary Filtration are presented in **Table 6.5-6**.

Table 6.5-6 Primary Filtration Advantages and Disadvantages

Advantages	Disadvantages
▪ Significantly smaller footprint as compared to traditional primary settling ▪ Improved BOD_5 and TSS removal as compared to traditional primary settling ▪ Reduces load to secondary treatment system ▪ Improved primary effluent quality when bypassing	▪ Limited experience for primary treatment ▪ Increases primary sludge production ▪ Cloth requires replacement every 4-7 years

6.5.5 High Rate Clarification

Ballasted flocculation, or high rate clarification (HRC), is a physical-chemical sedimentation process that uses coagulants, flocculants and media to improve floc bridging of suspended solids and settling properties. Actiflo® by Kruger is an example of a commercially available high-rate clarification system. As microfloc particles form, their specific gravity increases, thus improving the settling mechanism. The objective of the process is to increase specific gravities to values greater than two which allow the particles to settle in a shorter amount of time. The fast floc formation and decreased settling time required results in clarification rates that are up to ten times faster than those normally observed in traditional clarification processes

Chemicals used in high rate clarification units consist of coagulants such as ferric chloride or aluminum sulfate and flocculants, such as anionic polymers. Ballast materials may consist of microcarriers, such as sand particles, or sludge. The chemical addition and use of a ballast material reduce the coagulation-sedimentation time and allow the units to operate at high overflow rates while achieving increased pollutant removal. Additionally, the high overflow rates result in more compact footprints. **Table 6.5-7** presents a comparison of HRC to traditional primary settling.

Table 6.5-7 Comparison of HRC and Traditional Primary Settling Performance Parameters

Parameter	Traditional Settling	High Rate Clarification
▪ Overflow Rates	800-3,000 gpd/sf	28,880-86,400 gpd/sf
▪ Effluent Turbidity	<3 NTU	<0.5 NTU
▪ TSS Removal	50-70%	80-94%
▪ BOD_5 Removal	25-40%	48-75%
▪ Total Retention Time	1.5-2.5 hrs	4-7 min
▪ Chemical Use	None	Coagulants, flocculants and a microcarrier or sludge

The HRC train would consist of a coagulation tank, maturation tank, and sedimentation tank. Ferric chloride would be added as the screened and degritted flow enters the coagulation tank, in the flocculation tank polymer and microsand are added to enhance particle size and settling characteristics. The coagulated/flocculated/ballasted wastewater then enters the settling tank where rapid settling occurs with tube settlers. Sludge is collected and pumped through a hydrocyclone where the microsand is recovered and returned to the process. Sludge would be wasted to the gravity thickeners. A schematic of the Actiflo® high-rate clarification system is presented in **Figure 6.5-2**.

High-rate clarification offers highly efficient solids removal in a compact footprint and is appealing for high flow options. There are several comparably-sized facilities using Actiflo® for both dual use primary/wet weather treatment and as a primary treatment alternative. The units are capable of rapid startup and sitting idle for periods of time, ideal for wet weather treatment. However, the use of polymers in primary treatment alternatives is not a preferred solution for the WPCA. This additional factor will be considered during the evaluation process.

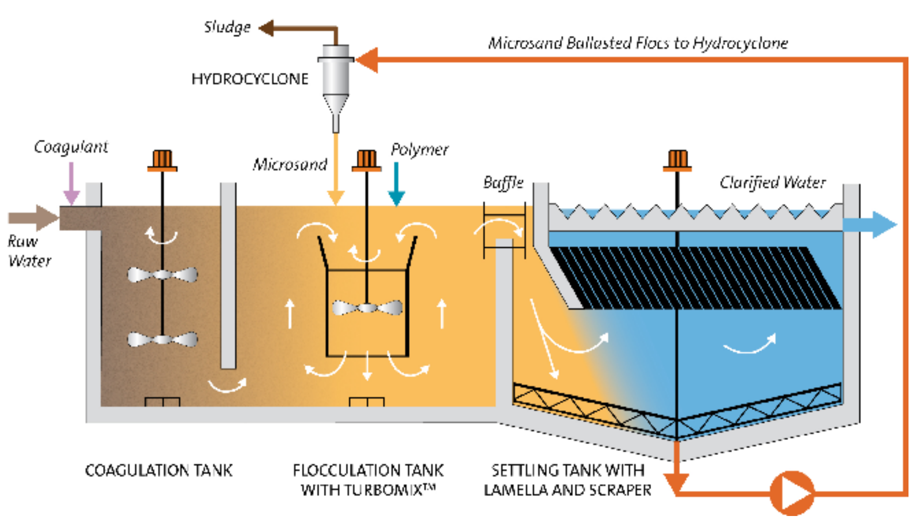


Figure 6.5-2
Schematic of Actiflo Ballasted Flocculation Process

The advantages and disadvantages of HRC are presented in **Table 6.5-8**.

Table 6.5-8 High Rate Clarification Advantages and Disadvantages

Advantages	Disadvantages
▪ Significantly smaller footprint as compared to traditional primary settling ▪ Experience at similarly sized facilities ▪ Improved BOD_5 and TSS removal as compare to traditional primary settling ▪ Reduces load to secondary treatment system ▪ Improved primary effluent quality when bypassing	▪ Increases primary sludge production ▪ More mechanically intensive than traditional primary settling ▪ Requires delivery, storage and handling of chemicals and polymers ▪ Thin sludge must be managed

6.5.6 Summary and Recommendation

The sites at the West Side and East Side WWTPs are space-limited. In addition, in order to maintain plant operations during construction, at the West Side plant, constructing an entirely new primary treatment system is advantageous. Additionally, the existing system is undersized for even the current flows. At the East Side plant, the existing traditional primary settling tanks may be retained and rehabilitated in some instances, such as supplemented with CEPT as necessary, but constructing an entirely new primary treatment system is advantageous as well.

Traditional rectangular primary settling tanks will be assessed for the 90 and 40 mgd at the West Side and East Side WWTPs, respectively. For higher flow rates, traditional primary settling will not be considered as they are size prohibitive given the restricted space available at both plants.

CEPT will not be considered at the West Side WWTP for any of the peak flow rates under consideration. For 90 mgd, new tankage would still be required for CEPT and it would be desirable to implement traditional primary settling over CEPT and eliminate the need for chemical usage. Even though CEPT utilizes higher SORs and has a reduced footprint when compared with traditional primary settling, CEPT facilities for 140 to 200 mgd peak flows would still be large structures that could not be accommodated without significant acquisition of land. At the East Side plant, CEPT will be considered for the 40 mgd peak flow scenario as the existing primary tanks would be adequately sized for the average and peak flows and could be reused. CEPT will not be considered for the 80 mgd flow scenario as the size of the required tankage is site prohibitive. In addition to the size concerns with the larger flow CEPT systems, another undesirable aspect of the system is the metal salt and polymer chemical requirement and its potential effect on sludge composition and dewatering, effect on biological nitrogen removal, and potential adverse impacts to receiving waters in the event of chemical and polymer overdosing.

Due to the removal performance and compact nature of both the cloth media filters and high rate clarification systems, these alternatives are appealing for high design flow options, and will continue to be evaluated further for all dry weather, wet weather, and total combined flow options.

In addition, hybrid alternatives and combinations of technologies will be retained for consideration.

6.6 Secondary Treatment and Nitrogen Removal

This section describes five alternatives evaluated for upgrading the biological processes at the West Side and East Side WWTPs. These alternatives include:

- Alternative Suspended Growth Activated Sludge Configuration
  - Four-Stage Bardenpho
- Integrated Activated Sludge Processes
  - Integrated Fixed Film Activated Sludge (IFAS)
- Membrane Bioreactors (MBRs)
- Membrane Aerated Biofilm Reactors (MABRs)
- Add-on Nitrogen Removal Processes
  - Downflow Denitrification Filters
  - Upflow Denitrification Filters

6.6.1 Design Criteria

6.6.1.1 Treatment Objectives

The biological process currently enables the West Side and East Side WWTPs to frequently comply with its current NPDES secondary treatment permit limits for BOD_5 and TSS. However, the existing process at the West Plant is unable to consistently achieve the annual nitrogen discharge limit established by the General Permit for Nitrogen Discharges. This permit establishes the West Side WWTP's limit at 1,041 pounds per day of Total Nitrogen (TN) on an annual average basis. The East Side WWTP has been able to outperform its permit limited of 362 pounds per day of TN on an annual average basis. Despite the good performance at the East Side plant over the last few years, the process is not considered to be reliable year-round under future flow and load conditions.

The treatment objectives of the secondary processes analyzed herein shall be capable of achieving the effluent National Pollutant Discharge Elimination System (NPDES) BOD_5 and TSS limits as well as the effluent TN limit at WWTP's projected design capacities, established in Section 5. The secondary processes shall also be capable of achieving effluent NPDES BOD_5 and TSS limits at the WWTPs' permitted capacities of 30 mgd at the West Side WWTP and 10 mgd at the East Side WWTP. The original activated sludge processes were designed to treat a maximum daily flow of 58 mgd at the West Side WWTP, and 24 mgd at the East Side WWTP to achieve conventional secondary treatment standards. These secondary treatment capacities will be maintained, consistent with historic operation and permit requirements. Wet weather flows in excess of 58 mgd and 24 mgd would continue to be diverted from primary treatment to disinfection.

To avoid over-sizing the biological systems for low level nitrogen removal, the West Side WWTP's activated sludge process shall be designed according to treatment objectives of two different flow and loading conditions.

- **Condition A: BNR + Conventional Treatment**: the secondary system will be designed to achieve effluent NPDES limits (e.g. BOD_5 and TSS) in addition to the effluent TN load under all flow and load conditions associated with the WWTP's projected 25.8 mgd design capacity (as discussed in Section 5).

- **Condition B: Conventional Treatment**: the secondary system will be designed to achieve effluent NPDES limits (e.g. BOD_5 and TSS) under all flow and load conditions associated with the WWTP's permitted flow capacity of 30 mgd. The secondary system may not be able to achieve the effluent nitrogen permit limits under all these flow and load conditions.

The design criteria for each of these conditions is presented in **Table 6.6-1**.

Following a similar design approach, East Side plant design conditions are as follows:

- **Condition A: BNR + Conventional Treatment**: the secondary system will be designed to achieve effluent NPDES limits (e.g. BOD_5 and TSS) in addition to the effluent TN load under all flow and load conditions associated with the WWTP's projected 6.4 design capacity.

Table 6.6-1 West Side WWTP Design Primary Effluent Flows and Loads

Parameter	Primary Effluent Loads	
	Condition A: BNR + Conventional Treatment	Condition B: Conventional Treatment (only)
Average Day Flow, mgd	**25.8 mgd**	**30.0 mgd**
BOD_5, lbs/day	27,000	31,000
TSS, lbs/day	24,000	27,000
TKN, lbs/day	5,200	6,000
Maximum Day, mgd	**49.3 mgd**	**58.0 mgd**
BOD_5, lbs/day	53,000	61,000
TSS, lbs/day	60,000	69,000
TKN, lbs/day	9,000	10,000
Maximum Month, mgd	**34.6 mgd**	**40.2 mgd**
BOD_5, lbs/day	40,000	46,000
TSS, lbs/day	45,000	52,000
TKN, lbs/day	7,400	8,600
Minimum Day, mgd	**17.1 mgd**	**19.9 mgd**
BOD_5, lbs/day	12,000	13,000
TSS, lbs/day	7,100	8,200
TKN, lbs/day	2,800	3,300

- **Condition B: Conventional Treatment**: the secondary system will be designed to achieve effluent NPDES limits (e.g. BOD_5 and TSS) under all flow and load conditions associated with the WWTP's permitted flow capacity of 24 mgd. The secondary system may not be able to achieve the effluent nitrogen permit limits under all these flow and load conditions.

The design criteria for each of these conditions is presented in **Table 6.6-2**.

Table 6.6-2 East Side WWTP Design Primary Effluent Flows and Loads

Parameter	Primary Effluent Loads	
	Condition A: BNR + Conventional Treatment	Condition B: Conventional Treatment (only)
Average Day Flow, mgd	**6.4 mgd**	**10 mgd**
BOD_5, lbs/day	4,900	7,700
TSS, lbs/day	3,100	4,800
TKN, lbs/day	1,300	2,000
Maximum Day, mgd	**14.1 mgd**	**24.0 mgd**
BOD_5, lbs/day	9,300	14,500
TSS, lbs/day	6,300	9,800
TKN, lbs/day	2,400	3,700
Maximum Month, mgd	**9.3 mgd**	**15.6 mgd**
BOD_5, lbs/day	6,700	10,500
TSS, lbs/day	5,600	8,800
TKN, lbs/day	1,800	2,700
Minimum Day, mgd	**3.7 mgd**	**5.8 mgd**
BOD_5, lbs/day	2,600	4,000
TSS, lbs/day	1,200	1,900
TKN, lbs/day	800	1,300

6.6.1.2 Wastewater Temperature Variation and Aerobic Solids Retention Time (SRT)

Biological treatment processes, and particularly nutrient removal processes, are sensitive to the temperature of the influent wastewater, and are least efficient under cold conditions, prevalent in the winter months or due to snow melt in a combined system. Because the WWTPs participate in the CT DEEP nitrogen trading permit, it is necessary to establish the range of wastewater temperature expected. The West Side plant's historic data shows that the minimum month (30-day) temperature experienced annually is about 10°C, occurring in February. The East Side plant's historic data shows that the minimum month (30-day) temperature experienced annually is about 11°C, also occurring in February. Because the NPDES Permit is based on a 12-month rolling average, it should also be noted that the average monthly wastewater temperature during the warmer months (June-October) is about 20°C at both WWTPs.

To reliably achieve the annual effluent TN loading limit, the biological processes needs to be able to nitrify (convert ammonia to nitrite and then nitrate) year-round. Sustaining nitrification year-round in an activated sludge process requires the biological process to maintain an aerobic solids retention time (SRT) – considering the aerobic or aerated volume only – sufficiently long to prevent washout of nitrifying bacteria during cold temperature conditions. The washout SRT is the residence time it takes for nitrifiers to replace their population. If the SRT is less than this amount, nitrifiers will not remain in the system, and will not be re-established until warmer temperatures, combined with longer SRT, are achieved. At the West Side WWTP's historic minimum monthly temperature of 10°C, a **minimum aerobic SRT of 10.8 days** is required when a safety factor of 2.5 is applied to the washout SRT. At the East Side WWTP's historic minimum monthly temperature of 11°C, a **minimum aerobic SRT of 9.9 days** is required when a safety factor of 2.5 is applied to the washout SRT. In warmer conditions, the operators could reduce the

SRT while still maintaining a safety factor of 2.5. At the historic warmer month (June-October) wastewater temperature of 21°C, a minimum SRT of 4.2 days is required at each of the WWTPs.

6.6.1.3 Secondary Clarification Modifications and Allowable MLSS Concentration

In a conventional activated sludge system, the wastewater treatment facility's (WWTF's) secondary clarification process limits the allowable mixed liquor suspended solids (MLSS) concentration in the upstream bioreactor tankage – and therefore impacts the bioreactor volume that would be required to meet process goals. State-point analysis was used to determine the maximum MLSS concentration for the existing three secondary clarifiers. Due to site constraints, it is not feasible to construct an additional clarifier without extending the site boundary into the boat yard at the West Side WWTP. Similarly, constructing a fourth secondary clarifier is not feasible at the East Side WWTP without extending the site boundary north.

The state point analyses were performed using the design maximum flows to the biological process of 58 mgd and 24 mgd (West Side and East Side WWTPs) and a sludge volume index (SVI) or 125 milliliters per gram (mL/g). The design SVI of 125 is based on historical performance for bioreactor configurations that include an anoxic bioselector zone, as would be the case in a BNR process configuration. Based on these criteria, the three existing secondary clarifiers at the West Side and East Side WWTPs can accommodate a MLSS concentration of 2,500 mg/L, with a factor-of-safety against clarifier failure of 1.3. This MLSS concentration is used as noted in the following development of alternatives.

Mechanical upgrades and equipment replacement would be required for BNR alternatives that utilize the existing secondary clarifiers at both West Side and East Side WWTPs, All process mechanical equipment would need to be replaced including nine influent stainless steel slide gates, two stainless steel slide gates between influent channels, longitudinal and cross collector chain and flight mechanisms, six stainless steel skimmings slide gates, skimmings weir and skim collection troughs. The West Side plant secondary clarifiers would require twenty-four fiberglass effluent troughs and forty-eight fiberglass weir plates. The East Side plant secondary clarifiers would require eighteen fiberglass effluent troughs and thirty-six fiberglass weir plates.

For BNR alternatives utilizing the existing secondary clarifiers, new return activated sludge (RAS) and WAS pumping systems would also be required. Six RAS pumps (two per secondary clarifier), equipped with VFDs and flowmeters, should be used to return mixed liquor to each bioreactor. RAS pumping capacity would be capable of returning 100% of the maximum month design flow (Condition B), per TR-16 recommendations. Two RAS pumps would draw from one of the three secondary clarifiers and would feed into a common RAS header that would divert mixed liquor to the pre-anoxic zone of six BNR trains.

Four WAS pumps (three duty, one standby) equipped with VFDs and flowmeters would be required at each plant. The WAS pumping systems would be capable of wasting 25 percent of each WWTP's average daily flow (Condition B) with one unit out of service, per TR-16 recommendations. WAS pumps would be connected by a common header with valves arranged so that each clarifier can be isolated with a single pump. Preliminary RAS and WAS pumping systems are presented, below in **Table 6.6-3**.

Table 6.6-3 West Side WWTP and East Side WWTP RAS and WAS Pumping Systems Preliminary Design Summary

Design Element	West Side WWTP	East Side WWTP
RAS Pumps		
Type	Vertical, non-clog, centrifugal	Vertical, non-clog, centrifugal
Number of units	6	6
Capacity, gpm	4,600 gpm	1,800 gpm
Motor Size, each	100 hp	30 hp
Turndown Required	85%	85%
WAS Pumps		
Type	Horizontal, non-clog, single stage, centrifugal	Horizontal, non-clog, single stage, centrifugal
Number of units	4 (3 duty, 1 standby)	4 (3 duty, 1 standby)
Capacity, gpm	1,740 gpm	600 gpm
Motor Size, each	40 hp	5 hp

6.6.2 Alternative Suspended Growth Activated Sludge Configuration

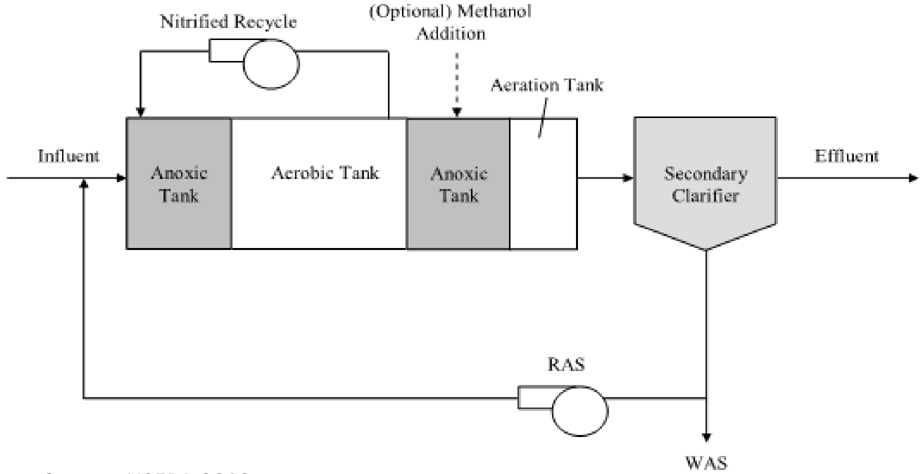
Potential modifications to the existing suspended growth activated sludge process were assessed for their ability to meet effluent TN limits at the design year flows and loads presented in the section above. The alternative process below utilizes the existing final settling tanks for solids separation.

6.6.2.1 Four-Stage Bardenpho with Continued Use of Secondary Clarifiers

The existing two-stage Modified Ludzack Ettinger (MLE) configuration (single, pre-anoxic zone followed by aeration zones) alone, is not capable of reliably achieving the effluent TN loading goal for the WWTP throughout the year, and subsequent treatment process(es), e.g. a four-stage process, will be required to remove additional nitrates. It is generally recognized that for a typical municipal wastewater, a single anoxic reactor configuration, or MLE process configuration, cannot reliably achieve an effluent total nitrogen limit lower than about 7-8 mg/L, let alone the effluent TN targets at each WWTP (<5 mg/L).

The four-stage Bardenpho process is a suspended growth activated sludge process that includes four stages of process tankage in series: a pre-anoxic stage, an aerobic stage, a post-anoxic stage, and a small re-aeration stage. BOD_5 removal and nitrification occur in the aerobic zone, and nitrified MLSS is recycled to the pre-anoxic zone for denitrification. The post-anoxic volume would provide additional denitrification of nitrate that is not recycled to the pre-anoxic zone. The second aerobic zone would serve as a “flash” re-aeration zone that strips the nitrogen gas from the mixed liquor to remove nitrogen from the process, thus reducing the potential for rising sludge in the secondary clarifiers.

Carbon addition to the post-anoxic zone is optional and would only be added as necessary to meet the permit limit. The external carbon supplements the available endogenous carbon, thereby increasing the denitrification rate in the post-anoxic zone to achieve greater nitrate removal. **Figure 6.6-1** displays a schematic of the four-stage Bardenpho process.



Source: USEPA 2010

Figure 6.6-1
Schematic of Four-Stage Bardenpho Process, with Optional Carbon Addition

Implementing a four-stage Bardenpho process to treat the West Side WWTP's design year flows and loads would require converting the existing MLE process to a four-stage treatment process and constructing additional bioreactors to achieve process volume with continued use of the existing secondary clarifiers.

To achieve adequate bioreactor volume, this conventional activated sludge process would require the construction of a new battery of bioreactors to operate in parallel with the existing BNR basins. To achieve an equal flow split between the two batteries of BNR basins, the construction of a distribution box would be required to direct half of the flow to the new "West Battery" which would be located in the boat yard and half of the flow to the modified "East Battery". Dimensions of this distribution box are presented in **Table 6.6-4.**

Table 6.6-4 West Side WWTP Primary Effluent Distribution Box Preliminary Design Summary

Design Element	Value
External Dimensions (L x W x SWD)	35-ft x 35-ft x 20-ft
Internal Dimensions (L x W x SWD)	27-ft x 27-ft x 18-ft
Rise Rate at 58 mgd	0.12 fps
Mixer	
Type	Hyperbolic
Number of units	1
Motor Size	40 hp

The new West Battery would be comprised of three, equally sized four-stage BNR basins. The design criteria for this new battery of BNR basins is presented in **Table 6.6-5**.

Table 6.6-5 West Side WWTP Design Criteria for New West Battery

Design Element	Value
Secondary Flows	
Design Average (*Condition B*)	15 mgd (*12.9 mgd*)
Peak Flow	29 mgd
Number of Trains	3
Total Process Volume	7.74 MG
Process Volume, each train	2.58 MG
Design Winter Aerobic SRT	10.8 days
Design MLSS	2,500 mg/L
Pre-Anoxic Zone	
Total Pre-Anoxic Volume	1.18 MG
Anoxic Volume, each train	0.39 MG
Number of Stages, each train	One (A)
Stage (zone) Dimensions (L x W x SWD)	A: 58-ft x 50-ft x 18-ft
Mixer Type Number of units, per train Motor Size	 Hyperbolic 2 10 hp
Aerobic Zone	
Total Aerobic Volume	4.92 MG
Aerobic Volume, each train	0.82 MG
Number of Stages, each train	One (B)
Stage (zone) Dimensions (L x W x SWD)	B: 245-ft x 50-ft x 18-ft
Internal Recycle Pumps Type Number of Units, total Capacity Motor Size, each	 Submersible propeller 3 11,944 gpm (300% ADF) 15 hp
Diffuser Type	
Post-Anoxic Zone	
Total Post-Anoxic Volume	1.48 MG
Post-Anoxic Volume, each train	0.74 MG
Number of Stages, each train	One
Stage (zone) Dimensions (L x W x SWD)	A: 75-ft x 50-ft x 18-ft
Mixer Type Number of units, per train Motor Size	 Hyperbolic 2 11 hp
Re-Aeration Zone	
Total Re-Aeration Volume	0.16 MG
Aerobic Volume, each train	0.05 MG
Number of Stages, each train	One
Stage (zone) Dimensions (L x W x SWD)	B: 8-ft x 50-ft x 18-ft
Diffuser Type	Fine bubble disk diffusers

The existing BNR basins still require additional volume to accomplish treatment objectives. The existing primary settling tanks would be converted to pre-anoxic zones. Flow through the primary settling tanks would remain unchanged to each of the six existing BNR basins. **Table 6.6-6** presents design criteria for the East Battery.

Table 6.6-6 West Side WWTP Design Criteria for Re-configured East Battery

Design Element	Value
Secondary Flows	
Design Average (*Condition B*)	15 mgd (*12.9 mgd*)
Peak Flow	29 mgd
Number of Trains	3
Total Process Volume	7.74 MG
Process Volume, each train	2.58 MG
Design Winter Aerobic SRT	10.8 days
Design MLSS	2,500 mg/L
Pre-Anoxic Zone	
Total Pre-Anoxic Volume	1.18 MG
Anoxic Volume, each train	0.39 MG
Number of Stages, each train	One (A)
Stage (zone) Dimensions (L x W x SWD)	A: 136-ft x 36-ft x 10.7-ft
Mixer Type Number of units, per train Motor Size	 Hyperbolic 2 10 hp
Aerobic Zone	
Total Aerobic Volume	4.92 MG
Aerobic Volume, each train	0.82 MG
Number of Stages, each train	Three (B, C, D)
Stage (zone) Dimensions (L x W x SWD)	B: 74.25-ft x 30-ft x 16.4-ft
Internal Recycle Pumps Type Number of Units, total Capacity Motor Size, each	 Submersible propeller 6 5,972 gpm (400% forward flow) 4 hp
Diffuser Type	Fine bubble disk diffusers
Post-Anoxic Zone	
Total Post-Anoxic Volume	1.48 MG
Post-Anoxic Volume, each train	0.74 MG
Number of Stages, each train	One
Stage (zone) Dimensions (L x W x SWD)	A: 67-ft x 30-ft x 16.4-ft
Mixer Type Number of units, per train Motor Size	 Hyperbolic 1 3 hp
Re-Aeration Zone	
Total Re-Aeration Volume	0.16 MG
Aerobic Volume, each train	0.05 MG
Number of Stages, each train	One (D)
Stage (zone) Dimensions (L x W x SWD)	B: 7-ft x 30-ft x 16.4-ft
Diffuser Type	Fine bubble disk diffusers

The secondary clarifiers at the West Side WWTP would require process mechanical equipment replacement in addition to replaced RAS and WAS pumping systems as presented in Table 6.6-3.

The East Side WWTP's existing BNR basin volume is not as deficient as the West Side WWTP's BNR volume. Despite adequate aeration volume within the existing East Side WWTP's BNR basins, the BNR system requires additional anoxic volume to accomplish denitrification treatment objectives. To do so, the existing primary settling tanks can be converted to pre-anoxic volume. Flow through the primary settling tanks to the existing BNR basins would remain unchanged. Design criteria for the four-stage process at the East Side WWTP is presented in **Table 6.6-7**.

Table 6.6-7 East Side WWTP Design Criteria for Four-Stage Bardenpho Process

Design Element	Value
Secondary Flows	
Design Average *(Condition A)*	6.4 mgd *(10 mgd)*
Peak Flow	24 mgd
Number of Trains	3
Total Process Volume	2.13 MG
Process Volume, each train	0.71 MG
Design Winter Aerobic SRT	9.9 days
Design MLSS	2,500 mg/L
Pre-Anoxic Zone	
Total Pre-Anoxic Volume	0.62 MG
Anoxic Volume, each train	0.21 MG
Number of Stages, each train	One (A)
Stage (zone) Dimensions (L x W x SWD)	99-ft x 26-ft x 10.7-ft
Mixer Type Number of units, per train Motor Size	 Hyperbolic 2 5 hp
Aerobic Zone	
Total Aerobic Volume	1.6 MG
Aerobic Volume, each train	0.26 MG
Number of Stages, each train	Three (B, C, D)
Stage (zone) Dimensions (L x W x SWD)	B: 43-ft x 20-ft x 13.8-ft
Internal Recycle Pumps Type Number of Units, total Capacity Motor Size, each	 Submersible propeller 6 4,630 gpm (400% forward flow) 3 hp
Diffuser Type	Fine bubble disk diffusers
Post-Anoxic Zone	
Total Post-Anoxic Volume	0.43 MG
Post-Anoxic Volume, each train	0.07 MG
Number of Stages, each train	One (E)
Stage (zone) Dimensions (L x W x SWD)	A: 34.4-ft x 20-ft x 13.8-ft
Mixer Type Number of units, per train Motor Size	 Hyperbolic 2 1 hp
Re-Aeration Zone	
Total Re-Aeration Volume	0.10 MG
Aerobic Volume, each train	0.02 MG
Number of Stages, each train	One (F)
Stage (zone) Dimensions (L x W x SWD)	B: 8.4-ft x 20-ft x 13.8-ft
Diffuser Type	Fine bubble disk diffusers

As discussed previously, the secondary clarifiers at the East Side plant would require process mechanical equipment replacement in addition to replaced RAS and WAS pumping systems. Unlike the RAS pumping system presented in **Table 6.6-3**, only four (three duty, one standby) RAS pumps would be required for this alternative, because there are only three pre-anoxic zones.

Table 6.6-8 presents the RAS pumping system requirements for this alternative.

Table 6.6-8 East Side WWTP RAS Pumping System with Four-Stage Bardenpho Process Preliminary Design Summary

Design Element	East Side WWTP
RAS Pumps	
Type	Vertical, non-clog, centrifugal
Number of units	3
Capacity, gpm	3,600 gpm
Motor Size, each	65 hp
Turndown Required	85%

6.6.3 Integrated Activated Sludge Processes

The suspended growth modification discussed in the previous section requires the construction of additional bioreactor volume (e.g. a new battery of BNR basins) to improve nitrogen removal performance of the activated sludge process. Integrated activated sludge processes would allow the West Side WWTP to improve the nitrogen removal performance of its secondary treatment system without building as many new process tanks. These processes would utilize the existing final settling tanks for solids separation, but they would allow a higher inventory of microorganisms to be maintained within the process without negatively impacting the performance of the settling tanks.

6.6.3.1 Integrated Fixed Film Activated Sludge (IFAS) with Continued Use of Secondary Clarifiers

The suspended growth modifications discussed in Section 6.6.2 require the construction of additional bioreactor volume to improve nitrogen removal performance of the activated sludge process. Integrated activated sludge processes would allow the WPCA to improve the nitrogen removal performance of its advanced secondary treatment system without the construction of additional tankage. These processes would utilize the existing final settling tanks for solids separation, but they would allow a higher inventory of microorganisms to be maintained within the process without negatively impacting the performance of the settling tanks.

IFAS incorporates millions of small, plastic biofilm carriers into the aerobic bioreactors of a suspended growth system. These carriers provide a large surface area for the growth of fixed-film organisms, which are kept in the aerobic zones with the use of carrier retaining screens. This allows for increased aerobic solids inventory and therefore increased treatment capacity for carbon oxidation and nitrification in a smaller volume. Biofilm biomass sloughs off the IFAS media depending on the influent load, hydraulics, and mixing intensity. Aeration must be provided by coarse- or fine-bubble aeration, as mechanical aerators can damage the media. Suspended growth activated sludge is incorporated into the process and passes through the bioreactors to settle in conventional clarifiers. This suspended biomass cycles through any anaerobic and anoxic zones required to achieve treatment goals, and these non-aerobic zones do not have any IFAS media. IFAS is a hybrid of suspended growth and attached growth biological nitrogen removal processes.

AnoxKaldnes™ IFAS by Kruger Inc.

There are different types of IFAS media and numerous manufacturers that can design and provide an IFAS system. The AnoxKaldnes system by Kruger Inc. was evaluated since this process has proven to be an industry leader in IFAS technology and since there are several larger installations, comparable in size to the West Side WWTP. **Figure 6.6-2** shows free floating plastic media which are used by Kruger. The media is shown with and without the biofilm that develops on the surface of the plastic carriers.

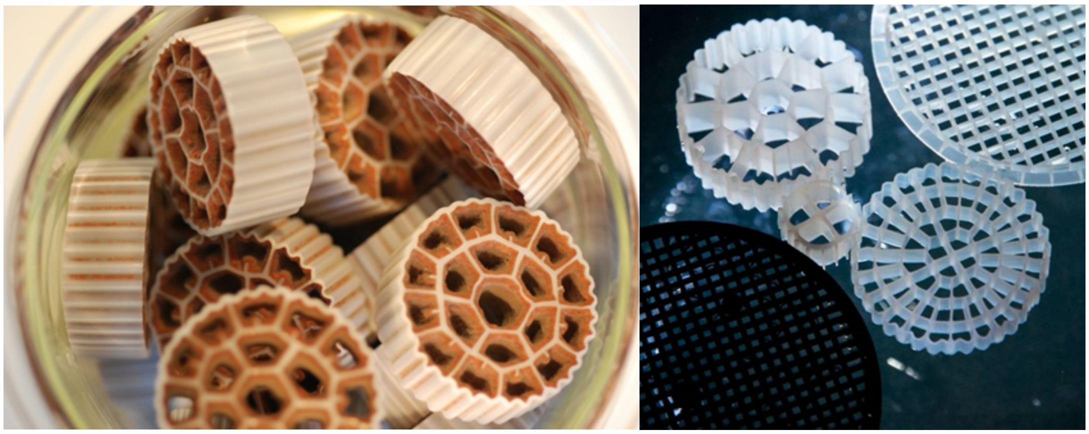


Figure 6.6-2
Typical Free Floating Plastic IFAS Media

The AnoxKaldnes media used are polyethylene wheels that have a protected specific surface area of 243.3 ft^2/ft^3, with a density slightly less than water. A coarse bubble aeration system is recommended to maintain a dissolved oxygen concentration of 4.0 to 5.0 mg/L. This differs from traditional activated sludge systems, which are typically designed for maximum oxygen transfer by maintaining a dissolved oxygen concentration of 2 mg/L. The AnoxKaldnes system does not require media recycle. Media-retaining screens with small openings (as small as 1/16-inch) are required. The process would require ¼-inch (6 mm) influent screens.

There are over 400 AnoxKaldnes installations worldwide. Most of these are of the moving bed biological reactor (MBBR) type. The MBBR is distinct from the IFAS process in that settled RAS is not returned to the front of secondary treatment. The IFAS media would be installed throughout the aerobic zone of a four-stage Bardenpho process to achieve the treatment goals. The process configuration differs between the two WWTPs.

The West Side WWTP's BNR basins require significant changes, as shown in **Figure 6.6-3**, to maintain a low enough hydraulic flux through the system to prevent media migration towards the end of the screen. The horseshoe flow path through the existing BNR basins would be eliminated (all internal baffle walls separating the existing tanks into the four equally sized zones would be demolished). The channel currently used to step-feed primary effluent would be utilized as the main bioreactor influent channel. The existing effluent channels to the secondary clarifiers would be left unchanged.

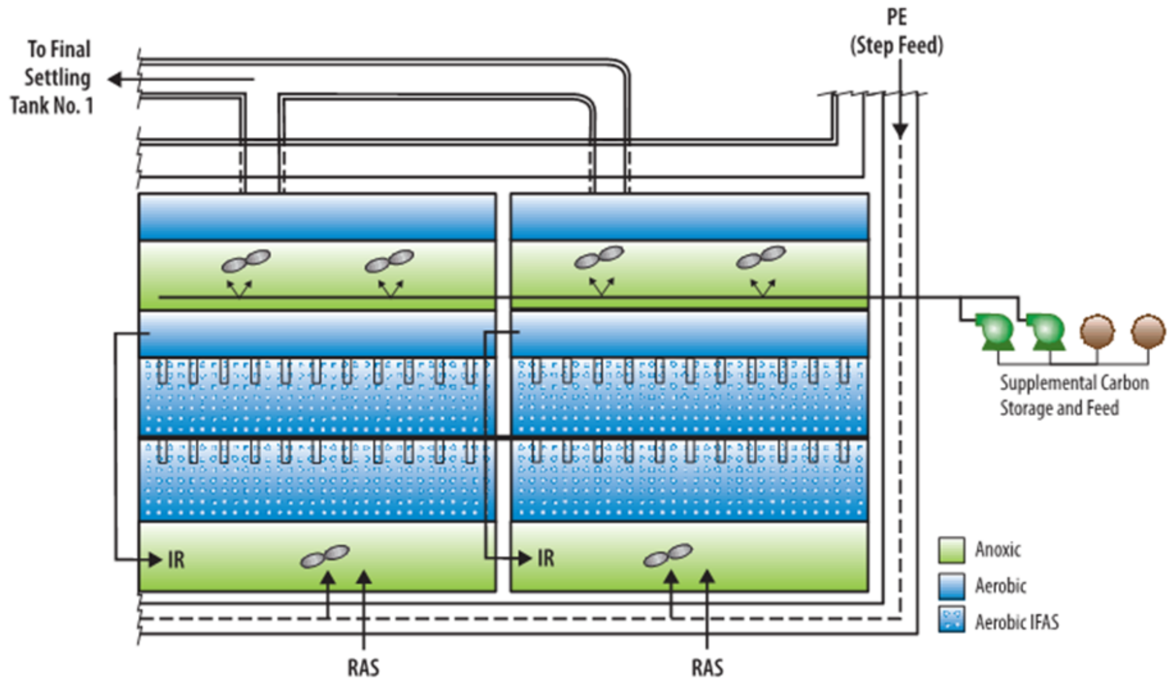


Figure 6.6-3
West Side WWTP Four-Stage Bardenpho IFAS Configuration

Table 6.6-9 presents the design criteria for the Anoxkaldnes design at the West Side WWTP.

Table 6.6-9 West Side WWTP IFAS (Anoxkaldnes) Process Design Data

Design Element	Value
Secondary Flows	
Design Average *(Condition B)*	30 mgd *(25.8 mgd)*
Peak Flow	58 mgd
Number of Trains	6
Total Process Volume	4.66 MG
Process Volume, each train	0.777 MG
Design Winter Aerobic SRT	10.8 days
Design MLSS	2,500 mg/L
Pre-Anoxic Zone	
Total Pre-Anoxic Volume	0.27 MG
Anoxic Volume, each train	0.045 MG
Number of Stages, each train	One (A)
Stage (zone) Dimensions (L x W x SWD)	A: 36.7-ft x 60-ft x 16.4-ft
Mixer Type Number of units, per train Motor Size	 Submersible 1 5.73 hp
IFAS Reactor (Aerobic)	
Total Aerobic Volume	2.43 MG
Aerobic Volume, each train	0.41 MG
Number of Stages, each train	Two (B and C)
Stage (zone) Dimensions (L x W x SWD)	B: 28.6-ft x 60-ft x 15.9-ft
Media Type	AnoxK™5
Fill of Biofilm Carriers	50%
Media Retention Screens, each train	44 (22 per stage)
Aeration System Type	Medium/Coarse Bubble
DeOxygenated Reactor	
Total Deoxygenated Reactor Volume	0.67 MG
Deoxygenated Volume, each train	0.11 MG
Number of Stages, each train	One (D)
Stage (zone) Dimensions (L x W x SWD)	D: 16-ft x 60-ft x 15.6-ft
Internal Recycle Pumps Type Number of Units, total Capacity Motor Size, each Average Internal Recycle Rate	 Submersible 12 8,785 gpm 12.6 hp 200%-300%
Aeration System Type	Medium/Coarse Bubble
Post-Anoxic Zone	
Total Post-Anoxic Volume	0.67 MG
Post-Anoxic Volume, each train	0.11 MG
Number of Stages, each train	One (E)
Stage (zone) Dimensions (L x W x SWD)	E: 16-ft x 60-ft x 15.6-ft
Mixer Type Number of units, per train Motor Size	 Submersible 2 1.65 hp
Re-Aeration Zone	
Total Re-Aeration Volume	0.67 MG
Aerobic Volume, each train	0.11 MG
Number of Stages, each train	One (F)
Stage (zone) Dimensions (L x W x SWD)	F: 16-ft x 60-ft x 15.6-ft
Diffuser Type	Medium/Coarse Bubble

The IFAS process, by Kruger can accomplish treatment objectives within the existing BNR tankage at the West Side WWTP. A design summary is presented in **Table 6.6-10**.

Table 6.6-10 IFAS Preliminary Design Summary

Design Element	Value
Typical Headloss (ft)	1.1-ft
Equipment Supplied by Vendor	HDPE carrier elements (ANOX-K5), 264 media retention screens, air sparge system (solenoid airflow valves and v-port ball valves), 18 medium bubble aeration system (includes modulating airflow control valve and thermal mass flowmeters), 12 scum screen assemblies (including spray header assemblies), 18 submersible mixers, 12 IR pumps, 1 PLC, 12 high level float switches, 24 LDO DO probes
Equipment Not Supplied by Vendor	Carbon storage and feed system

The secondary clarifiers at the West Side WWTP would require process mechanical equipment replacement in addition to replaced RAS and WAS pumping systems as presented in Table 6.6-3.

Minor changes to the existing bioreactors at the East Side WWTP are required to implement IFAS within the existing BNR basins, as shown **Figure 6.6-4**. The internal partition walls will need to be altered, slightly. To minimize hydraulic flux and prevent media migration towards the ends of the screens, a portion of the existing step feed channel will be used as a transfer channel. This transfer channel separates the two IFAS media zones. Despite this modification, the horseshoe flow path through the BNR basins remains unchanged along with the existing effluent channels to the secondary clarifiers would be left unchanged.

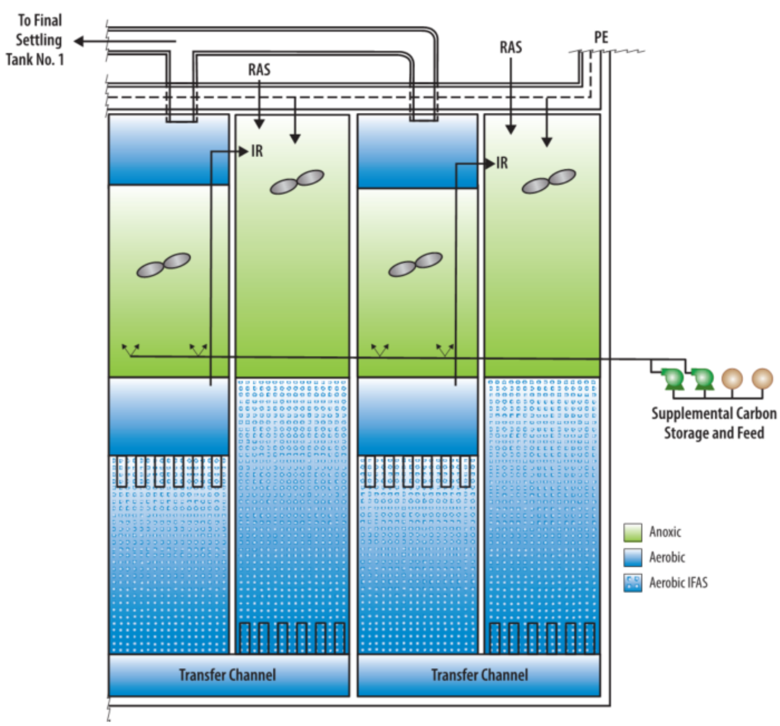


Figure 6.6-4
East Side WWTP Four-Stage Bardenpho IFAS Configuration

Table 6.6-11 presents the design criteria for the Anoxkaldnes design.

Table 6.6-11 East Side WWTP IFAS (Anoxkaldnes) Process Design Data

Design Element	Value
Secondary Flows	
Design Average *(Condition A)*	6.4 mgd *(10 mgd)*
Peak Flow	14.1 mgd
Number of Trains	6
Total Process Volume	2.16 MG
Process Volume, each train	0.36 MG
Design Winter Aerobic SRT	9.9 days
Design MLSS *(Max Month)*	2,500 mg/L
Pre-Anoxic Zone	
Total Pre-Anoxic Volume	0.53 MG
Anoxic Volume, each train	0.09 MG
Number of Stages, each train	One (A)
Stage (zone) Dimensions (L x W x SWD)	A: 43-ft x 20-ft x 13.8-ft
Mixer Type Number of units, per train Motor Size	 Submersible 1 4.2 hp
IFAS Reactor	
Total Aerobic Volume	0.92 MG
Aerobic Volume, each train	0.15 MG
Number of Stages, each train	Two (B, C)
Stage (zone) Dimensions (L x W x SWD)	B: IFAS 1: 43-ft x 20-ft x 13.8-ft
Media Type	AnoxK™5
Fill of Biofilm Carriers	25%
Media Retention Screens, each train	12 (6 per stage)
Aeration System Type	Medium/Coarse Bubble
DeOxygenated Reactor	
Total Deoxygenated Reactor Volume	0.13 MG
Deoxygenated Volume, each train	0.02 MG
Number of Stages, each train	One (D)
Stage (zone) Dimensions (L x W x SWD)	D: 10.75-ft x 20-ft x 13.8-ft
Internal Recycle Pumps Type Number of Units, total Capacity, each Motor Size, each Average Internal Recycle Rate	 Submersible 6 1,460 gpm 2.7 hp 100%-200%
Aeration System Type	Medium/Coarse Bubble
Post-Anoxic Zone	
Total Post-Anoxic Volume	0.40 MG
Post-Anoxic Volume, each train	0.07 MG
Number of Stages, each train	One (E)
Stage (zone) Dimensions (L x W x SWD)	E: 31.6-ft x 20-ft x 13.8-ft
Mixer Type Number of units, per train Motor Size	 Submersible 1 2.55 hp
Re-Aeration Zone	
Total Re-Aeration Volume	0.13 MG
Aerobic Volume, each train	0.02 MG
Number of Stages, each train	One (F)
Stage (zone) Dimensions (L x W x SWD)	F: 10.8-ft x 20-ft x 13.8-ft
Diffuser Type	Medium/Coarse Bubble

The IFAS process, by Kruger can accomplish treatment objectives within the existing BNR tankage. **Table 6.6-12** includes a preliminary design summary of the proposed IFAS system at the East Side WWTP.

Table 6.6-12 East Side WWTP IFAS Preliminary Design Summary

Design Element	Value
Typical Headloss (ft)	1.4 ft
Equipment Supplied by Vendor	HDPE carrier elements, 72 media retention screens, air sparge system (solenoid airflow valves and v-port ball valves), 18 medium bubble aeration system (includes modulating airflow control valve and thermal mass flowmeters), 12 scum screen assemblies (including spray header assemblies), 12 submersible mixers, 6 IR pumps, 1 PLC, 12 high level float switches, 18 LDO DO probes
Equipment Not Supplied by Vendor	Carbon storage and feed system

The secondary clarifiers at the West Side WWTP would require process mechanical equipment replacement in addition to replaced RAS and WAS pumping systems as presented in **Table 6.6-3.**

Nuvoda MOB™- Emerging IFAS Process

The MOB™ (Mobile Organic Biofilm™) process by Nuvoda is another IFAS technology that utilizes processed plant material (Kenaf) as the biofilm carrier, as opposed to plastic media utilized by the Anoxkaldnes process. The Kenaf is processed to create particles 1 mm in size. The particles have a high surface area and absorptive properties that are thought to enhance the fixed film process and granular properties of the sludge. The MOB granules are fully mobile and circulate throughout the BNR basins and secondary clarifiers. **Figure 6.6-5** shows free floating Kenaf media which are used by Nuvoda. The media is shown with and without the biofilm that develops on the surface of the Kenaf carriers.

Figure 6.6-5
Free Floating Kenaf Biofilm Carrier

The Kenaf particle carriers are removed from the secondary clarifiers with the sludge and are recycled to the head of the activate sludge process with the RAS. The carriers in the WAS are collected in a rotary screen and are returned to the head of the activated sludge process along with the RAS. Due to inherent media loss through the effluent and through the rotary screen, Kenaf needs to be routinely replenished to the process. **Figure 6.6-6** presents a process flow diagram of MOB's implementation into the 6.2 mgd Moorefield (WV) WWTP in 2018.

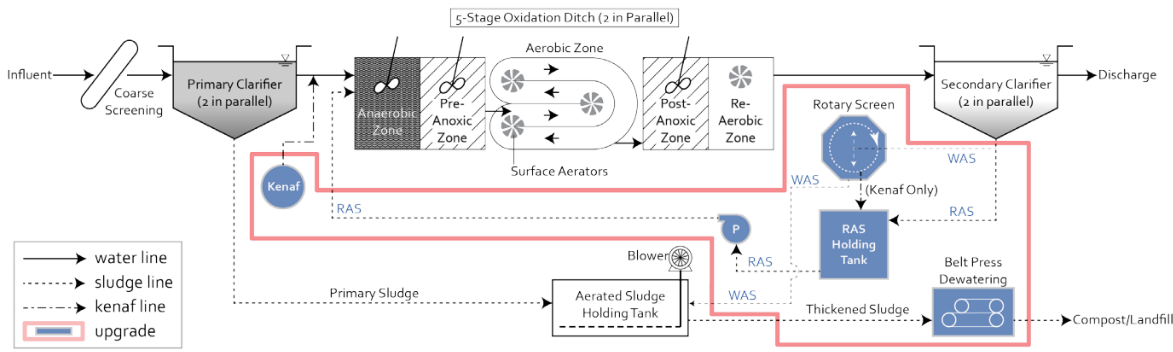


Figure 6.6-6
Example Process Flow Diagram of the MOB Process Implemented at the Moorefield WWTP

This is a very new technology that does not have many installations within the United States. The majority of installations are at industrial food processing and chemical WWTPs and not at municipal plants. There are no MOB installations at a plant of similar size to either of the WPCA's WWTPs. Because of the lack of similarly sized installations, and the lack of long-term operating data, the MOB process was not further evaluated as a viable IFAS process for implementation at the West Side and East Side WWTPs.

6.6.4 Membrane Bioreactors (MBRs)

6.6.4.1 Three Stage Activated Sludge Process with MBRs

Membrane bioreactors (MBRs) consist of biological treatment using suspended biomass combined with solids separation with microfiltration (pore sizes 0.1 to 0.4 µm) membranes. The membranes essentially replace the clarifiers in a conventional system. In the biological treatment component of an MBR, MLSS concentrations are higher than conventional activated sludge systems (≥10,000 mg/L). An MBR consists of a suspended growth biological reactor integrated with membranes (typically reinforced hollow fiber or flat-plate (sheet) membranes). The membranes are immersed directly in the process tank. A centrifugal or reversible pump is used to create a vacuum in the header connecting the membranes, drawing the treated water through the membranes in an outside-in flow path. Airflow is introduced at the bottom of the membrane module via coarse aeration to create a turbulence that scours the external surface of the membranes and keeps them clean and also provides process aeration. Supplemental oxygen for the biological treatment is provided by a separate diffused aeration system.

As shown in **Figure 6.6-7**, the membrane reactor would be combined with the existing MLE process and newly constructed post anoxic tanks. Large MLSS recycle (RAS) pumps are required to maintain high MLSS within the secondary system.

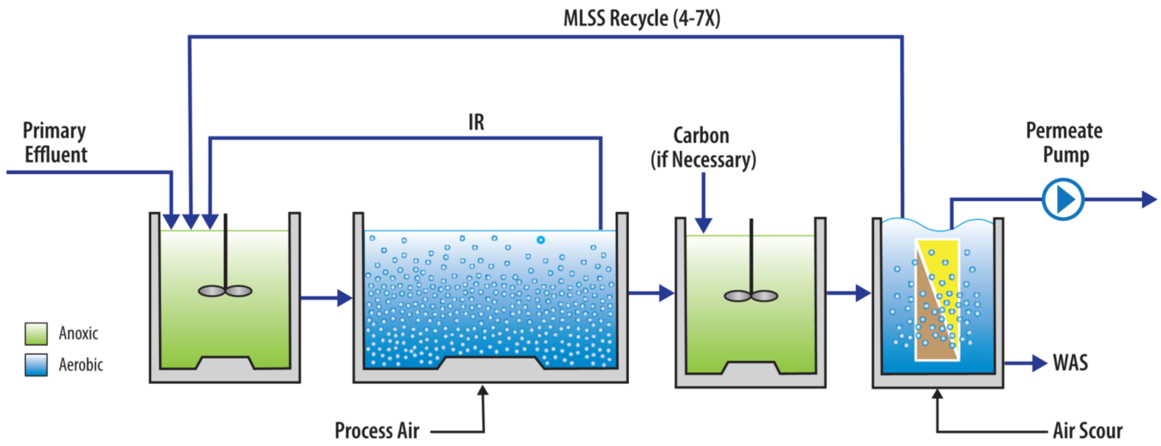


Figure 6.6-7
Schematic of an MBR Process

It is expected that MBRs in a four-stage Bardenpho configuration would achieve nitrogen removal to below 5 mg/L TN. Because the MBRs replace the existing secondary clarifiers, a higher MLSS concentration can be used to maximize BNR performance, thereby eliminating the need to expand aeration volumes. With that said, despite not needing to expand aeration volume, additional post anoxic volume is required to accomplish denitrification and achieve effluent nitrogen objectives.

MBRs require additional pumping for high recycles flows. If traditional primary clarifiers are used (in lieu of primary filtration), fine screening (2-mm) is required upstream of the MBR. This could be accomplished by a stand-alone fine screening facility Because of the high flows, primary effluent should pass through fine screens to eliminate the added recycle flows.

Table 6.6-13 presents preliminary design criteria for a fine screening facility. The fine screening facility footprints include an electrical room.

Table 6.6-13 Fine Screen Preliminary Design Summaries

Design Element	Value
West Side WWTP	
Maximum Primary Effluent Flow	58 mgd
Screen Type	Center-flow perforated plate
Number of Screens	7 (5 duty, 2 standby)
Capacity, each screen	12 mgd
Facility Footprint	2,850 ft^2
East Side WWTP	
Maximum Primary Effluent Flow	24 mgd
Screen Type	Center-flow perforated plate
Number of Screens	3
Capacity, each screen	12 mgd
Facility Footprint	1,950 ft^2

During the course of the Facilities Planning effort, no scenarios were developed that would require the use of the fine screening facility at the West Side WWTP, because the cost of constructing traditional primary clarifiers and a fine screen facility far exceeded the cost of a primary filter facility, as presented in Section 9. The use of primary filtration (10 um cloth) would afford a higher level of filtration compared to the fine screens (2 mm).

As discussed, the existing BNR basins' MLE configuration would remain unchanged. Mechanical upgrades would be recommended, that include the installation of hyperbolic mixers within anoxic zones, replacement of fine bubble diffused aeration system in aeration zones and replace internal recycle pumps.

At the West Side WWTP, three new post-anoxic tanks need to be constructed. **Table 6.6-14** presents design criteria for the activated sludge process to be used upstream of the MBR system.

Table 6.6-14 West Side WWTP Activated Sludge Process upstream of MBR Alternative Process Design Data

Design Element	Value
Secondary Flows	
Design Average (*Condition B)*	30 mgd (*25.8 mgd*)
Peak Flow	58
Number of Trains	6
Total Process Volume	7.56
Process Volume, each train	1.26
Design Winter Aerobic SRT	10.8 days
MLSS	7,000 mg/L
Pre-Anoxic Zone	
Total Process Volume	1.64
Process Volume, each train	0.2733
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
Mixer Type Number of units, per train Motor Size	 Hyperbolic 2 3 hp
Aerobic Zone	
Total Process Volume	4.92
Process Volume, each train	0.82
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 C: 74.25 ft x 30 ft x 16.4 ft D: 74.25 ft x 30 ft x 16.4 ft
Internal Recycle Pumps Type Number of Units, total Capacity, gpm Motor Size, each	 Submersible 6 10,417 gpm 12.2 hp
Post-Anoxic Tanks	
Total Process Volume	1.00
Process Volume, each tank	0.33
Number of Tanks	Three
Mixer Type Number of units, per train Motor Size	 Hyperbolic 1 20 hp

Design Element	Value
Additional Equipment	
RAS Pumps Number of units Capacity, gpm Motor Size	 6 17,400 gpm 375 hp
WAS Pumps Number of units Capacity, gpm Motor Size	 3 (2 duty, 1 standby) 2,600 gpm 55 hp

Because membranes take the place of the conventional secondary clarifiers, the existing secondary clarifiers at the West Side WWTP would be demolished. As discussed, large RAS flowrates are required to maintain the high MLSS concentrations within the secondary process (500% of average design flow (ADF)).

Table 6.6-15 presents design criteria for the activated sludge process to be used upstream of the MBR system at the East Side plant. No additional tankage is required.

Table 6.6-15 East Side WWTP Activated Sludge Process upstream of MBR Alternative Process Design Data

Design Element	Value
Secondary Flows	
Design Average *(Condition A)*	6.4 mgd *(10 mgd)*
Peak Flow	24
Number of Trains	6
Total Process Volume	2.12
Process Volume, each train	0.35
Design Winter Aerobic SRT	9.9 days
MLSS	7,000 mg/L
Pre-Anoxic Zone	
Total Process Volume	0.27
Process Volume, each train	0.044
Number of Stages, each train	One (A)
Stage (zone) Dimensions (L x W x SWD)	A: 21.4 ft x 20 ft x 13.8 ft
Mixer Type Number of units, per train Motor Size	 Hyperbolic 1 1 hp
Aerobic Zone	
Total Process Volume	1.05
Process Volume, each train	0.18
Number of Stages, each train	Two (B and C)
Stage (zone) Dimensions (L x W x SWD)	B: 64.6 ft x 20 ft x 13.8 ft C: 21.4 ft x 20 ft x 13.8 ft
Internal Recycle Pumps Type Number of Units, total Capacity Motor Size, each	 Submersible 6 2,963 gpm 3 hp
Post-Anoxic Zones	
Total Process Volume	0.80
Process Volume, each tank	0.13
Number of Stages, each train	One (D)

Design Element	Value
Stage (zone) Dimensions (L x W x SWD)	D: 64.6 ft x 20-ft x 13.8-ft
Mixer Type Number of units, per train Motor Size	 Hyperbolic 2 1.5 hp
Additional Equipment	
RAS Pumps Number of units Capacity, gpm Motor Size	 6 5,800 gpm 100 hp
WAS Pumps Number of units Capacity, gpm Motor Size	 3 (2 duty, 1 stanby) 870 gpm 7 hp

Because membranes take the place of the conventional secondary clarifiers, the existing secondary clarifiers at the East Side WWTP would be demolished. As discussed, large RAS flowrates are required to maintain the high MLSS concentrations within the secondary process (500% of ADF).

ZeeWeed MBR by Suez

There are different types of MBRs and numerous manufacturers that can design and provide an MBR system, including Evoqua, Suez, and Pall Corporation. Only one of the three manufacturers were considered for this evaluation. The ZeeWeed system by Suez was evaluated since this process has proven to be an industry leader in MBR technology, with most of the industry's largest and longest-operating MBR plants. The earliest MBR installation has been in operation for more the 10 years. The proposed MBR design utilizes LEAPmbr, which includes ZeeWeed 500 ultrafiltration membrane modules. **Table 6.6-16** present ZeeWeed Preliminary Design summaries for both WWTPs.

Table 6.6-16 West Side WWTP and East Side WWTP ZeeWeed Preliminary Design Summaries

Design Element	West Side WWTP Value	East Side WWTP Value
Number of Membrane Trains	12	6
Process Footprint per Train (ft x ft)	89 ft x 9 ft	82 ft x 9 ft
Total Process Footprint (ft^2)	9,650 ft^2	4,450 ft^2
Modules, each cassette	52	52
Tank Depth (ft)	13 ft	13 ft
Total Process footprint (ft^2)	19,800 ft^2	12,000 ft^2
Treatment Capacity per Unit (mgd)	5.3 mgd	4.8 mgd
Total Treatment Capacity of System	58 mgd with 1 in standby	24 mgd with 1 in standby
Equipment Supplied by Vendor	144 membrane cassettes and modules, membrane tank cassette mounting assemblies, permeate collection and air	66 membrane cassettes and modules, membrane tank cassette mounting assemblies, permeate collection and air

Design Element	West Side WWTP Value	East Side WWTP Value
	distribution header pipes, 12 membrane tank level transmitters and switches, 12 process pumps (valves, pressure gauges, and flow meters), 12 vacuum ejectors (with air compressors and refrigerated air dryers), 12 pressure transmitters, 12 turbidimeters, 1 air scour blower (valves, pressure gauges and flow switches), backpulse pumps (valves, switches, and flow meter), backpulse water storage tank, sodium hypochlorite chemical feed system, citric acid chemical feed system, 1 system control panel	distribution header pipes, 6 membrane tank level transmitters and switches, 6 process pumps (valves, pressure gauges, and flow meters), 6 vacuum ejectors (with air compressors and refrigerated air dryers), 6 pressure transmitters, 6 turbidimeters, 1 air scour blower (valves, pressure gauges and flow switches), backpulse pumps (valves, switches, and flow meter), backpulse water storage tank, sodium hypochlorite chemical feed system, citric acid chemical feed system, 1 system control panel
Equipment Not Supplied by Vendor	Traveling bridge crane/monorail (10,000-lb capacity) for membrane removal, lifting davits with hoist, 2-mm fine screens (if required), VFDs and MCC	Traveling bridge crane/monorail (10,000-lb capacity) for membrane removal, lifting davits with hoist, 2-mm fine screens (if required), VFDs and MCC

6.6.5 Membrane Aerated Biofilm Reactors (MABRs)

6.6.5.1 With Continued Use of Secondary Clarifiers

Membrane aerated biofilm reactors (MABRs) are a relatively new technology for aerobic wastewater treatment. The process relies of oxygen diffusion through gas permeable membrane into attached growth biofilms.

ZeeLung* by Suez is a MABR technology that intensifies biological treatment by supporting nitrifying biofilm on ZeeLung membranes installed in the mixed liquor within anoxic zone. The membrane cassettes are made of hollow fiber oxygen-permeable gas transfer membranes. Air is pumped through the module and oxygen is delivered by diffusion to the biofilm while substrate (ammonia and carbon) diffuse from the bulk solution into the biofilm. ZeeLung is an integrated activated sludge process that utilizes the fixed biofilm on the surface of the membranes in addition to the suspended growth bacteria in the bulk liquid. Because air is delivered through the membrane surface, the oxygen rich environment along the membrane media allows for nitrifiers to grow in the anoxic zone. This promotes simultaneous nitrification and denitrification and reduces the downstream aerobic SRT required for nitrification. **Figure 6.6-8** shows a ZeeLung installation concept in an anoxic tank.

Table 6.6-17 presents a summary of the preliminary design values for a Suez ZeeLung system at both WWTPs.

Despite the small footprint and minimal structural modifications required to accommodate the ZeeLung into the existing BNR basins at the West Side plant, the technology is very new (first ZeeLung installation in 2017) and has no similarly sized installations to date. As of July 2020, there were installations at only 10 commercial plants, three of which were in operation for more than two years. The combined full-scale installed capacity of these plants is only 5.3 mgd. Because of the lack of long-term operational data at a similarly sized plant, the ZeeLung was not considered to be a viable BNR alternative for the West Side WWTP.

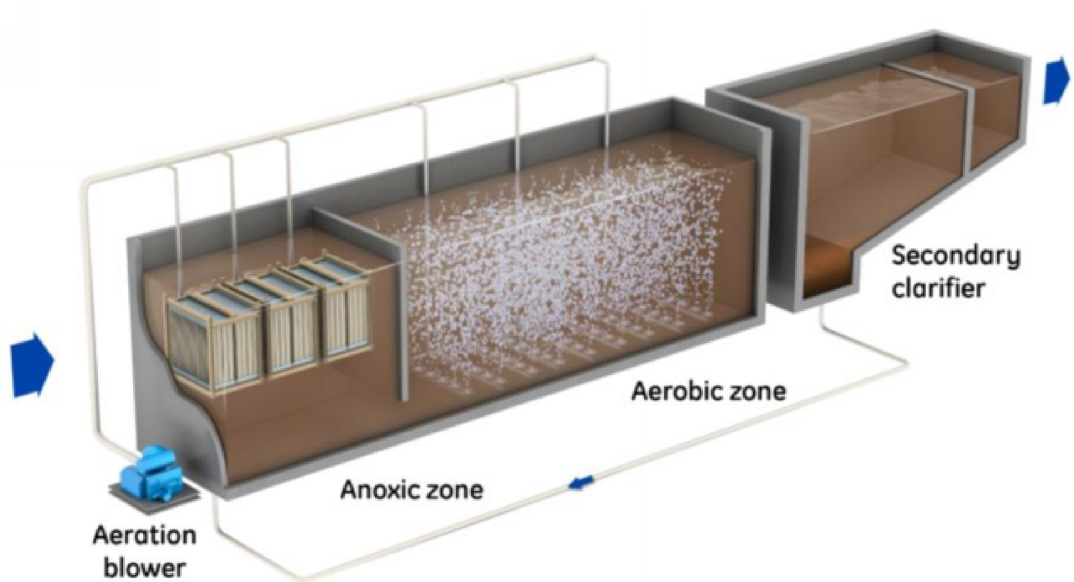


Figure 6.6-8
ZeeLung Installation Concept

Table 6.6-17 West Side WWTP and East Side WWTP ZeeLung Preliminary Design Summaries

Design Element	West Side WWTP Value	East Side WWTP Value
Number of Membrane Cassettes per tank	27	8
Total Number of Cassettes	162	48
Process Footprint per Train (ft x ft)	NA (process can be implemented within existing tankage)	NA (process can be implemented within existing tankage)
Total Process Footprint (ft^2)	NA (process can be implemented within existing tankage)	NA (process can be implemented within existing tankage)
Side Water Depth (ft)	16 ft	13.6 ft
Total Treatment Capacity of System	58 mgd	24 mgd
Typical Headloss (ft)	NA	NA
Equipment Supplied by Vendor	162 membrane cassettes and modules, membrane tank cassette support beams, blowers (duty/standby), valves, condensate removal system, oxygen transfer monitoring system, compressor, and system control panel with PLC	48 membrane cassettes and modules, membrane tank cassette support beams, blowers (duty/standby), valves, condensate removal system, oxygen transfer monitoring system, compressor, and system control panel with PLC
Equipment Not Supplied by Vendor	VFD for process blowers	VFD for process blowers

Because of the lack of long-term operational data at a similarly sized plant, the ZeeLung was not considered to be a viable BNR alternative for the East Side WWTP at the time of this Facilities Plan. However, should the WPCA wish to re-evaluate ZeeLung in the future for the East Side WWTP, more long-term operational data should become available and can be used to revisit the status of this technology and its acceptance throughout the industry.

6.6.6 Add-On Nitrogen Removal Process

An alternative to achieving additional nitrogen removal within the BNR secondary treatment process is to use an add-on tertiary treatment process. Denitrification filters are a subset of biologically active filters (BAFs) that are tailored specifically to provide denitrification to very low levels of effluent total nitrogen (less than 3 mg/L). Denitrification filters function similarly to typical deep-bed filters used in water and wastewater treatment applications. Treatment is provided by passing secondary effluent through a granular filter bed. The filter media is designed to not only remove effluent TSS, but to serve as surface area for growth of a denitrifying biofilm. An anoxic environment is maintained within the filter, and when nitrate-rich secondary effluent contacts the biofilm, the heterotrophic organisms within the biofilm convert the nitrate to nitrogen gas. A supplemental carbon source is required because there is virtually no BOD_5 remaining in the treated secondary effluent at this tertiary stage. Supplemental carbon is added to the filter influent and is usually flow-paced with fine dose adjustment based on a target effluent nitrate concentration. Denitrification filters also require periodic backwash to remove collected solids, and trapped nitrogen gas bubbles are periodically flushed from the system.

Denitrification filters rely on biological growth for nitrogen removal; therefore, some soluble orthophosphate (as much as 0.3-0.5 mg/L) must be present in the secondary effluent to ensure biofilm growth is not inhibited. The need for orthophosphate (OP) in the filter influent is a concern at both WWTPs since raw influent phosphorus is typically low. Denitrification filters were evaluated since they are one of the few processes that have a proven track record of achieving an effluent TN concentration as low as 3 mg/L.

Downflow denitrification and up flow denitrification filters were evaluated. Downflow denitrification filters are deep bed gravity sand filters that can be used for removing nitrogen to low levels (less than 3 mg/L) and most manufacturers utilize similar design criteria (hydraulic loading rates), so only one manufacturer's system was evaluated.

DeNiFOR™- Downflow Denitrification Filter

The DeNiFOR System by Suez is an attached growth, microbial process, schematic shown in **Figure 6.6-9**.

This gravity, downflow, packed-bed denitrification system is physically identical to a deep-bed down flow sand filter. Denitrifying microorganisms attach to the filter media, which provides the support system for their growth. A carbon source is added upstream of the packed-bed filter and a nitrified influent is filtered through the media. The filter media is composed of an expanded clay material. This media can filter out solids and serve as a support system for the denitrifying microorganisms. Solids build-up in the filter media is removed by a backwash process. The backwash starts with an air scour, followed by an air/water wash and then ends with a final water wash. Water is pulled from a clear well of filter effluent. An air bump is used to drive off the nitrogen gas (typically needed every 1-4 hours). The air bump occurs based on an increase in headloss due to the gas buildup. The headloss will diminish after an air bump up to a point until the solids buildup is too much and a backwash is required.

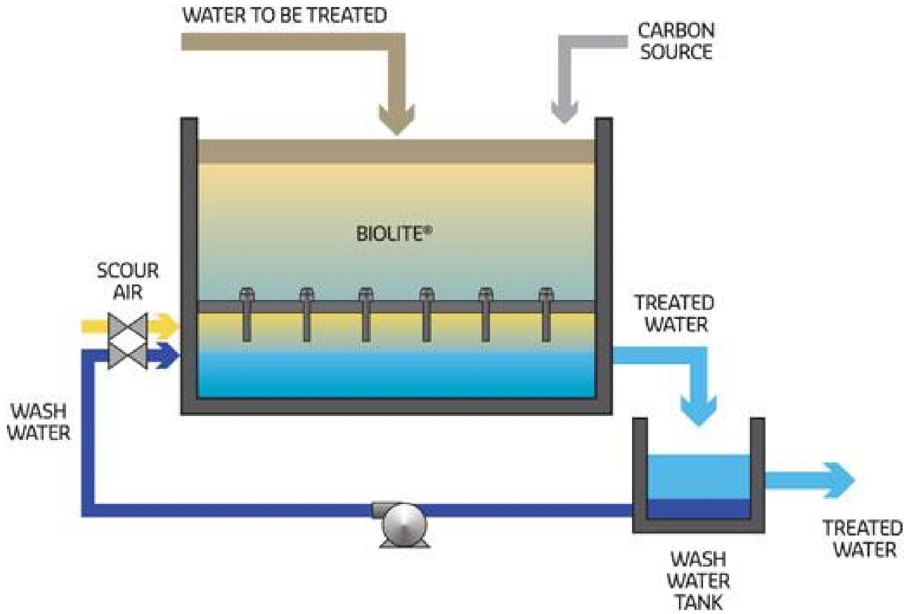


Figure 6.6-9
Schematic of the DeNiFOR Process

Table 6.6-18 presents a summary of the preliminary design values for a Suez DeNiFOR system at both plants. All support facilities and superstructures required for the process (including filter gallery, backwash waste tank and clean wash water tank) are included in the total system area.

Table 6.6-18 West Side WWTP and East Side WWTP DeNiFOR Preliminary Design Summaries

Design Element	West Side WWTP Value	East Side WWTP Value
Number of Units	8	4
Process Footprint per Unit (L x W)	35-ft x 20-ft	27.5-ft x 20-ft
Total Process Footprint (ft^2)	5,600 ft^2	2,200 ft^2
Sidewater Depth (ft)	15.5 ft	15.5 ft
Media Depth (ft)	6.0	6.0
Total Area of System (ft^2)	11,000 ft^2	5,460 ft^2
Equipment Supplied by Vendor	8 filters with underdrains, influent weir and air/water backwash distribution system, 8 influent gate valves, filter media, gravel support bed, influent channel, effluent and backwash waste channels with stilling baffle, 2 backwash scour air blowers, 2 backwash pumps, 1 backwash inlet strainer, 1 compressed air system (compressor, receiver, dryer, valves & gauges), 1 system control panel with PLC	4 filters with underdrains, influent weir and air/water backwash distribution system, 4 influent gate valves, filter media, gravel support bed, influent channel, effluent and backwash waste channels with stilling baffle, 2 backwash scour air blowers, 2 backwash pumps, 1 backwash inlet strainer, 1 compressed air system (compressor, receiver, dryer, valves & gauges), 1 system control panel with PLC
Equipment Not Supplied by Vendor	Carbon storage and feed system	Carbon storage and feed system

The DeNiFOR system was not identified as a feasible alternative for add-on nitrogen removal because of its large footprint (11,000 square feet at the West Side plant, and 5,460 square feet at the East Side plant). There is lot of equipment required for its backwash process which would substantially increase operational complexity and mechanical maintenance requirements.

Upflow filters use a submerged media bed for fixed film nitrogen removal down to 3 mg/L, but the manufacturers use different types of media which results in different design criteria (e.g. hydraulic loading rates) because of this, two different filters were evaluated.

Blue Nite®- Upflow Denitrification Filter

Blue Nite by Nexom is a biologically activated filter that utilizes supplemental carbon to accomplish denitrification and polish TSS, shown in **Figure 6.6-10.** The Blue Nite filter is a continuous backwash sand filter. Fixed-film heterotrophic bacteria convert nitrates to nitrogen gas. Bacteria and solids wasting are facilitated by the continuous backwash. The media wash box removes solids and excess biomass, which are directed to a waste line. The clean sand then falls by gravity back to the media bed.

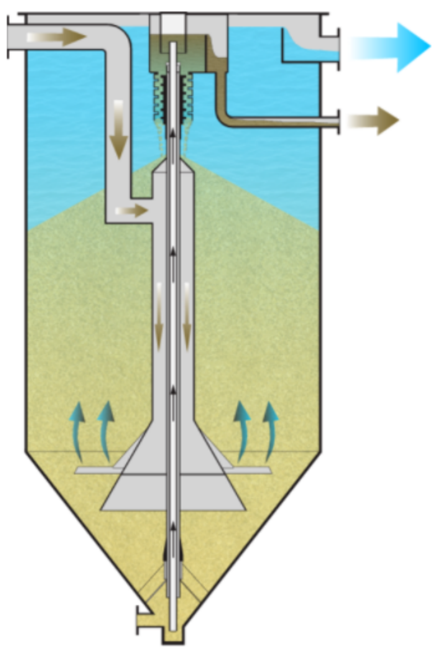


Figure 6.6-10
Schematic of the Blue Nite Filter

Table 6.6-19 presents a summary of the preliminary design values for a Nexom Blue Nite system at the West Side WWTP. Only filter areas are included in the total footprint.

The modular nature of the filters allows ease of system engineering, and ultimate expansion. However, the size of the filters is not amenable to a plant the size of the West Side WWTP. Two hundred filters (Model CF64-80) are required to accommodate the West Side WWTP's flows and loads, which is impractical from an O&M perspective, and the space required for the system cannot fit within the existing site.

Table 6.6-19 West Side WWTP and East Side WWTP Blue Nite Preliminary Design Summaries

Design Element	West Side WWTP Value	East Side WWTP Value
Number of Units	20 cells (10 filters per cell)	7 cells (10 filters per cell)
Process Footprint per Unit (ft x ft)	41.4-ft x 17.2-ft	41.4-ft x 17.2-ft
Total Process Footprint (ft^2)	16,000 ft^2	4,720 ft^2
Sidewater Depth (ft)	21 ft	21 ft
Media Depth (ft)	6.7 ft	6.7 ft
Equipment Supplied by Vendor	200 filters with internals and HDPE airlifts, 20 airlift control panels, 1 pneumatic system (2 VFD driven rotary screw compressors, filtration and refrigerated drivers), aluminum covers and supports over filters	70 filters with internals and HDPE airlifts, 7 airlift control panels, 1 pneumatic system (2 VFD driven rotary screw compressors, filtration and refrigerated drivers), aluminum covers and supports over filters
Equipment Not Supplied by Vendor	Filter system control panel (including PLC and HMI), carbon storage and feed system	Filter system control panel (including PLC and HMI), carbon storage and feed system

BIOSTYR® – Upflow Denitrification Filter

BIOSTYR is a biological aerated filter by Kruger, Inc. **Figure 6.6-11** that allows carbon removal, nitrification, and denitrification to be carried out in the same cell. It would be used for denitrification only at the West Side WWTP. It consists of polystyrene beads manufactured specifically and sized according to the required result. These beads are blocked under the ceiling of the concrete structure (the cell) which is fitted with nozzles which allow the treated water to flow out.

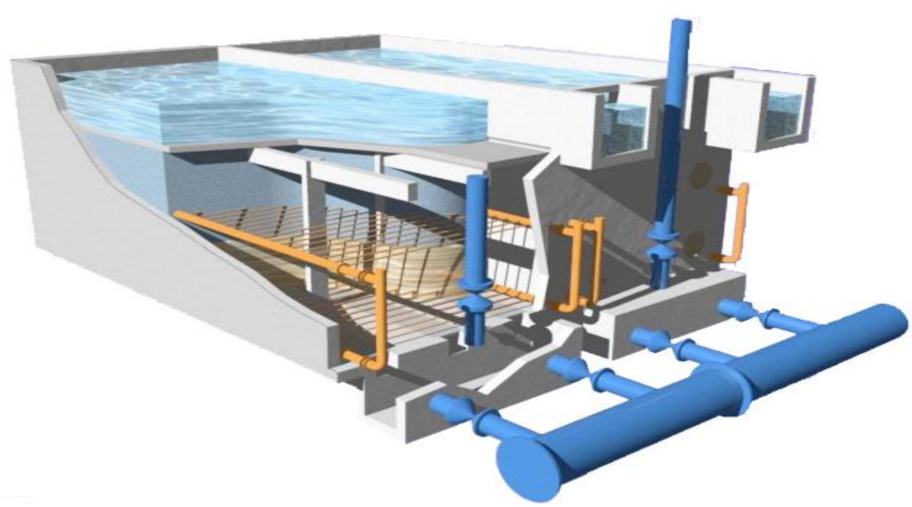


Figure 6.6-11
Schematic of the BIOSTYR Process

Feeding from the base causes "self-compacting" of the filter bed, with a beneficial effect on its ability to retain suspended solids. The denitrification process is carried out in a non-aerated environment by adding an external carbon source to transform the excess residual into gaseous nitrogen. The modular design of BIOSTYR makes it a suitable tool in cases of variable load because portions of the cells can be stopped and restarted quickly.

Biomass and suspended solids accumulated inside the filter medium must be removed regularly by periodic backwashes. The light weight of the material allows the system to be washed by simple gravity expansion of the filter medium using the treated water stored in the upper structure (flushing).

Table 6.6-20 presents a summary of the preliminary design values for a Kruger BIOSTYR system at the West Side WWTP. All support facilities (including pipe gallery, backwash waste equalization tank and equipment rooms) and superstructures required for the process are included in the total system area.

Table 6.6-20 West Side WWTP and East Side WWTP BIOSTYR Preliminary Design Summaries

Design Element	West Side WWTP Value	East Side WWTP Value
Number of Units	4	4
Process Footprint per Unit (ft x ft)	28 ft x 56.67 ft	16.3 ft x 43.3 ft
Total Process Footprint (ft^2)	6,235 ft^2	2,850 ft^2
Sidewater Depth (ft)	21.5 ft	21.5 ft
Media Depth (ft)	8.2	8.2 ft
Total Area of System (ft^2)	15,200 ft^2	7,800 ft^2
Equipment Supplied by Vendor	4 filters with nozzle slabs, media, backwash aeration grids (inlet header, purger header, distribution lines, SS piping, etc), channel cover plates, process valves, PD rotary lobe blowers, 3 sludge pumps, 1 compressed air system (compressor, receiver, dryer, valves & gauges), 1 system control panel	4 filters with nozzle slabs, media, backwash aeration grids (inlet header, purger header, distribution lines, SS piping, etc), channel cover plates, process valves, PD rotary lobe blowers, 2 sludge pumps, 1 compressed air system (compressor, receiver, dryer, valves & gauges), 1 system control panel
Equipment Not Supplied by Vendor	Carbon storage and feed system	Carbon storage and feed system

Despite the reliability of denitrification filters, the space for a new filter facility is not feasible on the plant's existing site. Furthermore, additional process tankage upstream of the denitrification filters would be required to accomplish nitrification goals, which occupies even more space. Denitrification filters were not considered to be a viable alternative for the West Side WWTP. Although additional aeration volume is not required at the East Side plant, the size of the facility required to accommodate denitrification filters does not fit onto the site.

6.6.7 Nutrient Removal Alternatives Summary and Recommendations

The following suspended growth nutrient removal processes were selected for further evaluation in Section 9. These processes would modify the existing MLE secondary treatment processes to improve nitrogen removal at the East Side and West Side WWTPs.

- Four stage Bardenpho with carbon addition (activated sludge configuration)
- AnoxKaldnes IFAS (integrated activated sludge process)
- Membrane bioreactors (alternative activated sludge configuration)

The following nutrient removal alternatives were not evaluated further for the reasons identified in this section. These alternatives are either too complex to operate at a large plant, take up too much space on site, or are too new to be recommended and considered for implementation at both WWTPs.

- MOB IFAS (integrated activated sludge process)
- Membrane Aerated Biofilm Reactors (MABRs)
- Downflow Denitrification Filters (add-on nitrogen removal process)
- Upflow Denitrification Filters (add-on nitrogen removal process)

6.6.8 Aeration Blowers

To supply air to the activated sludge process, the existing aeration blowers will be replaced. Aeration blowers will be designed to deliver the air requirements of the selected biological process. The aeration process typically has the highest energy demand of any process in an activated sludge treatment facility. It can account for 25 to 60 percent of a facility's total energy use. Therefore, the design of aeration systems must have the flexibility to handle variations in oxygen demand, including hourly, daily, and seasonal variations. Specific attention must be paid to minimum oxygen demands in the initial years of operation to ensure the blowers can be turned-down to meet the low end of the spectrum.

There are numerous different blower types which could be used in this application, but all consist of lobes, impellers, or screws mounted to a rotating shaft or shafts powered by a motor. Blowers used for activated sludge systems are typically classified as either:

1. Positive displacement (PD) blowers, which provide a constant volume of air over a wide range of operating discharge pressures; or
2. Centrifugal blowers, which provide a variable range of airflow over a narrower range of operating discharge pressures.

Alternative blower technologies are presented in **Figure 6.6-12.**

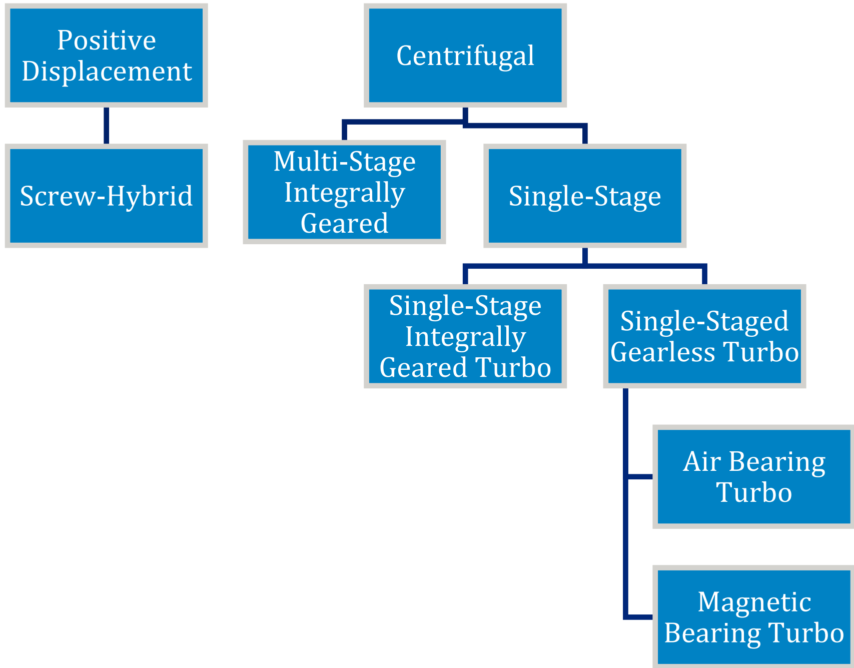


Figure 6.6-12
Alternative Blower Technologies

Centrifugal blowers can be further categorized as single-stage or multi-stage machines. Multistage centrifugal blowers use a series of impellers with vanes mounted on a rotating shaft (typically 3,600 rotations per minute (rpm)). Each successive impeller increases discharge pressure. Conversely, single-stage centrifugal blowers use a single impeller rotating at high speeds (exceeding 10,000 rpm and in some cases beyond 20,000 rpm) to provide the required discharge pressure. Positive displacement and single-stage centrifugal blowers are preferred for activated sludge systems because of their operating efficiencies. Multi-stage centrifugal blowers were not considered to be viable alternatives for either of the WWTPs.

Single stage centrifugal blowers can be further divided into the integrally geared (i.e. gear-driven) type or the gearless (i.e. direct-drive) type, as will be discussed further in the sections below. A more recent entrant to the market is the positive displacement-screw hybrid, which combines the turndown capabilities of rotary lobe PD blowers with the efficiency of screw centrifugal compressors; the latter are typically used for applications requiring higher discharge pressures than typical for many municipal facilities.

6.6.8.1 Preliminary Aeration Design Data

As a preliminary design point, CDM Smith evaluated maximum month oxygen demand to determine nominal airflow requirements for the West Side WWTP and East Side WWTP. General design criteria for each plant is presented in **Table 6.6-21.**

Table 6.6-21 West Side WWTP and East Side WWTP Preliminary Blower Design Criteria

Parameter	West Side WWTP	East Side WWTP
Number of Units	4 (3 duty, 1 standby)	3 (2 duty, 1 standby)
Airflow per blower, scfm	6,600 scfm	2,700 scfm
Discharge Pressure, psig	9.4 psig	7.6 psig

6.6.8.2 Positive Displacement Blowers

Positive displacement (PD) blowers, known as rotary lobe blowers, utilize two counter rotating shafts with two or three lobes to compress a defined volume of air per rotation. This delivers a constant volume of air despite any changes in the required pressure. Changes in air flow are accomplished with a VFD to lower the rotation speed of the shafts and therefore the frequency of air passing through the blower. Typically, PD blowers are provided in pre-packaged, acoustical enclosures to reduce noise. PD blowers can be turned down to 40% or less of rated flow, but turndown to less than 50% of rated flow will likely be more costly.

PD blowers were not considered to be a suitable alternative for the West Side WWTP due to the high airflow requirements. PD blowers were determined to be a viable blower technology for use at the East Side WWTP.

Standard PD blowers have been manufactured for over 150 years and are commonly produced by many manufacturers. Howden and Aerzen were consulted for this evaluation as both have over a century of blower experience and have been used on numerous, previous designs. A basic design summary for each is shown in **Table 6.6-22**, below.

Table 6.6-22 East Side WWTP Positive Displacement Blower Preliminary Design Summaries

Parameter	Howden	Aerzen
Number of Units	Three (2 duty / 1 standby)	Three (2 duty / 1 standby)
Blower Model	EAX2-250-728-200HP	GM 90S
Design Flow per Blower	2,700 scfm	2,700 scfm
Motor Size	200 hp	150 hp

The advantages and disadvantages of PD blowers are listed in **Table 6.6-23**.

Table 6.6-23 Positive Displacement Blowers Advantages and Disadvantages

Advantages	Disadvantages
• Widely used • Low capital cost • Ease of operation • Large turndown • Operates in wide range of conditions	• Typically, the least efficient blower technology (improved with screw-hybrid) • Requires frequent maintenance • Can by noisy (enclosures are commonly used for noise control) • Experiences slip, especially at high pressure or low speed

Screw-Hybrid Blowers

Screw-hybrid are a new entry to the blower sector and have a twisted lobe and shaft similar to a screw compressor. They operate in the same manner as a standard PD blower but exhibit higher efficiencies. **Table 6.6-24** presents the preliminary design summary for a hybrid blower.

Table 6.6-24 East Side WWTP Hybrid Blower Preliminary Design Summary

Parameter	Aerzen
Number of Units	Three (2 duty / 1 standby)
Blower Model	D98S Hybrid
Design Flow per Blower	2,700 scfm
Motor Size	150 hp

6.6.8.3 Centrifugal Blowers

Single-Stage Integrally Geared Turbo Blowers

Single-stage integrally geared turbo blowers use a gear box between the motor and the blower shaft to achieve the high speed required at the impeller. These units require pressure lubrication to provide oil to bearings on the blower and/or the gear boxes. Most manufacturers use a journal (or plain) bearing design, while some newer manufacturers have elected to standardize on a newer style of ceramic bearing which requires less oil. **Figure 6.6-13** provides a cutaway of the key mechanical components of a typical integrally geared turbo blower.

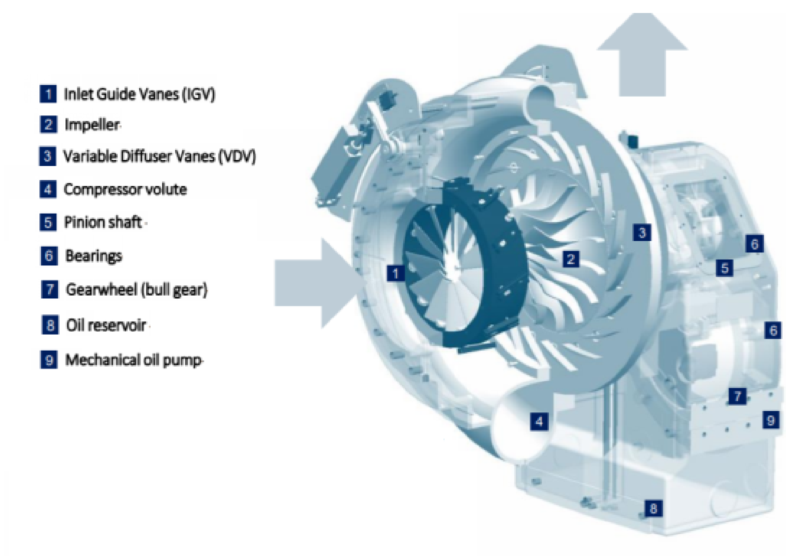


Figure 6.6-13
Mechanical Components of an Integrally Geared Turbo Blower[1]

[1] Adapted from NextTurbo Technologies brochure. Accessed at https://www.next-turbo.com/launch/datasheets/Brochure_booklet_en.pdf.

Geared blowers operate at a constant speed, and modulation of the air flow output is achieved with motor-actuated inlet guide vanes (IGVs) and variable diffuser vanes (VDVs). Although these blowers can be provided with only IGVs or only VDVs, often both are furnished to provide "dual-point control" which allows for independent control of flow and discharge pressure. Dual-point control offers the advantage of maintaining a high efficiency across the entire range of the blower curve from the maximum design point down to minimum flow at turndown, which is generally about 40 to 50 percent of the maximum output depending on the required operating conditions.

Geared turbo blowers can be furnished either open, with insulating blankets, or in an acoustic enclosure for noise reduction. **Figure 6.6-14** displays an integrally geared blower installed without an acoustic enclosure.

Figure 6.6-14
Integrally Geared Turbo Blower

For this evaluation, Howden and NexTurbo were consulted given their history of installations and ability to provide long-term service support throughout North America. A basic design summary for each is shown in **Table 6.6.25** below.

Table 6.6-25 West Side WWTP Single-Stage Centrifugal Integrally Geared Turbo Blower Preliminary Design Summary

Parameter	Howden	NexTurbo
Number of Units	Four (3 duty / 1 standby)	Four (3 duty / 1 standby)
Blower Model	KA10SV-GK200	GTB-T30-XY
Design Flow per Blower	6,600 scfm	6,600 scfm
Max Capacity per Blower	7,300 scfm	7,168 scfm
Motor Size	350 hp	350 hp

The advantages and disadvantages of integrally geared turbo blowers are listed in **Table 6.6-26**.

Table 6.6-26 Integrally Geared Turbo Blowers Advantages and Disadvantages

Advantages	Disadvantages
• More efficient than positive displacement blowers, especially with turndown • Can maintain good efficiency at turndown • Typically come with integral control system for surge protection • Smaller footprint	• Higher capital cost (compared to PD blowers) • Many moving parts • Can be noisy (enclosures are commonly used for noise control) • Lubricant required • Surge can be damaging • Limited turndown

Gearless Air Bearing High-Speed Turbo Blowers

Gearless turbo blowers mount the impeller directly on the shaft of a special motor operating at high speeds of up to 200,000 rpm. This design eliminates the need for belts and speed-increasing gears, as well as the need for lubricants. In order to obtain the high motor speeds, gearless turbo manufacturers can provide either a high-speed induction motor, a brushless DC high speed motor, or a permanent magnet synchronous motor (PMSM). The PMSM type is the most efficient type of high speed motor and can be furnished by several manufacturers including E&A, Synchronoy, and Turbo Power Systems. To modulate the motor and impeller speed and thus the blower flow output, a high frequency VFD or high frequency converter is used, as it must provide an output frequency much higher than the normal 60 Hz supplied by the power utility. A harmonic filter is also generally required to reduce negative impacts of harmonics on the plant's power system. **Figure 6.6-15** displays the major components of a gearless turbo blower.

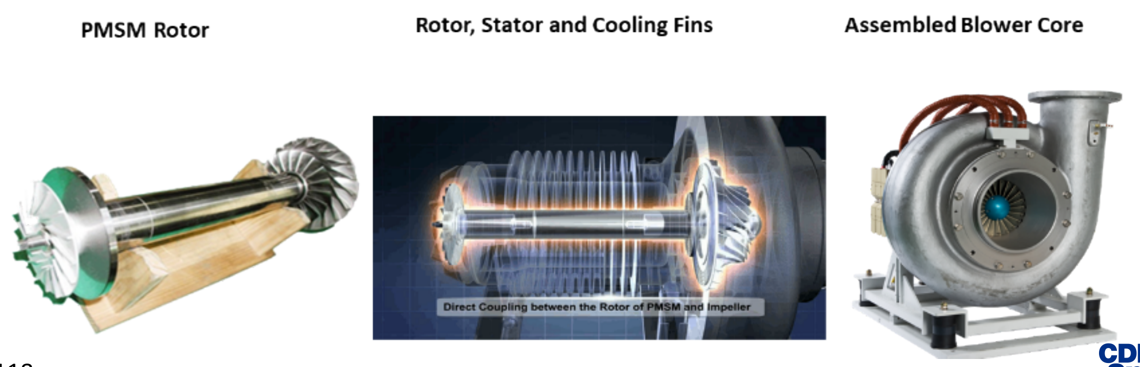


Figure 6.6-15
Components of a Gearless Turbo Blower (Courtesy of APG-Neuros)

Whereas geared turbo blowers use oil to lubricate the bearing, all gearless turbo blowers use special oil-less air bearings or magnetic bearings to support the blower and motor shaft.

An air foil bearing's stability control is "passive," meaning that no outside power or controller is required to maintain the bearing in position. Rather, the mechanical design and the operating conditions (i.e. sufficiently high speed) are used to keep the shaft levitated. The bearings do become subject to wear over time since they experience friction at the low speeds that occur during starting and stopping, but some manufacturers have implemented cooling fans or high strength coatings to extend bearing life.

There was a high incident of failures of air bearing gearless turbo blowers installed at numerous facilities across the country from 2007 to 2013. Failure analysis reports issued by several manufacturers at the time (including APG-Neuros and HSi) often pointed to dust, pollen, and other particulate material entering the blower enclosure, which can cause rotor imbalance and can also clog the cooling pathways, which was suspected to result in heating and expansion of the metal and ultimately contact and failure of the bearings.

Manufacturers have used preventative maintenance, bearing temperature monitors, and vibration sensors to provide indication of the performance and durability of the bearings to prevent blower failure.

For the purposes of this study only Neuros and Aerzen were contacted for proposals given their established record in the blower industry and reputation for responsive service support. A preliminary design summary of air bearing high-speed turbo blowers at the WWTPs is presented in **Table 6.6-27**.

Table 6.6-27 West Side WWTP and East Side WWTP Gearless Air Bearing High-Speed Turbo Blower Preliminary Design Summary

Parameter	West Side WWTP – Aerzen	West Side WWTP – Neuros	East Side WWTP – Aerzen	East Side WWTP - Neuros
Number of Units	Four (3 duty / 1 standby)	Four (3 duty/1 standby)	Three (2 duty / 1 standby)	Three (2 duty / 1 standby)
Blower Model	AT 400-0.8 T G5	NX300S-C070	AT 150-0.8S G5 Plus	NX150S-C060
Design Flow per Blower	6,600 scfm	6,600 scfm	2,700 scfm	2,700 scfm
Max Capacity per Blower	7,323 scfm	7,708		3,966 scfm
Motor Size	400 hp	300 hp	150 hp	150 hp

The advantages and disadvantages of air bearing turbo blowers are listed in **Table 6.6-28**.

Table 6.6-28 Air Bearing Turbo Blowers Advantages and Disadvantages

Advantages	Disadvantages
• More efficient than PD blowers, especially with turndown • Small footprint • No lubrication required • Typically come with integral control system for surge protection	• Higher capital cost than PD blowers (although less expensive than integrally geared) • Many moving parts • Surge can be damaging • Limited turndown • History of failures

Gearless Magnetic Bearing High-Speed Turbo Blowers

Gearless magnetic bearing turbo blowers are nearly identical to air bearing blowers in terms of the "air end" design and operational principles described above. The difference is that magnetic bearing blowers use dynamically actuated electromagnets to levitate the blower shaft so that it can rotate with minimal to no friction. The following excerpt and **Figure 6.6-16** from a WEFTEC paper published in 2013 by engineers from Spencer Turbine provides a useful summary of the "active" magnetic bearing system:

"The system is called active because sensors that are part of the bearing detect the actual location of the center of the rotating assembly and pass the information to a Magnetic Bearing Controller

(MBC). The controller decides if the power going to any of the magnets must be adjusted to achieve a more centered position and does so accordingly." [2]

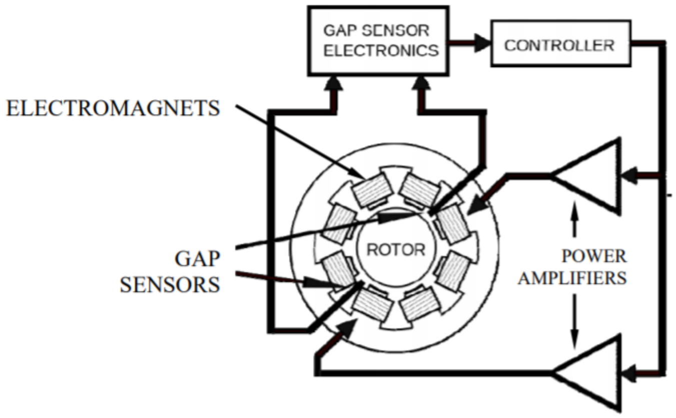


Figure 6.6-16
Magnetic Bearing Schematic (Irving and Ibets, 2013)

Since the magnetic bearing type must be actively powered to maintain the shaft in the correct position, several back-up systems must be included to protect the compressor in the event of a power outage. The back-up methods used by different manufacturers to provide power to the MBC in the event of a power outage include: (1) use of an uninterruptible power supply (UPS) with battery back-up, (2) use of the motor as a generator, or (3) use of a DC/DC converter. Either method is designed to keep the shaft levitated until it stops rotating. An additional back-up protection in the event of a failure of the UPS is the provision of "touchdown" roller bearings which bring the rotor down to speed more slowly than an instantaneous stop to prevent a catastrophic failure requiring a complete replacement of the compressor.

The gearless magnetic bearing blower technology was first installed in 1996 at the Botnia Pulp Mill in Joutseno, Finland. Since the ABS/Sulzer acquisition in 2002, over 3,000 HST units of the ABS/Sulzer magnetic bearing blower design have been installed globally, including more than 200 units in 80 plus facilities in North America. The ABS/Sulzer compressors continue to be manufactured in Finland, and Sulzer offers service support from its engineering office located in Meriden, Connecticut.

For this study, Sulzer was consulted for a quotation and sizing for the magnetic bearing blower technology due to their long-term experience. A preliminary design summary of magnetic bearing high-speed turbo blowers at the WWTPs is presented in **Table 6.6-29**.

[2] Irving, Aleksandra and Joseph Ibets (2013). "High Speed Bearing Technologies for Wastewater Treatment Applications." Proceedings of WEFTEC 2013 in Chicago Illinois. Paper accessed at https://www.spencerturbine.com//content/uploads/2019/05/High-Speed-Bearing-Technologies-for-WWT-Applications_022414.pdf

Table 6.6-29 West Side WWTP and East Side WWTP Gearless Magnetic Bearing High-Speed Turbo Blower Preliminary Design Summary

Parameter	West Side WWTP – Sulzer	East Side WWTP - Sulzer
Number of Units	Four (3 duty / 1 standby)	Three (2 duty / 1 standby)
Blower Model	HST 30-58-8-U350-48	HST 20-4500-1-U150-48
Design Flow per Blower	6,600 scfm	2,700 scfm
Max Capacity per Blower	7,323 scfm	3,400 scfm
Motor Size	350 hp	150 hp

The advantages and disadvantages of magnetic bearing turbo blowers are listed in **Table 6.6-30**.

Table 6.6-30 Magnetic Bearing Turbo Blowers Advantages and Disadvantages

Advantages	Disadvantages
• More efficient than positive displacement blowers, especially with turndown • Small footprint • No lubrication required • Typically come with integral control system for surge protection	• Higher capital cost than PD blowers (although likely less expensive than integrally geared) • Many moving parts • Surge can be damaging • Limited turndown • Requires backup power in the event of power loss

6.6.9 Aeration Blower Technology Alternatives Summary and Recommendations

In general, the conclusion of the technology review and initial screening process is that single-stage centrifugal blowers are the preferred technology for the West Side WWTP due to their higher efficiencies and smaller footprints. Similarly, single stage centrifugal blowers are the preferred alternative for the East Side WWTP over PD blowers.

At this stage of planning, CDM Smith did not perform a full life cycle cost analysis for individual blower technologies to confirm single stage centrifugal blowers are the most preferred alternative blower technology for both of the WWTPs. Due to variations in air demand of the various BNR process alternatives, the cost analysis varies significantly and could produce different results. Regardless of the BNR process alternative selected, it is preferred that the designs use the same blower technology to reduce operator training and to allow for potential standardization, The following blower alternatives will be further evaluated during design to select the optimal blower technology and configuration for the secondary process recommended by this Facilities Plan:

- Positive Displacement including Screw-Hybrid (East Side WWTP only)
- Single-Stage Integrally Geared Turbo
- Single-Stage Gearless Turbo
    - Air Bearing Turbo
    - Magnetic Bearing Turbo

6.7 Disinfection

This development and screening of disinfection alternatives includes an overview of the following mature and developing disinfection technologies to identify the most feasible and applicable technologies for the West Side and East Side WWTPs:

- Sodium Hypochlorite
- Chlorine Dioxide
- Peracetic Acid
- Ozone
- Ultraviolet Radiation

As detailed in Section 4, the elevation and hydraulic grade of the existing chlorine contact tanks are below the design flood elevation at both plants; therefore, reuse of the tanks is not feasible. The components for all technologies were assumed to be constructed in new structures. Depending on the final recommended treatment plant alternative, some disinfection alternatives may be considered for retrofit into other existing structures during design.

6.7.1 Sodium Hypochlorite

Chlorine disinfection (by sodium hypochlorite application) and chloramination is a well-established and commonly used disinfection technology and is currently used at both WWTPs.

6.7.1.1 System Components

A new sodium hypochlorite system would be similar to the existing systems and would include contact tanks, bulk chemical storage tanks, and feed pumps. This system would also require a dechlorination system to remove excess chlorine residual after the contact tanks and prior to discharge to the receiving waters.

Contact Tanks

New chlorine contact tanks would be designed with a length to width ratio in conformance with design guidelines (typically >40) and multiple passes to maximize flow path and reduce short-circuiting. Tanks would be designed to provide 30 minutes of contact time at average design flow, and 15 minutes at peak flow since providing 30 minutes at peak wet weather flows would result in oversized tanks. Two contact tanks would be provided. CFD modeling would be used in design to optimize chemical mixing and contact time to validate the design. Designing to this lesser standard will require approval from CT DEEP.

Bulk Chemical Storage Tanks

New high density, cross-linked polyethylene storage tanks would allow for storage of the 12.5% to 15% delivered sodium hypochlorite solution. Total storage volume would be dependent on average and maximum flows as well as chemical degradation, but at least two tanks would be provided for redundancy.

Feed Pumps

Feed pumps would be provided to supply chemical to the contact tanks and other application points in the WWTPs (e.g. plant water system, RAS bulking control). For the contact tanks, the pumps would be sized to provide chlorine feed rates at all effluent flows for a range of dosage. Chemical would be dosed based on plant flow and trimmed based on chlorine residual.

Dechlorination System

To remove excess chlorine, sodium bisulfite would be added after the chlorine contact tanks. This reaction is considered nearly instantaneous so does not require a contact tank. This system would be similar to the existing dechlorination system consisting of chemical storage tanks and feed pumps. A mixer could be used to increase distribution of sodium bisulfite into the effluent flow. Sodium bisulfite would be dosed based on chlorine residual and flow.

6.7.1.2 Advantages and Disadvantages

A summary of the advantages and disadvantages of chlorination are provided in **Table 6.7-1**. Chlorination with sodium hypochlorite is a mature wastewater disinfection technology that is widely used and well-understood. Its current use at the WWTPs would make permitting an expanded system straightforward. However, use of chlorine has potential for increasing the formation of disinfection by-products (DBP) in the treated effluent.

Table 6.7-1 Advantages and Disadvantages of Chlorination/Dechlorination

Advantages	Disadvantages
Well established technology	Sodium hypochlorite degrades as a function of time and temperature
Well understood disinfection mechanisms	Can produce DBPs, dependent upon mode of operation
Easiest to apply	Increases total dissolved solids
Readily available	Requires quenching (dechlorination)
Commonly permitted for WRRF disinfection	Requires chemical delivery, storage and handling

6.7.2 Chlorine Dioxide

Chlorine dioxide gas (ClO_2) is a proven bactericide that has been widely used for disinfection of potable waters; however, experience with wastewater disinfection is very limited although it is gaining interest in use as a disinfectant for produced water from hydrofracturing operations. Few pilot studies have been conducted for municipal wastewater applications, and no known major municipal wastewater applications are operated in the U.S. This is primarily because it must be generated on-site due to chemical instability and safety concerns; chemical costs are high; and the overall system is complex to operate and maintain, relative to chlorination. Because this is not a readily used technology in the wastewater field and is complex to operate, it will not be considered further for the West Side and East Side WWTPs.

6.7.3 Peracetic Acid

Peracetic Acid (PAA) is oxidizer that has been applied to the food, beverage, medical and pharmaceutical industries for disinfection, and its use has recently expanded to include wastewater treatment facilities, mainly in Europe. The United States Environmental Protection Agency (USEPA) has approved PAA for use as a disinfectant to treat wastewater; however, its full-scale use at WWTPs in the U.S. is limited. As of 2020 there are at least 40 full-scale PAA disinfection systems operating across the U.S. It is most often considered at WWTPs with difficult effluent or stringent permit requirements aimed to minimize DBP formation. Additionally, PAA does not require quenching which makes it an attractive alternative to sodium hypochlorite. However, PAA is more expensive than sodium hypochlorite and its dose is plant dependent and can be higher than sodium hypochlorite.

Applicability is often dependent on pilot testing to determine dose and effectiveness. PAA has been pilot tested at facilities all across North America, including Ohio, New York, Mississippi, the Great Lakes Basin and Ontario, Canada. However, it is not currently approved for use in Connecticut as a wastewater disinfectant. Given that pilot testing would need to be performed, approval from state would be required, operation may be more expensive, and use is limited in the wastewater sector, PAA will not be considered further for the Bridgeport WWTPs. If pilot testing is performed, and CT DEEP is amenable for consideration, this disinfectant could be resurrected during preliminary design.

6.7.4 Ozone

Ozonation is a mature disinfection technology that merits consideration, even though only a few WWTPs in the U.S. currently use ozone. Historically, ozone has been used as a drinking water treatment technology more than a wastewater treatment technology; and many of the recent advances in ozone generation and dissolution technology have been developed by the drinking water industry making ozone more economical over the past decade. The improved economics of ozone generation, along with consideration of secondary benefits of ozone including low DBP formation potential, color removal and ability to capture and reuse vent gas for the biological system are resulting in increasing interest in its application at WWTPs.

6.7.4.1 System Components

Ozone is an unstable gas that decomposes to elemental oxygen very rapidly after generation and must be generated on-site for use. In general, the major system components for an ozone disinfection system typically include:

- Oxygen supply system
- Ozone generators and the associated power supply units (PSUs)

- Ozone contactors and the associated ozone gas transfer system
- Ozone contactor off-gas handling and residual ozone gas destruction systems and ozone gas monitoring and control systems

6.7.4.2 Advantages/Disadvantages

The primary advantages of ozonation for wastewater disinfection are that it is a highly effective disinfectant that acts in a relatively short contact time; it decomposes rapidly so there are no residuals that must be quenched. The process does not form halogenated DBPs. The primary disadvantage of ozonation is that the process has high capital costs. Other considerations are that ozone is very reactive and corrosive; therefore, ozone in off-gases from ozone contactors must be destroyed if the gas is not consumed during contacting. The higher capital cost can be justified in applications where there are secondary considerations, such as color removal or DBP control that would make this technology more attractive, but these are not significant drivers for this project. Ozonation is a mature technology, but because it is not currently widely used for wastewater disinfection in the United States and is complex and expensive to operate, it will not be considered further as a disinfection option at the WWTPs. A summary of the advantages and disadvantages of ozone disinfection are provided in **Table 6.7-2**.

Table 6.7-2 Advantages and Disadvantages of Ozone

Advantages	Disadvantages
Short contact time	Complicated equipment to provide high transfer efficiency
Rapid decomposition, no harmful residual to remove	Complex O&M
No halogenated DBP formation	High capital costs
Increases dissolved oxygen in effluent	Must destroy off-gassed ozone
Capable of oxidizing color, odor, other organics	Can generate DBPs, e.g. bromate
Mature technology that is widely used	High power use
	Safety concerns when compared to other alternatives
	Corrosion
	Uncertainty of dosage

6.7.5 Ultraviolet Light

Ultraviolet (UV) light was discovered approximately 150 years ago, and the first commercial UV lamp was made in the early 1900s. Technological advances allowed its first application to water disinfection in 1907 in France, and the first UV system for municipal wastewater was installed in the mid-1970s. In the mid-1980s, the U.S. EPA named UV disinfection as a “best available technology” for wastewater disinfection and since then, the use of UV disinfection has grown within the wastewater industry, as an increasing number of communities have adopted UV irradiation in place of chlorination. Wastewater treatment plants are adopting UV due to the lack of disinfection byproducts, increased operator safety, and concern about meeting regulatory requirements.

6.7.5.1 System Components

In the general arrangement for UV disinfection, wastewater flows through a confined chamber/reactor containing arrays of UV lamps, and the UV radiation from the lamps inactivates microorganisms in the wastewater. A typical UV system consists of a power supply, an electrical system, reactors, lamps, quartz sleeves, a mechanical system to hold the lamps, and a control system. A UV intensity sensor and a cleaning system for periodical removal of fouling on the quartz sleeves may also be included.

UV systems can be classified by the hydraulic design of the UV reactor (closed vessels or open channels and high- or low-pressure lamps), the orientation of the lamps (horizontal, vertical, or inclined), and the arrangement of the lamps relative to the direction of flow (parallel versus perpendicular). Depending on plant flow rates and redundancy requirements, several reactors in series and/or parallel may be required. For open-channel UV systems, which are normally provided for disinfection of secondary effluent, the influent flow rate is usually divided equally among a number of open channels in parallel. Each channel typically contains two or more banks in series. An effluent flow control device such as a weighted flap gate, an extended sharp-crested weir, or an automatic level controller, is often used to regulate the submergence of the uppermost UV lamps in each bank.

6.7.5.2 Advantages and Disadvantages

UV is a mature technology that is well understood. It is widely used and permitted and should therefore be straightforward to have permitted for the WWTPs. The advantages and disadvantages of UV disinfection are summarized in **Table 6.7-3**.

Table 6.7-3 Advantages and Disadvantages of UV

Advantages	Disadvantages
No need to generate, handle transport or store toxic/hazardous chemicals	Micro-organisms can sometimes reverse the destructive effects of UV
No effect from disinfectant residuals on aquatic life	Large UVT fluctuations in effluent can be problematic for efficient UV operation
Shorter contact time	With no disinfection residual, regrowth could occur
Mature technology that is widely used	High energy requirement
Smaller footprint than sodium hypochlorite	Preventative maintenance to reduce fouling is a must

6.7.6 Initial Screening of Disinfection Technologies

As described in the technology review above, three of the five disinfection technologies were removed from consideration: chlorine dioxide, PAA, and ozone. These were removed for being too complex or too new to recommend and consider for implementation but may be reconsidered in final design if new advancements or additional installations make any of them more favorable. The two technologies carried forward for a more detailed evaluation specific to the WWTPs were sodium hypochlorite and UV.

6.7.7 Disinfection Design Criteria

In evaluating sodium hypochlorite and UV, it is important to understand that full plant alternatives will include options with various peak flow rates and different effluents either secondary effluent, primary effluent or a blended effluent so a mix of technologies may be warranted. At the West Side WWTP, the range of peak flow rates considered are between 90 mgd and 200 mgd. At the East Side WWTP, the range of peak flow rates considered are between 40 mgd and 80 mgd. The disinfection system must be capable of treating all flow up to the peak flow rate. This evaluation will consider the maximum flow rates at each plant, assuming similar evaluation results at lower peak flows.

6.7.7.1 Sodium Hypochlorite Design Criteria

As previously mentioned in the technology review for sodium hypochlorite, chlorine contact tanks will be sized to provide 30 minutes of contact time at average design flow. However, the tanks will only be sized for 15 minutes at wet weather flow. CDM Smith has experience with dose response at numerous CSO facilities which demonstrates that 15 minutes of contact time is sufficient in a properly designed tank. To ensure proper contact time in the tanks, a length to width ratio of at least 40 will be used in conjunction with multiple passes to prevent short circuiting and maximize the flow path.

To design the sodium hypochlorite feed system, the design dose will be set to the current average dose used at each plant. The current average dose at each plant is 4.0 mg/L at the West Side WWTP and 5.4 mg/L at the East Side WWTP. These doses are higher than most other treatment plants but will be used for sizing as conservative values. The feed systems should be capable of providing up to a maximum dose of 20 mg/L per TR-16. Storage tanks will provide at least 7 days of chemical at estimated maximum day usage. Storage of chemical for wet weather flows may be difficult as sodium hypochlorite is subject to decomposition. The half-life of sodium hypochlorite stored in 80º F at an initial concentration of 15% is approximately 65 days, (i.e. storing 15% hypo for 65 days will reduce the concentration to 7.5%).

6.7.7.2 UV Design Criteria

The design criteria for UV is very different from those established for sodium hypochlorite as the disinfection mechanisms vary greatly. Because UV light inactivates pathogens by destroying their genetic material, in order to predict the number of pathogens destroyed by a particular UV system, the dose of required UV radiation must be calculated. The dose is a function of the UV radiation intensity and the exposure time that wastewater is retained in the UV reactor. The equation used to calculate UV dose is shown below:

$$\text{UV Dose} = I \times t$$

Where: I = UV intensity, in milliwatts per square centimeter (mW/cm2)
t = exposure time, in seconds (s)
UV Dose, in mW-s/cm2 or milliJoules per square centimeter (mJ/cm2)

The actual UV intensity and exposure time are complex functions of the UV system, operating parameters and water quality. For example, in order to reach pathogens, the UV radiation must travel through the quartz sleeve, wastewater and particles (if the microbes are embedded in particles). Consequently, the UV intensity actually reaching the target organisms is lower than that at the surface of the UV lamp and varies throughout the reactor. There are a number of factors that can impact delivered UV dose including relevant water quality parameters.

Water quality affects the performance of a UV system by altering the UV intensity within the reactor and, consequently, the UV dose received by the organisms in the wastewater. The most important water quality parameters are the UV transmittance (UVT) of the water and the TSS concentration and particle size. In addition, dissolved solids may foul the quartz sleeves surrounding the lamps and decrease the effective UV output; therefore, an understanding of the water hardness, iron content and other dissolved organics in the wastewater is important to designing and evaluating a UV disinfection system.

UVT is defined as the percentage of UV light, at the 254 nm wavelength, not absorbed (i.e. transmitted) after passing through a 1-centimeter water sample. The UVT is one of the critical water quality parameters determining the UV intensity that will act on the microorganisms. As UV rays travel through wastewater, their intensity is attenuated continuously because some substances in wastewater absorb some of the UV light. The relationship between intensity and transmittance is directly proportional, i.e., the higher the transmittance the higher the intensity available.

TSS will absorb and scatter UV light, thus lowering the UVT. Similarly, the higher the TSS concentration, the higher the UV dose required. Additionally, the size of these solids highly affects the disinfection process. Large suspended solids have the capability of screening or shading the target microbes, preventing them from receiving their required UV dose.

Other water quality parameters, such as dissolved organics, total hardness and iron absorb UV light and affect UV intensity. Increased concentrations of these parameters can decrease UV intensity and the effectiveness of a UV disinfection system. High concentrations of dissolved organics have been shown to absorb UV light.

In addition to absorbing UV light, high iron concentrations affect the performance of UV disinfection systems by precipitating iron on the UV lamps, thus promoting lamp fouling. Also, increased concentrations of inorganic magnesium and calcium carbonates can increase fouling of the UV lamp quartz sleeves. Collimated beam tests conducted on effluent can contribute to developing log inactivation curves which can help establish a UV dose.

Since the effectiveness of a UV system is impacted by the water quality, the dose required for dry weather flows will differ from the dose required for wet weather flows that will only receive primary treatment. Multiple UV vendors were consulted to establish the UV doses for wet weather as shown in **Table 6.7-4**, but additional UVT and/or collimated beam testing should be conducted during preliminary design to confirm the doses if UV disinfection is recommended. Performing these tests during pilot testing of primary treatment technologies would be ideal for establishing UV dose.

Table 6.7-4 UV Dose for Dry and Wet Weather

	Dry Weather	Wet Weather
Minimum UV Transmittance	65%	50%
UV Dose	30-45 mJ/cm^2	45 mJ/cm^2
Maximum Suspended Solids	15 mg/L	15 mg/L

6.7.8 Disinfection Alternatives – Detailed Evaluation

This section presents more detailed alternative and the non-economic and economic evaluations of the most viable disinfection technologies previously identified at the maximum and minimum peak flow rates under consideration for each WWTP.

Three alternatives are presented for each WWTP at each flow rate:

1. Sodium hypochlorite for all flow
2. UV for all flow
3. Hybrid - UV for secondary treated flow and sodium hypochlorite for wet weather flow

6.7.8.1 Disinfection Alternatives – West Side WWTP – 200 MGD

The first alternative would treat all flow with sodium hypochlorite, matching what is performed today at the WWTP. This alternative would be simple and energy efficient (in comparison to UV); however, it requires a large footprint for the contact tanks and chemical storage would need to be managed carefully to ensure the sodium hypochlorite is used and does not degrade significantly when unused. **Table 6.7-5** presents a preliminary design for a 200 mgd sodium hypochlorite disinfection system.

Table 6.7-5 200 mgd Sodium Hypochlorite Design Summary

Design Element	Value
Average Design Flow, Contact Time = 30 min	30 mgd
Peak Design Flow, Contact Time = 15 min	200 mgd
Number of Tanks	3
Tank Width	13 feet
Tank Length	530 feet
Tank Depth	13 feet
Number of Passes per Tank	4
Total Footprint (All Tanks)	170’ x 150’
Number of Sodium Hypochlorite Chemical Storage Tanks	4
Sodium Hypochlorite Storage Tank Size	8,700 gallons
Sodium Hypochlorite Feed Rate at Average Flow	40 gallons per hour
Days of Storage at Average Flow	36
Number of Sodium Hypochlorite Pumps	4
Number of Sodium Bisulfite Chemical Storage Tanks	2
Sodium Bisulfite Storage Tank Size	2,000 gallons
Sodium Bisulfite Dose at Average Flow	4 gallons per hour
Days of Storage at Average Flow	41 days
Number of Sodium Bisultfite Pumps	2

The second alternative would treat all flow with UV. This design assumes an open channel, vertical lamp configuration. This system presents a significant footprint reduction when compared to the sodium hypochlorite alternative; however, it would require efficient primary treatment of wet weather flows to ensure UV treatment is not inhibited. Testing for UVT and/or collimated beam testing is recommended during design to confirm design elements and dose. **Table 6.7-6** presents a preliminary design for a 200 mgd UV disinfection system.

Table 6.7-6 200 mgd UV Design Summary

Design Element	Value
Average Design Flow	30 mgd
Peak Design Flow	200 mgd
UVT	50%
UV Dose	45 mJ/cm^2
Number of Channels	6
Channel Width	5 feet
Channel Depth	8 feet
Number of UV Modules per Channel	6
Total Footprint	90’ x 70’
Maximum Power Draw	1,010 kW

The third alternative would use a hybrid of the two technologies and treat the secondary effluent with UV and the bypassed, wet weather flow with sodium hypochlorite. This design assumes an open channel, vertical lamp configuration for the UV component. The benefit of this alternative is it utilizes UV in its most efficient application with treatment of secondary effluent flow and directs all wet weather flow to chlorine contact tanks where water quality is not as important to effective disinfection. **Table 6.7-7** presents a preliminary design for a 200 mgd hybrid disinfection system.

Table 6.7-7 200 mgd Hybrid Design Summary

Design Element	Value
UV System	
UV Average Design Flow	30 mgd
UV Peak (Secondary) Flow	58 mgd
UVT	65%
UV Dose	30-45 mJ/cm^2
Number of UV Channels	4
UV Channel Width	5 feet
UV Channel Depth	8 feet

Design Element	Value
Number of UV Modules per Channel	5
UV Total Footprint	60' x 50'
UV Maximum Power	240 kW
Sodium Hypochlorite System	
Peak Flow, Contact Time = 15 min	142 mgd
Number of Tanks	2
Tank Width	14 feet
Tank Length	540 feet
Tank Depth	14 feet
Number of Passes per Tank	4
Total Footprint (All Tanks)	150' x 120'
Number of Sodium Hypochlorite Chemical Storage Tanks	3
Sodium Hypochlorite Storage Tank Size	8,700 gallons
Number of Sodium Hypochlorite Pumps	4
Number of Sodium Bisulfite Chemical Storage Tanks	2
Sodium Bisulfite Storage Tank Size	5,400 gallons
Number of Sodium Bisulfite Pumps	2

6.7.8.2 Non-Economic Evaluation – West Side WWTP

Table 6.7-8 presents the ranking for each of the non-economic criteria for the three disinfection alternatives at the West Side WWTP. A ranking of 5 indicates the most favorable rating, while a ranking of 1 indicates least favorable.

Table 6.7-8 Non-Economic Disinfection Alternative Rankings

Rating Legend 5= favorable 3= neutral 1= unfavorable	Weight	Sodium Hypochlorite	UV	Hybrid
Success at Other Installations/Reliability	8%	5	5	5
Site Utilization	10%	1	5	3
Maintenance of Plant Operations	2%	5	5	3
Ease of Operations	3%	3	3	1
Ease of Maintenance	3%	3	3	1
Neighborhood Impacts	4%	3	3	3
Ability to Phase Implementation	2%	3	3	3
Sludge Impacts	2%	3	3	3
Energy Efficiency	4%	5	1	3
Chemical Handling/Hazards	2%	1	5	3
Non-Economic Total Weighted Score	**40%**	**24.8**	**31.2**	**24.8**

Table 6.7-9 includes the detailed evaluation used to determine the ranking for each of the alternatives. Because site utilization includes a 10% weighting in the non-economic rankings, this component is of great importance if trying to keep disinfection within the existing plant property boundaries. Selecting a sodium hypochlorite system would require a very large footprint and it is anticipated that this cannot fit on the site with other required treatment processes. Therefore, this alternative has been eliminated from consideration. The UV and hybrid alternatives were brought forward to be evaluated on an economic basis.

6.7.8.3 Economic Evaluation – West Side WWTP – 200 MGD

This section presents further evaluation of the disinfection alternatives on an economic basis, including planning-level cost estimates for capital cost, annual O&M cost, and 20-year life cycle cost.

Economic Evaluation Assumptions

Opinions of probable construction cost (OPCCs) were developed in order to assess the differences in lifecycle costs between the two alternatives; they include contractor's overhead and profit (OH&P), construction contingency, and escalation to the midpoint of construction. The OPCCs established for each alternative include allowances for site remediation and disposal of materials likely to be encountered during construction of the new facilities, based on site investigations previously conducted. The costs also include other site work allowances and demolition.

The following list includes a summary of the major assumptions that were common to each annual O&M estimate. Specific items related to each system are defined later in this section.

- All costs were calculated on an annual average basis, assuming an average daily flow of 30 mgd.
- All costs associated with chemical addition are based on 365 days of operation.
- All costs associated with the UV energy consumption assumes 365 days of operation.
- Replacement parts and maintenance time was estimated using values obtained from manufacturers and previous experience in evaluating disinfection options.

Cost Comparison

Table 6.7-10 presents the total opinion of probable construction cost (OPCC) for the main components associated with each disinfection alternative, the O&M costs, and the life cycle cost (as present worth). The OPCCs for this section include construction contingency but do not include project contingency or engineering costs. The present worth was calculated using the methodology described in Section 2.

This page intentionally left blank.

Table 6.7-9 Disinfection Alternative Non-Economic Evaluation

Alternative Process Description / Non-Economic Criteria	Sodium Hypochlorite Disinfection West Side WWTP - 200 mgd	UV Disinfection West Side WWTP - 200 mgd	Hybrid Disinfection UV - Secondary Effluent Sodium Hypochlorite - Wet Weather West Side WWTP - 200 mgd
Success at Other Installations/ Reliability	Technology has been successfully implemented at other large-scale facilities and is commonly used for CSO facilities. Disinfection mechanism is well understood and used currently at the WWTP.	Technology has been successfully implemented at other large-scale facilities and is used at CSO facilities with higher dose. UV is becoming the new standard for wastewater disinfection.	Both technologies have been successfully implemented at other large-scale facilities. Implementing a hybrid approach will utilize both in an efficient way.
Site Utilization	Largest process footprint. Contact tanks would take up a lot of site space, more than the current contact tanks. Likely would need to construct on adjacent, undeveloped land.	Smallest process footprint. Smaller than existing chlorine contact tanks and can be implemented within the existing site boundaries. Allows for construction or expansion of other facilities.	Having two systems would be less space efficient than one, but could likely be constructed within the existing site boundaries.
Maintenance of Plant Operations	Given the size of this system, the contact tanks would be constructed in adjacent, undeveloped land. This would be easier for construction but would require additional land.	UV could be installed in a location of a previous process which would allow for demolition of existing contact tanks and construction of new facilities in their place.	Constructing two facilities could be complicated depending on the final sequence. The UV system would likely be constructed first in a location of a previous process which would allow for demolition of existing contact tanks and construction of new chlorine contact tanks. The negative to this option is for a period of time, disinfection may be limited to only secondary effluent. Alternatively, the UV system could be oversized to 90 mgd to meet the current peak flow and accommodate disinfection during wet weather events.
Ease of Operations	Operation would be similar to operations today. Management of chemicals may require more attention as more chemical may be needed on site to accommodate increased peak flows.	Operation would be fairly straightforward. The system would be flow paced to automatically open gates and turn on additional lamp modules as needed to provide proper disinfection. Process could be automated to increase dosage when plant is bypassing during wet weather events.	See descriptions for individual systems for individual details. Operation may become more complicated with two systems especially if a flow split is necessary. Chemical management will be important as sodium hypochlorite may sit idle and degrade when not in use.
Ease of Maintenance	Maintenance would be similar to what the operators experience currently.	Maintenance would be different from current operations as a UV system would require cleaning and replacement of UV lamps. However, it is assumed that this would be approximately the same level of effort as maintaining a sodium hypochlorite system.	Maintaining two systems would likely require more effort than maintaining one system.
Neighborhood Impacts	Neighborhood impacts would be limited to truck deliveries of chemicals. This was considered a minor impact.	This alternative does not have any known neighborhood impacts.	Neighborhood impacts would be limited to truck deliveries of chemicals. This was considered a minor impact.
Ability to Phase Implementation	Because the system is flow paced, changes in flow would have little impact on the disinfection process.	Because the system is flow paced, changes in flow would have little impact on the disinfection process.	Because both systems are flow paced, changes in flow would have little impact on the disinfection process.
Sludge Impacts	This factor is not applicable to disinfection.	This factor is not applicable to disinfection.	This factor is not applicable to disinfection.
Energy Efficiency	This is the most energy efficient alternative as the only power consumption is for the chemical pumps.	UV is the least energy efficient alternative as the UV lamps are powered on electricity. Power draw can be high during wet weather flows.	This option would still use energy for the UV system, but only treating dry weather flows would be more efficient as the UV light would be more effective at the higher UVT levels.
Chemical Handling/Hazards	Both sodium hypochlorite and sodium bisulfite would be required for this alternative.	No chemicals would be required, but UV light can be a hazard to operators. Training and safety protocols are effective at mitigating this risk.	Both sodium hypochlorite and sodium bisulfite would be required for this alternative.

This page intentionally left blank.

Table 6.7-10 Estimated Costs for Disinfection Alternatives

	UV Disinfection	Hybrid Disinfection UV – Secondary Effluent Sodium Hypochlorite – Wet Weather
	West Side WWTP – 200 mgd	West Side WWTP – 200 mgd
Total OPCC	$15,300,000	$14,300,000
Annual O&M Cost Estimate	$250,000	$230,000
Present Worth of 20-year Life Cycle Costs[1]	**$20,000,000**	**$18,600,000**

[1]See Section 2 for present worth formulas.

6.7.8.4 Disinfection Alternatives Overall Evaluation – West Side WWTP – 200 MGD

Table 6.7-11 presents the non-economic weighted scores (from **Table 6.7-8**) and the economic rankings and weighted scores (based on the costs from **Table 6.7-10**) for the two disinfection alternatives. The cost component of the evaluation consists of a maximum score of 60 points. The lowest present-worth 20-year lifecycle cost that meets all of the project objectives scores the maximum 60 points. Each alternative that exceeds the lowest cost scores a percentage of the maximum points equal to the incremental difference between its cost and the lowest cost.

Table 6.7-11 Overall (Economic and Non-Economic) Evaluation of Disinfection Alternatives

Criteria	Weight	UV Disinfection	Hybrid Disinfection UV – Secondary Effluent Sodium Hypochlorite – Wet Weather
		West Side WWTP – 200 mgd	West Side WWTP – 200 mgd
Weighted Non-Economic Score	40%	31.2	24.8
Weighted Economic Score	*60%*	56	60
Overall Evaluation Score	**100%**	**87.2**	**84.8**

The UV alternative received the highest non-economic score due to a combination of site utilization, no chemical usage, and ease of operability and maintenance due to only having one system. The economic scores were very close and therefore, the overall evaluation was primarily driven by the non-economic score making UV the highest scoring alternative. Thus, implementation of the UV system at the West Side WWTP is recommended for full plant alternatives dependent on confirmation of UV dose and treatment of wet weather flows. CDM Smith is confident this can be accomplished, but further testing is required to confirm the UV dose and design criteria. If UV cannot be designed for treatment of all flow, then the hybrid alternative should be pursued to utilize sodium hypochlorite for wet weather disinfection.

6.7.8.5 Disinfection Alternatives – East Side WWTP – 80 MGD

The first alternative would treat all flow with sodium hypochlorite, matching what is performed today at the WWTP. This alternative would be simple and energy efficient (in comparison to UV); however, it requires a large footprint for the contact tanks and chemical storage would need to be managed carefully to ensure the sodium hypochlorite is used and does not degrade significantly when unused. **Table 6.7-12** presents a preliminary design for an 80 mgd sodium hypochlorite disinfection system.

Table 6.7-12 80 mgd Sodium Hypochlorite Design Summary

Design Element	Value
Average Design Flow, Contact Time = 30 min	10 mgd
Peak Design Flow, Contact Time = 15 min	80 mgd
Number of Tanks	3
Tank Width	10 feet
Tank Length	390 feet
Tank Depth	10 feet
Number of Passes per Tank	3
Total Footprint (All Tanks)	140’ x 100’
Number of Sodium Hypochlorite Chemical Storage Tanks	2
Sodium Hypochlorite Storage Tank Size	8,700
Sodium Hypochlorite Feed Rate at Average Flow	18 gallons per hour
Days of Storage at Average Flow	40
Number of Sodium Hypochlorite Pumps	4
Number of Sodium Bisulfite Chemical Storage Tanks	2
Sodium Bisulfite Storage Tank Size	750
Sodium Bisulfite Feed Rate at Average Flow	1.3 gallons per hour
Days of Storage at Average Flow	45
Number of Sodium Bisulfite Pumps	2

The second alternative would treat all flow with UV. This design assumes an open channel, vertical lamp configuration. This system presents a significant footprint reduction when compared to the sodium hypochlorite alternative; however, it would require efficient primary treatment of wet weather flows to ensure UV treatment is not inhibited. Testing for UVT and/or collimated beam testing is recommended during design to confirm design elements and dose. **Table 6.7-13** presents a preliminary design for an 80 mgd UV disinfection system.

Table 6.7-13 80 mgd UV Design Summary

Design Element	Value
Average Design Flow	10 mgd
Peak Design Flow	80 mgd
UVT	50%
UV Dose	45 mJ/cm^2
Number of Channels	3

Design Element	Value
Channel Width	5 feet
Channel Depth	8 feet
Number of UV Modules per Channel	6
Total Footprint	70’ x 50’
Maximum Power Draw	450 kW

The third alternative would use a hybrid of the two technologies and treat the secondary effluent with UV and the bypassed, wet weather flow with sodium hypochlorite. This design assumes an open channel, vertical lamp configuration for the UV component. The benefit of this alternative is it utilizes UV in its most efficient application with treatment of secondary effluent flow and directs all wet weather flow to chlorine contact tanks where water quality is not as important to effective disinfection. **Table 6.7-14** presents a preliminary design for an 80 mgd hybrid disinfection system.

Table 6.7-14 80 mgd Hybrid Design Summary

Design Element	Value
UV System	
UV Average Design Flow	10 mgd
UV Peak (Secondary) Flow	24 mgd
UVT	65%
UV Dose	30-45 mJ/cm^2
Number of UV Channels	2
UV Channel Width	4 feet
UV Channel Depth	8 feet
Number of UV Modules per Channel	4
UV Total Footprint	60’ x 30’
UV Maximum Power	100 kW
Sodium Hypochlorite System	
Peak Flow, Contact Time = 15 min	56 mgd
Number of Tanks	2
Tank Width	10 feet
Tank Length	400 feet
Tank Depth	10 feet
Number of Passes per Tank	3
Total Footprint (All Tanks)	140’ x 70’
Number of Sodium Hypochlorite Chemical Storage Tanks	2

Design Element	Value
Sodium Hypochlorite Storage Tank Size	5,400 gallons
Number of Sodium Hypochlorite Pumps	2
Number of Sodium Bisulfite Chemical Storage Tanks	2
Sodium Bisulfite Storage Tank Size	750
Number of Sodium Hypochlorite Pumps	2

6.7.8.6 Non-Economic Evaluation – East Side WWTP

The non-economic evaluation for the East Side WWTP is identical to that used for the West Side WWTP. The ratings are presented in **Table 6.7-8** with a detailed evaluation with descriptions in **Table 6.7-9**. Similar to the West Side WWTP, the site utilization component is of great importance if trying to keep disinfection within the existing plant property boundaries. Selecting a sodium hypochlorite system would require a very large footprint and it is anticipated that this cannot fit on the site with other required treatment processes. Therefore, this alternative has been eliminated from consideration. The UV and hybrid alternatives were brought forward to be evaluated on an economic basis.

6.7.8.7 Economic Evaluation – East Side WWTP – 80 MGD

This section presents further evaluation of the disinfection alternatives on an economic basis, including planning-level cost estimates for capital cost, annual O&M cost, and 20-year life cycle cost.

Economic Evaluation Assumptions

OPCCs were developed in order to assess the differences in lifecycle costs between the two alternatives; they include contractor's OH&P, construction contingency, and escalation to the midpoint of construction. The OPCCs established for each alternative include allowances for site remediation and disposal of materials likely to be encountered during construction of the new facilities, based on site investigations previously conducted. The costs also include other site work allowances and demolition.

The following list includes a summary of the major assumptions that were common to each annual O&M estimate. Specific items related to each system are defined later in this section.

- All costs were calculated on an annual average basis, assuming an average daily flow of 10 mgd.
- All costs associated with chemical addition are based on 365 days of operation.
- All costs associated with the UV energy consumption assumes 365 days of operation.
- Replacement parts and maintenance time was estimated using values obtained from manufacturers and previous experience in evaluating disinfection options.

Cost Comparison

Table 6.7-15 presents the total OPCC for the main components associated with each disinfection alternative, the O&M costs, and the life cycle cost (as present worth). The OPCCs for this section include construction contingency but do not include project contingency or engineering costs. The present worth was calculated using the methodology described in Section 2.

Table 6.7-15 Estimated Costs for Disinfection Alternatives

	UV Disinfection	Hybrid Disinfection UV – Secondary Effluent Sodium Hypochlorite – Wet Weather
	East Side WWTP – 80 mgd	East Side WWTP – 80 mgd
Total OPCC	$8,100,000	$8,400,000
Annual O&M Cost Estimate	$110,000	$95,000
Present Worth of 20-year Life Cycle Costs[1]	**$10,510,000**	**$10,200,000**

[1]See Section 2 for present worth formulas.

6.7.8.8 Disinfection Alternatives Overall Evaluation – East Side WWTP – 80 MGD

Table 6.7-16 presents the non-economic weighted scores (from **Table 6.7-8**) and the economic rankings and weighted scores (based on the costs from **Table 6.7-15**) for the two disinfection nitrogen removal alternatives. The cost component of the evaluation consists of a maximum score of 60 points. The lowest present-worth 20-year lifecycle cost that meets all of the project objectives scores the maximum 60 points. Each alternative that exceeds the lowest cost scores a percentage of the maximum points equal to the incremental difference between its cost and the lowest cost.

Table 6.7-16 Overall (Economic and Non-Economic) Evaluation of Disinfection6.11- Alternatives

Criteria	Weight	UV Disinfection	Hybrid Disinfection UV – Secondary Effluent Sodium Hypochlorite – Wet Weather
		East Side WWTP – 80 mgd	East Side WWTP – 80 mgd
Weighted Non-Economic Score	*40%*	31.2	24.8
Weighted Economic Score	*60%*	60	59
Overall Evaluation Score	**100%**	**91.2**	**83.8**

The UV alternative received the highest non-economic score due to a combination of site utilization, no chemical usage, and ease of operability and maintenance due to only having one system. The economic scores were very close and therefore, the overall evaluation was primarily driven by the non-economic score making UV the highest scoring alternative. Thus, implementation of the UV system at the East Side WWTP is recommended for full plant alternatives dependent on confirmation of UV dose and treatment of wet weather flows. CDM Smith is confident this can be accomplished, but further testing is required to confirm the UV dose and design criteria. If UV cannot be designed for treatment of all flow, then the hybrid alternative should be pursued to utilize sodium hypochlorite for wet weather disinfection.

6.8 Reuse

As part of the treatment plant upgrade, the plant water system will be replaced with a new system to provide non-potable water to various uses around the plant site, including washdown stations in the sludge processing area and around process tanks, motive water as necessary for chemical systems for odor control and/or disinfection, process water required for flushing and/or fluidization and pump seal water. Depending on the extent of improvements, other uses that could be considered on-site include, toilet flushing, irrigation for landscaping, and a potential water feature on the site. The size of the plant water system at each plant for conventional uses will be assessed in preliminary design.

There may also be opportunities for non-potable water uses off site, in the area proximate to the plant for instance at the adjacent boat yard, the aquaculture school, and the asphalt plant across Bostwick Ave. Effluent criteria for off-site uses would likely require a higher level of treatment than required for discharge to the harbor, however, the WPCA could charge for this water use to recoup additional costs for treating to a higher level. If membrane filtration is incorporated the final effluent quality will likely meet reuse standards.

There also may be an opportunity to provide dual use of the infrastructure added for primary treatment. That is, the high rate clarification system, or the cloth filtration system could be used during lower flow periods to polish secondary effluent and provide reuse quality water. If the WPCA views this as a viable option, and there are known entities who would be interested in non-potable water from the plant, the feasibility of implementing such flexibility will be assessed in preliminary design.

6.9 Effluent Pumping Station

6.9.1 Introduction

The West Side and East Side WWTPs currently discharge by gravity from the chlorine contact tanks out through the outfall pipe to Cedar Creek and the Bridgeport Harbor, respectively. According to the hydraulic profile in the record drawings of the existing plants, the chlorine contact tank weir is just at or below the 25-year storm high water elevation, which would result in plant back up when the tidal elevation exceeds that level. Current regulatory guidance requires uninterrupted operation of the treatment works during the 100-year flood event. This will require pumping of final effluent to enable the plant to pass flow.

In recent years, the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM) have been updated based on more detailed survey information and land use resulting in revisions to the various flood level elevations (i.e. 25-year flood, 100-year flood, 500-year flood). Additionally, for the protection of critical equipment and infrastructure, the CT DEEP is now requiring that the plants discharge against a receiving water equal to the 100-year flood elevation.

Given existing and proposed plant hydraulics, as well as the desire to increase the peak flow through the existing plants, it became evident that gravity flow of the plant effluent under all conditions would not be possible.

As a result of the combination of these various factors, effluent pump stations are necessary at both plants. The stations will be not be required to operate on a continuous basis, as gravity discharge will be possible for the majority of the average and even elevated flows at all but the highest of the receiving water elevations. However, when there is a significantly elevated receiving water elevations, effluent pumping will be required for all flows. The following presents an approach to the effluent pump station along with a discussion of the various types of potential pumps.

6.9.2 Design Approach/Criteria

The West Side and East Side WWTP effluent pumping stations will only require operation during significant storm and/or flooding events. The approach to the design and layout of the effluent pumping stations will be such that they can be easily bypassed during normal plant flows and operations. Bypassing the station will reduce normal gravity flow head losses to increase the time when the plant can discharge by gravity, allow for the station wet well to be drained to allow for inspection and maintenance, and will keep any pumps, equipment or piping from being permanently submerged.

Under normal flows when the station is not needed, the effluent flow will pass through or around the effluent pumping station structure, bypassing the station wet well. When the receiving water level increases to point where gravity flow is no longer possible, effluent flow will then be diverted into the pumping station wet well. The pumps will lift the flow and the effluent be will be reintroduced back into the final effluent outfall conduit. A series of gates and flap valves will be installed in the wet well and the associated inlet and outlet channels/conduits to direct flow as needed under the two operational scenarios. **Figure 6.9-1** below presents a design of an effluent pumping station with a similar gravity flow bypass approach.

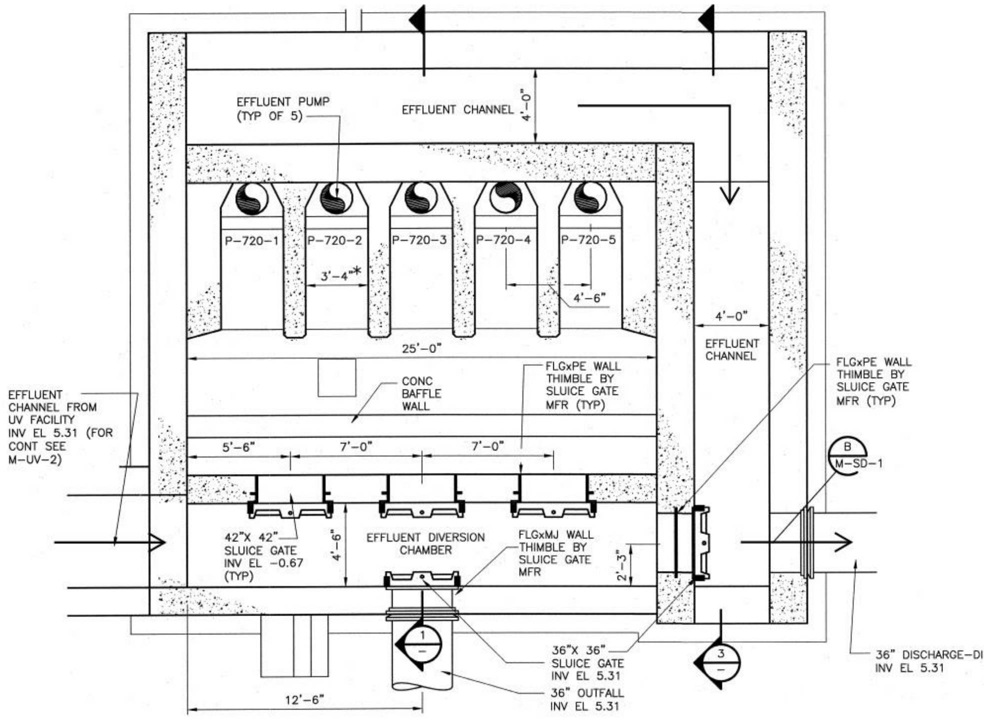


Figure 6.9-1
Example Effluent Pumping Station with Bypass

Each station will be designed to pump the full peak flow, dry and wet weather. For the West Side WWTP, the station would be a 90 mgd to 200 mgd station, and for the East Side WWTP, it would be 40 mgd to 80 mgd station, depending on the ultimate peak flow recommended for each plant. To get a rough estimate of required discharge head for the proposed pumps, preliminary hydraulic calculations were performed assuming the highest peak flows. These initial computations indicate that the stations would be required to deliver a total lift of approximately 20 to 25 feet to overcome the necessary static lift and the associated station and pipeline losses.

The approach for the outfall pipes from the effluent pumping stations to the final discharge points will be to utilize existing piping to the extent possible, provided it is capable of handling the pressure associated with a pumped discharge. However, sections of new outfall piping will be required at both plants in order to integrate the new effluent pumping station into the existing piping, and to route flow to and from the location of the new pumping station. Where existing piping is to be reused, any existing junction boxes or manholes along the piping will be evaluated with respect to the expected pressure in the outfall to determine if they can remain, if they need to be modified with sealed/bolted manholes or accesses, if they need to removed, etc.

Station Location

The effluent pumping stations will be the final process in the plant flow stream. This allows for all the treatment processes to flow similarly regardless of whether or not the plants require effluent pumping. Locating processes downstream of the effluent pumping station would not be possible as the hydraulic grade line at the effluent end of the pumping station will vary depending on whether or not the station is in operation.

Pump Quantity

The West Side and East Side WWTPs will both have wide ranges of flow in the future regardless of the peak flow that is designed around. The West Side plant flow could range from 30 mgd ADF up 200 mgd peak, while the East Side plant flow could range from 10 mgd ADF up 80 mgd peak. With the expected range of flows for both effluent stations, multiple pumps will be required at each station to accommodate the range of flows.

Similar to the influent pumping station, one approach would be to provide a quantity of pumps that are of the same size. For example, for the West Side plant, a 200 mgd peak flow could have seven 33.3 mgd pumps in a six plus one arrangement. 33.3 mgd pumps would be on the order of 215-250 hp based on conceptual estimates for anticipated discharge head/pressure. Similarly, for the East Side plant, this could be six 16-mgd pumps for the 80 mgd flow in a five duty plus one standby pump arrangement. 16 mgd pumps would be on the order of 100 hp based on conceptual estimates for anticipated discharge head/pressure.

The second alternative for pump quantity and size would be to provide pumps of different capacities/sizes. Similar to the influent pumping station, the driver for this approach would be to provide a pump(s) sized specifically to accommodate the typical daily flows to improve daily pump operation and efficiency. The balance of the required pumps would then be sized to accommodate the less frequent high/peak flows. The station standby pump would be the size of the larger pump so that the station would not lose any capacity if any one pump was out of service. However, if the stations are not anticipated to see the minimum daily flows, as minimum plant flows occurring during a 100-year flood would be highly unlikely, then the use of varying size pumps may not be required.

Table 6.9-1 below represents summary of potential pump scenarios for the West Side and East Side WWTPs.

Table 6.9-1 Pump Size/Quantity Example Scenarios

Parameter	West Side WWTP 90 mgd	West Side WWTP 200 mgd	East Side WWTP 40 mgd	East Side WWTP 80 mgd
Equal Size Pumps	3+1 @ 30 mgd	6+1 @ 33.33 mgd	4+1 @ 10 mgd	5+1 @ 16 mgd
Varying Size Pumps	3 @ 20 mgd 1+1 @ 30 mgd	3 @ 20 mgd 3+1 @ 47.5 mgd	N/A	2@ 20 mgd 3+1 @ 20 mgd

The size and quantity of the pumps at each of the two new pump stations will be determined during final design when final facility layout is set, and the final plant hydraulics are calculated. At that time, the required discharge head of the pump station can be accurately determined. The design team will also review historic and projected flow data in more detail to better understand the frequency of average, minimum, and peak flows. The design team will then work with pump vendors to determine a pump quantity/size arrangement that can best address average flows, minimum flows, and peak flows taking into account pump efficiencies, best operating points, and turndown.

6.9.3 Pump Alternatives

The potential pump technologies for the effluent pump station are similar to the options for the influent pumping station and include:

- Axial flow pumps
- Centrifugal pumps (dry-pit direct coupled, dry-pit extended shaft, dry-pit submersible, and wet-pit submersible)
- Archimedes screw type pumps.

See section 6.4 for a detailed description of these pumps. A summary of these types of pumps is presented again in **Table 6.9-2**.

6.9.4 Wet Well Alternatives

There are two main styles of pump station wet wells that are applicable for the size effluent pumping stations required at the West Side and East Side WWTPs. They are long trench style wet wells and rectangular wet wells. See Section 6.4 for detailed descriptions of these types of wet wells.

6.9.5 Pump Evaluation

A summary of the potential pumps and their advantages and disadvantage is presented in **Table 6.9-2**.

While Archimedes screw pumps are efficient and relatively “simple” with respect to operation and control, and they would be applicable for the high capacity/low lift nature of the proposed effluent stations, they require much larger footprints when compared to similar capacity rotodynamic dry pit and wet pit pump stations. Therefore, Archimedes screw pumps stations are not applicable for either the West Side or East Side WWTP effluent pump stations based on the limited footprint available for siting these new facilities.

Axial flow pumps are efficient at pumping large quantities of flow at lower discharge heads/pressure, less than 40 feet. Based on the preliminary estimate of pump station discharge needs (approximately 20 to 25 feet), axial flow pumps are applicable for either the West Side or East Side WWTP influent pump station. Preliminary inquiries with pump vendors indicate that there are available pump selections that make these types of pumps very viable options.

With their wide range of flow capacities and discharge head capabilities, all of the dry-pit pump options are applicable for consideration at the West Side and East Side WWTPs.

Similar to the dry-pit centrifugal pumps, the wet-pit submersible pumps offer a wide range of flows and discharge head capabilities and there are available pump selections for the range of proposed flows at the West Side and East Side WWTPs. They also offer a pump station with a reduced footprint as there is no station dry-pit, and advantage over the dry-pit alternatives.

Table 6.9-2 Pump Alternatives Analysis

Pump Type	Motor Arrangement	Advantages	Disadvantages
Archimedes Screw Pump (Open or Closed)	Geared motor	• No variable-frequency drives (VFDs) required, easiest to control • Can run dry, no NPSH requirements • Pumps are less affected by changes in wet well (inlet) water surface level	• Large pump footprint • Large and heavy assemblies requiring specialized maintenance • Maintenance accessibility issues • Not as efficient as other pumps, always lifting to high elevation • Aboveground discharge channel required • Cannot discharge to a header assembly • Can have maintenance issues with bearings
Axial Flow	Line Shaft Motor	• Pump operation is less affected by discharge conditions when set up in an open discharge • High capacity per pump - efficient with reduced power/motor size • Motor is accessible without pulling pump • Easier maintenance than submerged options • Can meet the required hydraulic conditions	• Limited discharge pressures and greater submergence required • Additional access required for motor maintenance • More sensitive to static head changes than centrifugal pumps when installed in hard-piped application • VFDs required
	Submersible	• Pump operation is less affected by discharge conditions when set up in an open discharge • High capacity per pump - efficient with reduced power/motor size • Reduced structure footprint	• Limited discharge pressures and greater submergence required • Pump must be removed for motor maintenance and inspection • More sensitive to static head changes than centrifugal pumps when installed in hard-piped application • Pump/motor and seal repair and maintenance is labor intensive and may require work at specialty shop • VFDs required
Centrifugal Volute Pump	Wet-Pit Submersible	• Reduced structure footprint - elimination of pump dry pit • Lower power costs • Higher discharge heads compared to axial flow and screw pumps	• Maintenance requires pump removal from wet pit • Seals can be more difficult to maintain • Unable to visually inspect pumps and discharge piping • Pump/motor repair and maintenance is labor intensive and may require work at specialty shop • Reapir to discharge piping requires half the staiton to be offline • Permanently submerged and susceptible to water intrusion • Seal failure could lead to severe damage to motor and/or cooling system • VFDs required
	Dry-Pit Submersible	• Easier accessibility for inspection • Lower power costs • Higher discharge heads compared to axial flow and screw pumps • Submersible capability protects the pumps and station against flooding of the dry-pit	• Motor cooling required (open or closed loop cooling system) • Requires additional subsurface structure for dry pit • Seals and leak sensors can be more difficult to maintain • Pump, motor, seal repair and maintenance is labor intensive and may require work at specialty shop • Seal failure could lead to severe damage to motor and/or cooling system • VFDs required
	Dry-Pit Vertical Direct Coupled	• Pumps and motor located in dry gallery - easier inspection • Standard electrical motors, no internal cooling system • Easier maintenance than wet-pit and dry-pit submersible options • Lower power costs • Higher discharge heads compared to mixed/axial flow pumps • Motor is accessible without pulling pump • N long extended shafts	• Requires additional sub surface structure for dry pit, larger footprint • Susceptible to catestrophic damage if dry-pit floods from pump, piping, or wet well wall failure • VFDs required
	Dry-Pit Vertical Extended Shaft	• Pumps and motor located in dry gallery - easier inspection • Less susceptible to damage from partial dry-pit flood - motor located above grade and above flood elevation • Easier maintenance than wet-pit and dry-pit submersible options • Lower power costs • Higher discharge heads compared to mixed and axial flow pumps • Motor is accessible at grade without pulling pump	• Requires additional sub surface structure for dry pit • Requires additional level in structure for motor mounting • Proper support of shaft is required • Issues with shaft alignment or vibration could lead to premature failure of pump, shaft, seals, or bearings leading to leaks • VFDs required

This page intentionally left blank.

This page intentionally left blank.

6.9.6 Summary and Recommendation

The axial flow pumps, dry pit centrifugal, and submersible wet pit centrifugal pumps are viable options for the effluent pump stations at both facilities. The West Side WWTP will be required to intermittently convey up to 90 to 200 mgd, while the East Side WWTP will be required to convey up to 40 to 80 mgd intermittently. Based on the current 100-year flood water surface elevations that these stations will be required to pump against, the preliminary hydraulic profiles for the upgraded plants for the highest potential flows (200 mgd and 80 mgd), and an assumption of pump station head losses, both stations will be required to provide a total effluent lift of approximately 15 to 25 feet.

With high flow, low head pumping conditions, axial flow pumps are the recommended approach for the effluent pump stations at both the West Side and East Side WWTPs. This type of pumping condition is one of the main uses for these types of pumps. While dry pit or wet pit submersible pumps could be utilized as they are capable of these ranges of flow, they tend to be less efficient at the anticipated low head design points. It is recommended that submersible pumps installed in the column be provided. The main advantage with this approach is that it will minimize the size and scope of the building above the pumps, as opposed to extended shaft axial flow pumps with motors installed on top of the pump column. With submersible pumps, it is also possible to completely eliminate a building above the pump columns, and simply erect an exterior monorail or bridge crane above the columns to remove the pumps.

Wet Well

Axial flow pumps can be installed in either trench style or rectangular shaped wet wells. Given the likelihood of integrating the effluent pump station into or directly adjacent to the proposed disinfection process, the need to provide a bypass around the station wet well for normal dry weather flows, and the uncertainty at this time as to exactly how these two processes will be situated and laid out on the site, conceptual level effluent pumping station footprint requirements and costs are based on a rectangular wet well approach as a more "square" facility may be easier to accommodate than a longer, narrower facility. This approach will be reviewed during final design when the detailed layout of disinfection facility and effluent pumping station structure, and their placement on the site, are further advanced to confirm that this design intent is the optimal solution.

6.10 Effluent Outfall

This section discusses alternatives for discharge of the treated effluent from the WPCA's West Side and East Side WWTPs.

6.10.1 Introduction

Several outfall alternatives were investigated for a variety of flow rates from the WPCA's West Side and East Side WWTPs. **Figure 6.10-1** shows the alternatives that were initially considered.

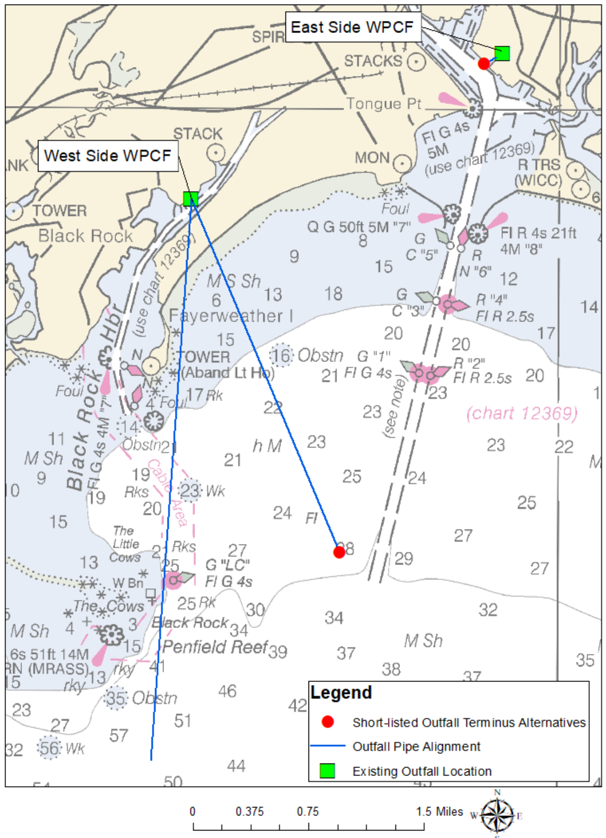


Figure 6.10-1
Existing and Candidate Alternative Locations for Discharges from WPCA's West Side and East Side WWTP

West Side plant options included:

- No action – retaining the existing outfall with the current day peak discharge of 90 mgd
- Consider the following set of peak flows (200, 180, and 140 mgd) and the following discharge locations:
  - Use the existing outfall as is
  - Inspect, clean and consider rehabilitation of the existing outfall
  - Move outfall offshore to about 28-ft deep water MLW west of the terminus of the dredged channel
  - Move outfall farther offshore to a water depth more than 50-ft deep water MLW south of Penfield Reef.

Regarding moving the outfall from the West Side plant offshore, preliminary initial dilution model simulations indicated that the dilution available at the location west of the terminus of the dredge would likely be sufficient to address any water quality concerns, and thus, the location south of Penfield Reef was dropped from any further consideration.

East Side plant options included:

- No action – retaining the existing outfall with the current day peak discharge of 40 mgd
- Increase the peak flow to 80 mgd
  - Use the existing outfall as is
  - Inspect, clean, and consider rehabilitation of existing outfall
  - Move outfall to the seawall facing Bridgeport Inner Harbor

Table 6.10-1 summarizes the candidate outfall sites that were short-listed for analysis.

Table 6.10-1 Short-listed Outfall Alternatives for WPCA's East Side and West Side plant Discharges

	West Side plant	East Side plant
Existing location - No action	X	X
Existing location - Inspect, clean and, where needed, rehab of existing pipes	X	X
Discharge into Bridgeport Inner Harbor from seawall adjacent to the East Side plant		X
Discharge into the Outer Harbor to the west of the entrance to the main shipping channel	X	

6.10.2 Description of Alternatives

6.10.2.1 No Action

In the no action alternatives for both the West Side and East Side WWTPs, the discharge would continue to be through each plant's existing outfall pipe.

The West plant's outfall discharges through a 72-inch pipe at a headwall along the north side of Cedar Creek in Black Rock Harbor near the Captains Cove Seaport restaurant and amongst docks for pleasure craft (**Figure 6.10-2**). The outfall was constructed in 1948, and its existing condition is discussed in Section 4.2.6.

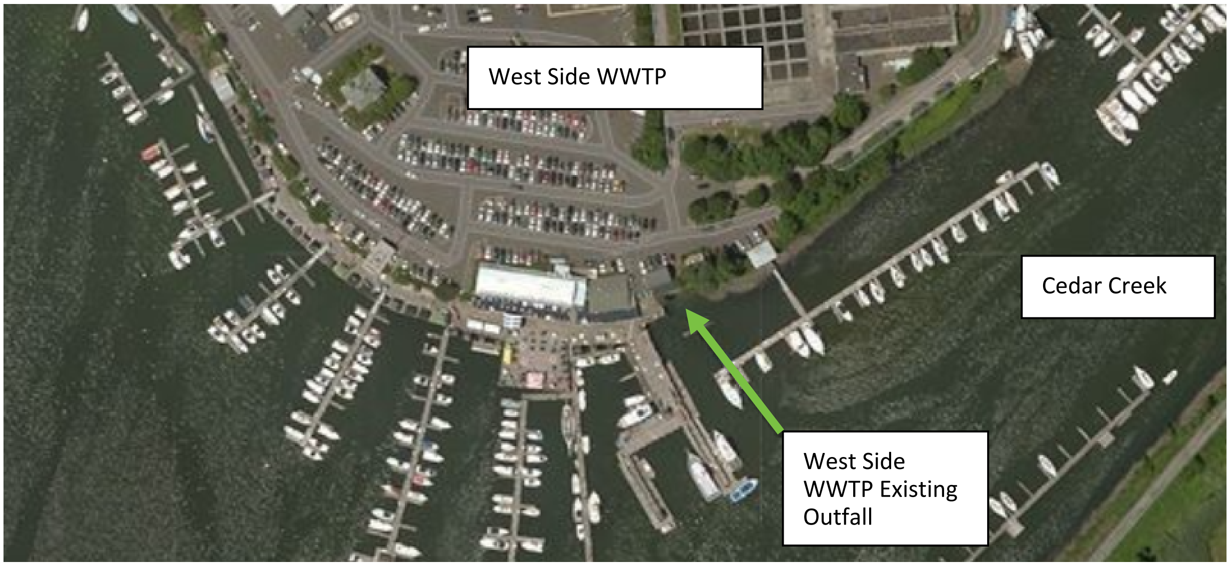


Figure 6.10-2
Location of West Side WWTP and its Existing Outfall

The East Side plant's outfall discharges through a 60-inch reinforced concrete pipe (RCP) at the shore of the Powerhouse Channel, which is a dead-end channel off the east side of the Pequonnock River in Bridgeport Inner Harbor (**Figure 6.10-3**). Some sections of the outfall were built circa 1950s, while others were constructed in 1969. The existing condition of the pipe is presented in Section 4.3.6.

The outfall pipe is extended into the channel with metal sheet piling driven into the channel bottom that directs the flow across the intertidal zone at about a 30-degree angle toward the Inner Harbor. Even with this longer flow path directed toward the long axis of Powerhouse Channel, the channel is a relatively confined area that will limit the initial dilution of the effluent.

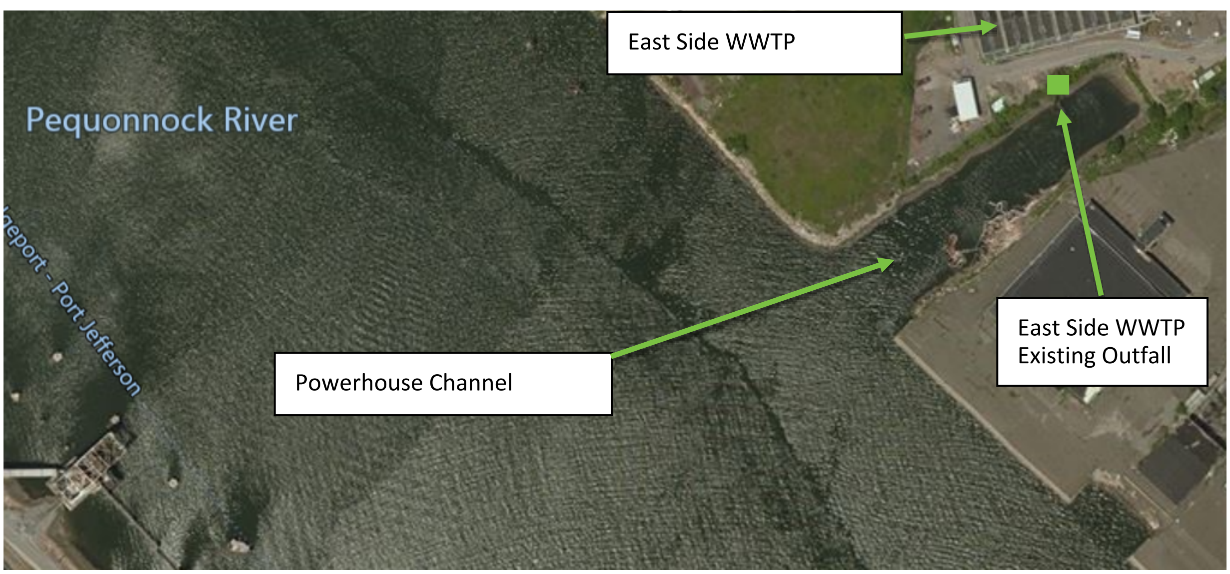


Figure 6.10-3
Location of East Side WWTP and its Existing Outfall

6.10.2.2 West Side WWTP Existing Location with Rehabilitated Outfall

This alternative would retain the discharge location into Cedar Creek but would include inspecting, cleaning and, where needed, rehabilitating the existing 72-inch RCP outfall pipe. The outfall was inspected in January 2020; the inspection is described in Section 4.2.6 and included in Appendix E.

In general, the video inspection found the pipe to be in relatively good shape with the following issue noted:

- Rock/debris located in pipe invert

Based on the video inspection, the following actions are recommended for rehabilitating the existing outfall pipe:

- Investigate structure with tangled PVC lines just upstream of Manhole 1
- Investigate opening in crown of pipe at 207 feet downstream of Access Point 1 that contains wood structure and wire
- Remove rock and debris from the pipe invert

Future rehabilitation efforts could include CIPP lining of the 800-foot stretch of 72-inch RCP using the various manhole access points and junction chambers. This would require a full bypass and dewatering, which would require a cofferdam at the outlet to prevent tidewater from entering the pipeline. The pipeline would also require cleaning to remove any build up on the pipe interior to reestablish the original inside diameter. Any active infiltration would have to be either stopped by grouting or installing a pre-liner. CDM Smith assumed a fully deteriorated pipe design would be needed for the new liner making it a stand-alone pipe that does not rely on the original host pipe for support. The planning-level cost range for the lining effort including bypass and cleaning would be $2-3 million. Open cut replacement should also be considered if relocating the line is desired. Cost for open cut replacement would depend on route chosen which would affect pipeline length and depth as well as required surface restoration efforts.

6.10.2.3 East Side WWTP Existing Location with Rehabilitated Outfall

This alternative would retain the discharge location into the Powerhouse Channel but would include inspecting, cleaning and, where needed, rehabilitating the existing 60-inch RCP outfall pipe.

Satellite images show the extent of the metal sheeting extension of the outfall pipe is extensively deteriorated compared to that shown on the 1992 design drawings (**Figure 6.10-4**).

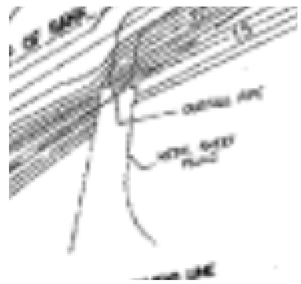


Figure 6.10-4
East Side Plant Outfall Design Plans (1992) and Current Condition from Google Map Image

In January 2020, a video inspection was conducted of the East Side plant's outfall pipe; it is described in Section 4.3.6 and included in Appendix E. In general, the video inspection found the pipe to be in relatively good shape with the following issues noted:

- Rock/debris located in pipe invert

Based on the video inspection, the following actions are recommended for rehabilitating the existing outfall pipe:

- Replace the metal sheeting that directs the outfall discharge across the intertidal zone and toward the main harbor channel
- Remove floating debris from pipeline
- Remove rock/debris from pipe invert

Future rehabilitation efforts could include CIPP lining of the 600-foot stretch of 60-inch RCP utilizing the various junction chambers. This would require a full bypass and dewatering that would require a cofferdam at the outlet to prevent tidewater from entering the pipeline. The pipeline would also require cleaning to remove any build up on the pipe interior to reestablish the original inside diameter. Any active infiltration would have to be either stopped by grouting or a pre-liner installed. CDM Smith assumed a fully deteriorated pipe design would be needed for the new liner making it a stand-alone pipe that does not rely on the original host pipe for support. The planning-level cost range for the lining effort including bypass and cleaning would be $1.3 - 1.9 million. Open cut replacement should also be considered if relocating the line is desired. Cost for open cut replacement would depend on route chosen which would affect pipeline length and depth as well as required surface restoration efforts.

6.10.2.4 East Side WWTP Inner Harbor Location

This alternative would move the discharge terminus to the seawall facing Bridgeport Inner Harbor. The alignment is shown on **Figure 6.10-5**. This route extends onto adjacent property and an approximate 200-ft long easement would need to be negotiated with the current landowner.

Due to the wide range of future effluent flows (minimum flow = 4 mgd; maximum flow = 80 mgd; (**Section 5.4**) it is not possible to maintain good initial dilution for a single port discharge. Therefore, CDM Smith recommends discharging treated effluent through two ports: one would convey typical low flows (say 4 to 14.2 mgd, which would span minimum through maximum day flows at 2050), while the other would handle flows up to the peak hour flow of 80 mgd. The split of flows can be revised during design as long as reasonable initial dilution can be obtained across the range of discharge flows for each port.

Several alternatives exist for conveying the treated effluent from the UV disinfection facility/effluent pump station to the harbor. As shown in Figure 6.10-5, both pipes would be between 250 and 350 feet long, depending on the final layout of the UV disinfection facilities and effluent pump station, and would be installed along the north side of the Powerhouse Channel to its intersection with the Pequonnock River. The decision of which alternative to use will depend on easement constraints and tradeoffs of hydraulics and capital and O&M costs among the pump station and the piping alternatives.

For this facilities plan, CDM Smith developed a concept sizing for the outfall system, which has two pipes from the treatment plant site to the shore of the Inner Harbor. A distribution box, located on the current plant site, would be used to direct flow to the smaller pipe but allow higher flows to discharge through the larger pipe.

The smaller of the two pipes would be 30 inches in diameter but would be fitted with a 30 by 18-inch diameter reducer at the shore; it would be the active outfall most of the time. The system was sized to convey the maximum day flow of 14.2 mgd.

The larger 60-inch diameter pipe would convey flows up to the peak flow of 80 mgd but would be fitted with a 60 by 36-inch diameter reducer. Because it would be infrequently used, CDM Smith recommends a tide gate or check valve (e.g., a Red Valve duckbill) be added to the end of the pipe.

For this two-pipe concept, the distribution of flow between the smaller and larger diameter outfall pipes would be controlled at the effluent pump station. Most of the time plant effluent would be conveyed by gravity through the pump station until a combination of high flow and/or high-water levels in the Inner Harbor require activation of the pumps.

Figure 6.10-5
East Side Plant with Existing Outfall and Alternative Outfall Location in the Inner Harbor

The effluent pump station would be divided in half, one half for typical dry weather flows (4 to 14.2 mgd) with either of two pumps discharging to the 30-inch diameter outfall pipe and the other half with up to four pumps discharging wet weather flows (up to 80 mgd) to the 60-inch diameter outfall. Plant effluent would enter the dry weather wet well side of the pump station and conveyed either by gravity or pumps to the 30-inch diameter outfall until it reached its capacity. When the effluent flow exceeds the capacity of the 30-inch dry weather outfall, plant effluent would overflow from the dry weather wet well to the adjacent wet weather wet well and be conveyed either by gravity or pumps to the 60-inch outfall. Gravity flow would be through flap gates located between the wet wells and discharge chambers at the pump station.

Construction would be cut-and-cover and a concrete headwall would need to be installed with wing walls extending about two feet into the channel to protect the terminus of the outfall pipe.

6.10.2.5 West Side WWTP Outer Harbor Location

This alternative would move the existing West Side WWTP discharge location to Bridgeport Outer Harbor about 11,000 feet southeast of the treatment plant, and 3,000 feet to the northeast of the end of the dredged channel to Bridgeport Inner Harbor. A multiport diffuser would be added to the outfall terminus to enhance the initial mixing of the treated effluent.

The location was selected because it is outside of the main shipping lanes in Bridgeport Harbor and far enough offshore so that the main currents (as modeled by LMS, 2002) should carry the effluent past the shoal extending from the Fairfield shoreline to Black Rock and Penfield Reef out into Long Island Sound. The location is also beyond the state-managed shellfish bed leases that are shown on the most recent state maps (Section 6.10.3.4).

The multiport diffuser would consist of a buried manifold oriented perpendicular to the main current as determined by the RMA-10 modeling performed by LMS (2002) of Bridgeport Harbor. Each riser would have two horizontally discharging, alternating ports each (in a T arrangement) extending one to two feet above the seabed; **Figure 6.10-6** provides a conceptual image of the multiport diffuser.

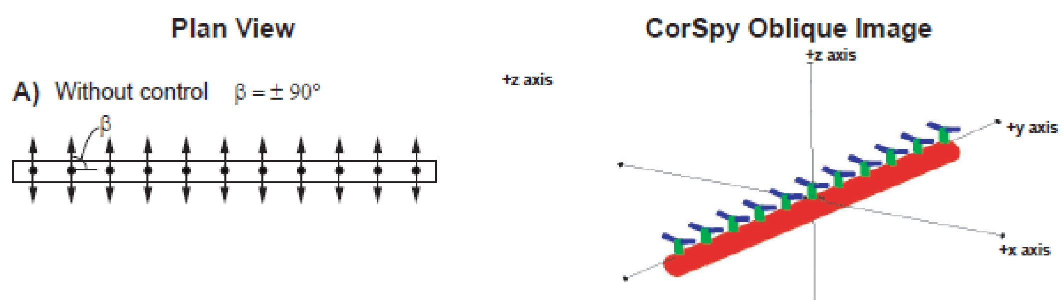


Figure 6.10-6
Conceptual Alternating Multiport Diffuser (Image from *CORMIX User Manual* (Doneker and Jirka, 2007)

Table 6.10-2 provides details of the outfall and multiport diffuser design for the current and three larger peak flow rates. The diffuser design criteria selected for this initial screening are preliminary and would need to be further evaluated in a concept design once flow rates are better established. In general, the design criteria were selected to have the densimetric Froude number be larger than 1, ensure the sum of the port areas was smaller than the area of the manifold pipe, maintain manifold velocities close to the range of 1 to 10 ft/s, and maintain port velocities as close to the range of 1.6 to 15 ft/s. The latter criterion was the most difficult to meet and any future concept design may need to consider the addition of a variable orifice check valve to maintain target port velocities under low plant flows.

Table 6.10-2 Design Criteria for an Offshore Outfall for the West Side WWTP for Various Peak Effluent Flows

	Peak Effluent Flow Rates			
	90 mgd	140 mgd	180 mgd	200 mgd
Outfall Length	11,000 ft	11,000 ft	11,000 ft	11,000 ft
Outfall Pipe Inside Diameter	6 ft	6 ft	6 ft	7 ft
Diffuser Length	320 ft	320 ft	320 ft	320 ft
Diffuser Manifold Diameter	6 ft	6 ft	6 ft	7 ft
Number of Risers	16 risers	16 risers	16 risers	16 risers
Number of Ports	32 ports	32 ports	32 ports	32 ports
Port Spacing	20 ft	20 ft	20 ft	20 ft
Port Diameter	7 inches	8.5 inches	9 inches	10 inches

Several options exist for construction of the outfall pipe to this offshore location; in all cases, CDM Smith assumed the diffuser manifold will be buried with risers extending above the seabed to accommodate the twin ports for discharge.

- Directional drill across Cedar Creek only and then cut-and-cover excavation to the diffuser site.
- Directional drill to the eastern shore of Seaside Park and then cut-and-cover excavation
- Tunnel – there would be a launch shaft at the West Side plant and a TBM would be used to bore with an outside diameter minimum of about 96 inches. It is expected the shaft that would be used to connect the tunnel to the multiport diffuser on the seabed could also be used to remove the TBM when the bore is complete.

6.10.3 Evaluation of Outfall Alternatives

Basic planning criteria for the project are described in Section 2. The planning period for this Wastewater Facilities Plan is 30 years (2020 – 2050). Outfalls are generally designed for a longer time span because they have a longer usable life (e.g. the current WPCA outfalls are 50-70 years old). While flow projections were not completed beyond 2050, it is unlikely that peak system flows will increase beyond those included in this facilities plan because the peak flows are dominated by increasing the wet weather flow to the treatment plants, and thus, represent an engineered maximum flow.

6.10.3.1 Outfall Evaluation Criteria

The following criteria are used to evaluate the outfall options:

- Ability to meet permit limits and water quality standards
- Whether the effluent can be discharged by gravity or needs to be pumped
- Potential for shell fishing impacts
- Potential for navigation impacts

- Potential for aesthetic impacts
- Potential for impacts to public recreation areas
- Permitting requirements
- Constructability
- Reliability
- Cost
- Public input

The criteria and evaluation of them for the candidate outfall sites is described below. Criteria are either rated quantitatively, qualitatively relative to the specific criteria or use the scheme for non-economic evaluations described in Section 2: Favorable, Neutral or Unfavorable.

6.10.3.2 Meet Permit Limits and Water Quality Standards

Existing Permit Limits and Compliance

The treatment facilities operate, and discharge treated effluent under the terms and conditions of the NPDES permit Nos. CT0100056 (West Side WWTP) and CT0101010 (East Side WWTP). The West Side plant permit expires June 30, 2024. The East Side plant permit expires October 28, 2020 (The WPCA submitted the East Side plant permit renewal application in April 2020.)

Section 1.4 describes the current NPDES permits for the West Side and East Side plants. Nitrogen in the effluent is controlled by CT DEEP's General Permit for Nitrogen and expires on December 31, 2023.

Review of the compliance reports submitted to CT DEEP's Discharge Monitoring Report (DMR) through EPA's Enforcement and Compliance History Online (ECHO) system indicate the following exceedances of permit limits:

1. West Side plant (since most recent permit effective July 2019): once for daily max BOD_5, four times for daily max TSS, once each for maximum fecal coliform and enterococci, once for 30-day geomean for enterococci, and 3 times for TSS removal percent.
2. East Side plant (3 years for September 2018 to September 2020): two for daily max TSS, three for maximum fecal coliform, and two for maximum geomean enterococci and six for 30-day geomean enterococci.

Two notes about the ECHO compliance data: (1) ECHO does not include data for the annual average load limit for total nitrogen and (2) a shorter duration was used to evaluate non-compliance events at the West Side plant because significant changes were made in the most recent permit renewal that corrected an error in the previous renewal.

The West Side and East Side plants also have 48-hour acute screening whole effluent toxicity (WET) testing requirements. Testing is performed quarterly with *Daphnia pulex* and *Pimephales promelas* where the pass/fail requirement is 90% survival on average in full strength effluent (the control must also have 90% survival on average for the test to be valid). Review of WET testing results reported in the CT DEEP's DMR system since 2016 indicates that the effluent from both plants passed these permit limits, with the exception of the West Side plant's *D. pulex* test in March 2017 (two required subsequent monthly tests passed) and both organisms for the East Side plant in December 2016.

Future Compliance

The plan to meet effluent requirements is described in Section 2.7; the updated facilities will be designed to meet current permit NPDES limits for BOD_5, TSS, total nitrogen, fecal coliform, enterococci, and total residual chlorine; the East Side plant also has an effluent limit for copper. CT DEEP is currently reassessing the nitrogen loads to Long Island Sound, and thus, the plant's design will include the flexibility to add supplemental carbon to lower effluent nitrogen concentrations if this is required in the future.

Therefore, it is expected that the future updated treatment plants will meet their permit limits.

Flexibility for the Future

Because both plants have a record of compliance with the current NPDES permit requirements and future upgrades is anticipated to improve on that performance, the ability to meet permit limits and water quality standards is judged to be adequate for all scenarios involving the existing outfalls under current flows. Increases in peak flows (though not design flows) are planned for the new treatment plants; it is not anticipated that permit limits will change due to the increased high flows.

These outfalls, however, have poor initial dilution. The LMS (2002) report documented the results of 5-day dye studies of the, then, existing discharges and measured dilution in the immediate vicinity of the outfalls. At the West Side plant, the observed dilution ranged from 1.3:1 to 2:1 within 20 feet of the discharge; approximate one mile from the outfall the dilution typically reached 100:1. While effluent flow is not reported in the LMS report, the effluent flows were likely below average to average as any storm flows are quickly conveyed through the wastewater network and treatment plant.

A five-day dye study of the East Side plant (also conducted by LMS) indicated slightly higher dilution – between 3:1 to 4:1 in the immediate vicinity (20 feet) of the discharge, which reached 100:1 within 0.5 to 0.7 miles of the discharge. Again, typical dry weather flows are expected to have occurred during the dye test. Dilutions would be even lower when elevated or peak flows occur.

Such low dilutions do not provide flexibility for meeting new or more stringent water quality criteria, such as receiving water standards that will likely be developed for PFAS compounds in the coming years. To investigate improvement in dilution for discharges from the East Side and West Side plants, CORMIX (an EPA developed and approved model) was used with the designs described in Section 6.10.2.

CORMIX (Doneker and Jirka, 2007) was used to estimate near-field dilution of the alternative outfall locations. The model requires inputs on effluent and ambient receiving water conditions. To the extent possible, the model inputs were derived from the LMS (2002) report that modeled both near- and far-field conditions of the existing and hypothetical new discharges. Three CORMIX modules exist and all three were used on this project:

- CORMIX1 simulates single port submerged discharges
- CORMIX2 simulates multiport submerged discharges
- CORMIX3 simulates surface shoreline discharges from a single port

The simulations that were made used the same ambient and effluent conditions to allow comparison among the outfall alternatives; if a new outfall is selected, then additional model simulations will be needed to examine dilutions across a broader range of velocity and stratification, and effluent conditions. Inputs to the model included:

- Effluent Flow – only peak flows were considered as these produce the least dilution
- Effluent Density – 998 kg/m^3
- Stratification – an expected mid-summer condition of 1 sigma-t units of linear stratification at the West Side plant Outer Harbor alternative and while uniform conditions were simulated in the shallow water at the East Side plant Inner Harbor alternative.
- Ambient Velocity – Typical slack (0.5 ft/s) and max (1.6 ft/s) tide velocities taken from LMS (2002) figures with current vectors for the West Plant, and 0.05 ft/s (slack) and 0.1 (max) at the East Side plant.

The modeling results are shown in **Table 6.10-3** for the West Side plant. Dilutions reported in this table are from CORMIX2 simulations and represent dilution at the end of near-field region (generally when turbulent mixing ends and additional mixing is a passive process). They indicate a significant (10-50 times) improvement in dilution by discharging effluent through a submerged multiport diffuser in the Outer Harbor near the entrance to the shipping channel.

Dilutions at lower flow rates are not reported for the Outer Harbor outfall alternative for the West Side plant because they are expected to be higher than those for peak flows as long as the ultimate diffuser design incorporates a check valve to maintain higher discharge velocities at lower plant flows.

Table 6.10-3 Results of Initial Dilutions Simulations for the Outer Harbor Alternative for the West Side WWTP at Four Different Peak Effluent Flows

Plume Characteristics at End of the Near Field	Peak Effluent Flow Rates			
	90 mgd	140 mgd	180 mgd	200 mgd
Dilution Slack Tide	39.6:1	28.3:1	23.7:1	22.2:1
Dilution at Max Tide	95:1	61.6:1	49.3:1	43.6:1
Distance at Slack Tide	340 feet	260 feet	320 feet	350 feet
Distance at Max Tide	130 feet	150 feet	160 feet	170 feet
Width at Slack Tide	620 feet	800 feet	960 feet	1,070 feet
Width at Max Tide	200 feet	340 feet	340 feet	350 feet
Plume Rise Height	Surface	Surface	Surface	Surface

The model results for the East Side plant's Inner Harbor outfall are also higher (**Table 6.10-4**) than those measured in the dye study for the existing discharge. Dilution results are shown from the typical and high effluent flows expected to be discharged from each outfall pipe: 6.4 to 14.2 mgd for the smaller 30-inch outfall pipe fitted with an 18-inch reducer; 14.2 to 80 mgd for the larger 60-inch outfall pipe fitted with a reducer to 36 inches. Even though the water depth is only slightly deeper at the location facing the Inner Harbor, the placement of the outfall below the water surface and the larger discharge velocity resulting from use of a reducer to mimic an outfall port results in improved initial dilution over that measured for the existing discharge. Dilutions reported in Table 6.10-4 are centerline dilutions when the edge of the plume (the distance from the shore to the center of the plume + the half width of the plume) is 660 feet out into that channel – a distance that allows for a zone of passage as required by CT DEEP's Zone of Influence rules in subsection (l) of section 22a-426-4 of the Regulations of Connecticut State Agencies.

Table 6.10-4 Results of Initial Dilution Simulations for the Inner Harbor Alternative for the East Side WWTP

Plume Characteristics at End of the Near Field	30-inch Outfall with 18-inch Port from CORMIX1 Simulations		60-inch Outfall with 36-inch Port from CORMIX 3 Simulations	
	6.4 mgd	14.2 mgd	14.2 mgd	80 mgd
Dilution Slack Tide	19.5:1	26.9:1	8:1	18.6:1
Dilution at Max Tide	43:1	23.4:1	8:1	18.4:1

6.10.3.3 Gravity or Pumped Discharge

Due to the requirement that treated wastewater be able to be discharged against the 100-year flood elevation plus three feet (Section 2.5), all of the options for discharging from the West Side and East Side plants will require pumping during combinations of high plant flow and high water levels.

Both the Outer Harbor alternative for the West Side plant and Inner Harbor alternative for the East Side plant will require more frequent pumping due to the higher head loss included in the outfall designs to enhance initial mixing.

Because pumping will be required for all alternatives because of the increase in water level against which the effluent must be pumped, all alternatives are graded as unfavorable.

6.10.3.4 Potential for Shellfishing Impacts

Figure 6.10-7 indicates that Bridgeport's Inner Harbor and Black Rock Harbor are closed to shellfishing (https://portal.ct.gov/-/media/DOAG/Aquaculture/2019/Fairfield_to_Stratford.pdf -- accessed 8/31/2020); these areas are where the existing outfalls and CSOs discharge.

The Outer Harbor outfall alternative for the West Side plant is located just beyond the state managed shellfish beds. This area of the Outer Harbor is designated as Restricted Relay. According to the CT Department of Agriculture web page, restricted relay is "a classification used to identify a growing area where harvested shellstock is relayed to Approved or Conditionally Approved waters for natural cleansing or depuration. An area may be classified as Restricted Relay when a sanitary survey finds a limited degree of pollution and levels of fecal pollution, human pathogens, or poisonous or deleterious substances so that shellstock can be made safe for human consumption by either relaying, depuration, or low acid-canned food processing."

If the West Side plant's outfall were to be relocated to the Outer Harbor, a new permanent closure zone would be designated around the outfall, similar to, but likely larger than, the red circle in the southwest portion of the map that is designated around Fairfield's outfall.

The alternatives that include the existing outfall and the Inner Harbor alternative for the East Side plant are all rated as neutral because they discharge to an area that is already permanently closed to shellfishing. The Outer Harbor alternative for the West Side plant is rated as unfavorable because a new region of the harbor would have to be permanently closed to shellfishing.

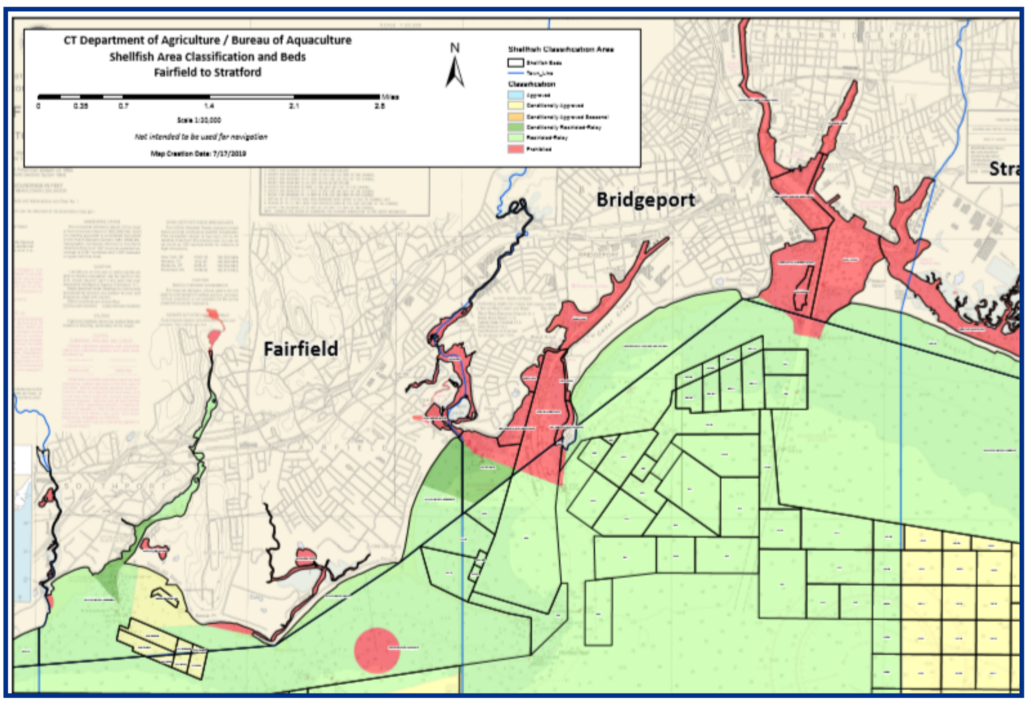


Figure 6.10-7
Shellfish Classifications and State-Managed Bed Designations in the Bridgeport Area

6.10.3.5 Potential for Navigation Impacts

This criterion measures the degree to which navigation in the harbor by recreational boating or commercial vessels would be impacted.

Both new outfall options for the West Side and East Side plants have the potential for minor impacts to navigation.

The West Side plant outfall alignment to the Outer Harbor discharge locations has the potential for navigation impacts during construction as the new outfall alignment would need to cross the dredged channel in Cedar Creek. This potential can be mitigated by selecting construction techniques (e.g. directional drilling under the channel). The terminus of the outfall was selected because it does not need to cross shipping lanes into either Bridgeport or Black Rock Harbors.

The concept design East Side plant alternative discharge location to the Pequonnock River described in Section 6.10.2.2 includes wing walls that could extend one to two feet into the harbor and could result in a minor navigation impact to boat traffic along the shore.

6.10.3.6 Potential for Aesthetic Impacts

Both existing discharges have a high potential for aesthetic impacts as they are surface discharges to shallow water. In addition, the West Side plant's discharge flows immediately through dockage for pleasure boats.

There is a reduced potential for aesthetic impact for the East Side plant Inner Harbor outfall alternative because the outfall pipe will be submerged, and the discharge is to an open water body as compared to the dead-end Powerhouse Channel. It is likely, however, that an expression of the discharge will still be visible on the surface as a boil, particularly at low tide.

The Outer Harbor alternative for the West Side plant has minimal chance for aesthetic impacts due to its deeper water depth and the use of a multiport diffuser to significantly improve the initial mixing of the effluent discharge.

6.10.3.7 Potential Impacts to Public Recreation Areas

Public recreation areas in Bridgeport and the adjacent communities include:

- Bridgeport – Seaside Park and Pleasure Beach
- Stratford – Long Beach
- Fairfield – Jennings Beach

In general, potential impacts to public recreation areas are small because the upgraded plants will discharge well treated, disinfected effluent.

If a future discharge had an upset and disinfection was not sufficiently effective to meet permit limits, discharges from the East Side plant are sufficiently close that bacteria concentrations at Pleasure Beach and Long Beach would be expected to be similar. The Outer Harbor outfall candidate location offers benefits for lessening impacts to public recreation areas as compared to the existing location due the larger initial dilution afforded to the offshore site and the greater distance to public recreation areas to further dilute the discharged effluent.

6.10.3.8 Permitting Requirements

Construction of a new outfall will require several permits, while the permit requirements of the cleaning and possible rehabilitation of the existing outfalls will depend on the work that is determined to be required. **Table 6.10-5** lists the likely permits required for each alternative of this project.

The initial stage in the permitting process is to complete a DEEP Pre-Application Questionnaire, which requires information on the proposed project and particular activities of the project so that jurisdiction of various permitting agencies can be determined so that the agencies can be involved from the start of a proposed project. A completed questionnaire is the precursor to a pre-application meeting, where the components of the proposed project are discussed and permitting requirements will be identified.

Table 6.10-5 Potential Permit Requirements for the Outfall Options

Permit	No Action	Existing Outfall Cleaned and Possibly Rehabbed		East Side Plant Inner Harbor Discharge	West Side Plant Outer Harbor Discharge
		East Side Plant	West Side Plant		
Local Permits					
• Planning and Zoning	None	Likely	Unlikely	Likely	Likely
State Permits					
• LISP Structures, Dredging and Fill, including 401 WQ Cert.	None	Required	Unlikely	Required	Required
• LISP Tidal Wetlands	None	Required	Unlikely	Required	Required
• DEP General Permit for Discharge of Stormwater and Dewatering Wastewaters Associated with Construction Activities	None	Unlikely	Unlikely	Required	Required
• CEPA Review	None	Unlikely	Unlikely	Likely	Likely
• Updated NPDES Permit	None	Likely	Likely	Required	Required
Federal					
• USACE Section 10 and 404	None	Unlikely	Unlikely	Required	Required

Table 6-10.5 shows that the cleaning and possible rehabilitation of the existing outfall at the East Side plant is likely to have more permitting requirements than the cleaning and possible rehabilitation at the West Side plant because of the need to replace the existing sheeting at the East Side plant's outfall. Permitting requirements for either of the new outfall options are similar.

6.10.3.9 Constructability

Constructability is a measure of the ease and risk associated with construction. This criterion includes consideration of the impact of adverse weather, conflicts with existing utilities, soil conditions and geotechnical analysis, a general assessment of the likelihood of encountering unknowns, and the ease with which Bridgeport could acquire an easement or access from property owners.

Neither no action options nor the West Side plant cleaning and possible rehabilitation option have construction associated with them, as the clean activities are more clearly classified as maintenance. The East Side plant cleaning and possible rehabilitation option would have minor construction associated with replacing the sheeting, which has lower constructability risk.

The anticipated construction on the Inner Harbor outfall for the East Side plant is relatively straight forward. The dual outfall pipes would be laid on land in a common trench largely through a vacant lot, fitted with standard reducers and discharge through a new headwall facing the Inner Harbor. At this concept stage, while soil conditions are unknown, CDM Smith does not anticipate major conflicts with existing utilities, maintenance of plant operations should be relatively straight forward as the treatment plant can continue to use the existing outfall until new outfall is complete, and weather should not pose a significant risk. There is uncertainty about the ability to obtain an easement through the adjoining property. In sum, the construction risk for the Inner Harbor outfall option for the East Side plant is graded low risk.

In contrast, the Outer Harbor option for the West Side plant is graded high risk. Several construction options exist, and more information is needed before one could select among them. This includes data on subsurface conditions and a geotechnical evaluation to determine if a cut-and-cover method would need to be pile supported and the depth and technique TBM or remotely operated micro tunnel boring machine (MTBM)) best suited for a tunnel option. The degree of unknowns is high for the Outer Harbor for West Side plant offshore outfall option and include regulatory acceptability, potential for impact on shellfish lease holders (Section 6.10.3.4).

6.10.3.10 Reliability

Reliability is defined as the level of assurance that an alternative will continuously operate over the expected operating conditions throughout the life of the project. Options are compared based on their ability to avoid service interruptions.

Bridgeport's existing outfalls have reliably served as discharge locations for decades. The outfall structures are simple pipes and as long as they are structurally sound can continue to serve as effective discharge points. They are, however, decades old and some of the WWTP options would result in doubling or more of their peak flows. Hydraulic analysis indicates that discharge velocities at the highest peak flows could be accommodated by the existing pipes at peak velocities. Due to their age, the existing outfalls should be inspected regularly (e.g. every five years) and repaired/rehabilitated as needed.

New outfalls would increase the reliability of the WPCA's discharge conveyances by virtue of their being custom designed and built for the peak flows that are selected to be discharged at each outfall.

The no action alternatives are graded as unfavorable due to the age of the outfall pipes, which means that they have a shorter remaining anticipated useful life. The alternatives that include cleaning and rehabilitation of the existing outfall area was graded as neutral (as long as they are inspected and maintained), while the two alternatives with new outfalls are graded as favorable.

6.10.3.11 Cost

Cost is defined as a planning-level capital cost to construct the alternative include appropriate contingencies and engineering costs. O&M costs would vary based on electricity cost for pumping when needed, and periodic outfall inspections.

There is no additional cost associated with the no action alternatives; however, given the age of the existing outfall pipes they should be inspected every 5 years and repairs and rehabilitation should be undertaken as needed.

Costs for cleaning the existing outfalls range from $75,000 to $125,000 for the East Side plant and $100,000 to $150,000 for the West Side plant.

The estimated cost for the East Side plant Inner Harbor candidate location is $1.3 million. While the cost of extending the West Side plant's outfall to a location near the entrance to the shipping channel would be $100 million to 200 million. Note that a detailed cost estimate was not performed for the West Side plant offshore outfall option because its high cost compared to the environmental benefit indicates it is not the preferred alternative. Approximate costs were taken from similar sized tunnels priced to connect the two treatment plants, which was eliminated in the early alternatives' analysis.

6.10.3.12 Public Input

A public meeting was held on October 29, 2020 to receive input on the Facilities Plan. Several commenters indicated a preference for moving the West Side plant outfall from Black Rock Harbor to the Outer Harbor location. Comments included information on a newly (2019) included sampling station in Black Rock Harbor as part of the Save the Sound sampling effort, concerns about ongoing water quality impairments in the harbor, and concerns about increasing the flow through the West Side plant outfall.

No comments were received on the East Side plant outfall recommendations.

6.10.3.13 Summary

Table 6.10-6 presents a summary of the evaluation of criteria for the alternative outfall locations.

Table 6.10-6 Summary of Outfall Evaluation Criteria

Outfall Criteria	No Action		Existing Outfall Cleaned and Possibly Rehabbed		East Side Plant Inner Harbor Discharge	West Side Plant Outer Harbor Discharge
	East Side Plant	West Side Plant	East Side Plant	West Side Plant		
Ability to meet Permit Limits and WQS	Unfavorable	Unfavorable	Unfavorable	Unfavorable	Favorable	Favorable
Gravity or Pumped Discharge	Unfavorable	Unfavorable	Unfavorable	Unfavorable	Unfavorable : Slight Increase Pumping Frequency	Unfavorable: Larger Increase in Pumping Frequency
Potential for Shellfishing Impacts	Neutral: Bridgeport's Inner Harbor and Black Rock Harbor are currently permanently closed to shellfishing					Unfavorable: New Closure Area would be Designated

Outfall Criteria	No Action		Existing Outfall Cleaned and Possibly Rehabbed		East Side Plant Inner Harbor Discharge	West Side Plant Outer Harbor Discharge
	East Side Plant	West Side Plant	East Side Plant	West Side Plant		
Potential for Navigation Impacts	None	None	None	None	Minor	Minor
Potential for Aesthetic Impacts	High	Very High	High	Very High	Moderate	None
Potential Impacts to Public Recreation Areas	Neutral: No change from existing conditions					Favorable
Permitting Requirements	None	None	Few	Unlikely	Most	Most
Constructability	NA	NA	Low Risk	NA	Low Risk	High Risk
Reliability	Unfavorable	Unfavorable	Neutral	Neutral	Favorable	Favorable
Planning Level Cost	--	--	$75-125K	$100-150K	$1.3M	$100-200M*
Public Input	No input	Some prefer the outer harbor option	No input	No input	No input	Preferred option of some

* A detailed cost estimate was not performed for the West Side plant offshore outfall because its high cost compared to the environmental benefit indicates it is not the preferred alternative. Approximate costs were taken from similar sized tunnels priced to connect the two treatment plants, which was eliminated in the early alternatives' analysis.

6.10.4 Recommendations

6.10.4.1 West Side WWTP

The West Side plant should continue to discharge through its existing outfall pipe though the pipe should be cleaned and rehabilitated as described in Section 6.10.2.2. The planning level cost estimate for this effort is estimated to be between $100,000 and $150,000.

Because the West Side plant outfall is now over 70 years old, it should be inspected frequently: every five years. This is important because the peak flows through the outfall are being designed to more than double from 90 mgd today to 200 mgd as the West Side plant takes on more wet weather flow. The higher flows could increase the rate of age-related deterioration of the pipe. While marine outfall pipes over 100 years old continue to be in service, it would be prudent to include monies in long-term capital planning for a more extensive rehabilitation of the outfall pipe, as described in Section 6.10.2.2. If a fully deteriorated pipe rehabilitation was required, a 2020-dollar planning level cost estimate would be between $2-3 million.

6.10.4.2 East Side WWTP

Retaining the existing outfall location is a low-cost solution, but CDM Smith does not recommend this alternative. Water quality reasons for replacing the outfall pipe include:

- The dead-end channel is poorly flushed allowing it to become filled with poorly diluted effluent creating the potential for effects to aquatic life and fish movement.

- The poorly flushed dead-end channel could be a repository for solids as solids tend to settle when freshwater is discharged to salt water.
- The water surface over the "plane" jet will probably have a visible disturbance, particularly at slack water and calm wind conditions.

From an engineering perspective, the layout of the new East Side plant is going to require a new connection to be made to the shore (either the existing outfall or a new terminus location). The length of outfall pipe is similar in both cases. Therefore, with costs a neutral factor, the prudent decision is to move the outfall so that it discharges from the headwall directly to the main channel of the Inner Harbor. This move improves initial mixing of the outfall, which increases flexibility and reliability of the outfall to meet changes in regulatory requirements throughout its anticipated useable life.

6.11 Residuals Management

6.11.1 Introduction

As presented in Section 4, the West Side and East Site WWTPs currently thicken primary and waste activated sludge generated on site and truck thickened sludge off-site for additional treatment and disposal. This reduces the assets on-site for residuals management and puts the onus on others for ultimate disposal. However, this comes at an estimated cost of about $2.6 million annually, to haul sludge off site, as well as the "cost" of increased truck traffic through the City and the state. Based on the deficiencies of the existing residuals management systems presented, it is recommended that new residuals management processes be installed at both plants as part of the overall plant improvements.

As the wastewater industry moves towards a resource recovery model, options to capture the value of the resources in the biosolids, including nutrients, carbon and energy, are investigated herein. These benefits must be balanced with the available land area at the site, the complexity of operations of some of these systems, the market potential of the product as well as potential for increased odor generation. The improved liquid treatment train proposed will also improve the quality of residuals generated (by providing upstream grit removal), and increase the quantity of sludge generated, through improved removal efficiency and increased treatment plant flow. In addition, certain liquid treatment processes (e.g. primary filtration) could increase the carbon capture in the primary sludge, increasing the energy value of the biosolids.

This section first presents the range of estimated sludge quantities expected with the upgraded treatment system, then evaluates the following alternatives for primary and waste activated sludge thickening, sludge dewatering, digestion, drying and ultimate disposal.

Thickening

Dissolved Air Floatation

Gravity Belt Thickeners

Rotary Drum Thickeners

Thickening Centrifuges

Gravity Thickeners

Emerging Technologies

Dewatering

Belt Filter Press

Dewatering Centrifuges

Rotary Press

Screw Press

Stabilization

Digestion

Gasification

Incineration

Drying

6.11.2 Future Solids Production

Based on the influent flows and loadings established in Section 5 in combination with expected unit process performance and sludge yield of the candidate liquid treatment train processes, a range of preliminary primary and waste activated sludge quantities were established to evaluate the various residuals management processes. Average day and maximum month quantities are presented for the estimated initial year of operation through the 2050 planning period.

6.11.2.1 Primary Sludge Production

Tables 6.11-1 and 6.11-2 present initial year and future primary solids production for the West Side and East Side plants for two primary treatment alternatives – traditional primary settling tanks and primary filtration. Values assume grit will be removed in an advanced grit removal system in the headworks. It is also assumed that the solids content of the primary sludge removed from a traditional primary settling tank would be approximately 1.1 percent. Solids content from primary filtration is estimated at 0.2 percent.

Table 6.11-1 West Side WWTP Primary Solids Production: Initial and Design Year

	Initial Year (2027)			Design Year (2050)		
	Solids Content (%)	Load (lbs/day)	Volume (gal/day)	Solids Content (%)	Load (lbs/day)	Volume (gal/day)
	Average Day Conditions					
Traditional Primary Settling Tanks	1.1	23,500	255,800	1.1	34,400	375,400
Primary Filtration	0.2	33,500	2,009,600	0.2	49,200	2,949,600
	Maximum Month Conditions					
Traditional Primary Settling Tanks	1.1	45,000	490,200	1.1	66,000	720,300
Primary Filtration	0.2	64,200	3,851,300	0.2	94,400	5,659,500

Table 6.11-2 East Side WWTP Primary Solids Production: Initial and Design Year

	Initial Year (2029)			Design Year (2050)		
	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)
	Average Day Conditions					
Traditional Primary Settling Tanks	1.1	3,500	37,800	1.1	6,000	65,900
Primary Filtration	0.2	5,000	297,400	0.2	8,600	518,000
	Maximum Month Conditions					
Traditional Primary Settling Tanks	1.1	6,400	69,600	1.1	11,200	122,000
Primary Filtration	0.2	9,100	546,800	0.2	16,000	959,200

6.11.2.2 Waste Activated Sludge Production

Tables 6.11-3 and 6.11-4 include estimated initial and design year waste activated sludge production for the West Side and East Side plants. A conservative net yield of 1.1, and secondary solids content of 1.1% were used to determine sludge loads and quantities for treatment alternatives utilizing both traditional primary and BNR systems. For alternatives utilizing both primary filtration and IFAS, a net yield of 0.9 and secondary solids content of 1.1% were used.

Table 6.11-3 West Side WWTP Waste Activated Sludge Production: Initial and Design Year

	Initial Year (2025)				Design Year (2050)			
	Net Yield	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)	Net Yield	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)
	Average Day Conditions							
WAS – Traditional Primary	1.1	1.1	22,400	243,800	1.1	1.1	32,300	352,500
WAS – Primary Filters	0.9	0.7	14,200	243,200	0.9	0.7	20,500	351,900
	Maximum Month Conditions							
WAS – Traditional Primary	1.1	1.1	33,900	369,300	1.1	1.1	49,000	533,900
WAS – Primary Filters	0.9	0.7	21,500	369,000	0.9	0.7	31,200	533,800

Table 6.11-4 East Side WWTP Waste Activated Sludge Production: Initial and Design Year

	Initial Year (2025)				Design Year (2050)			
	Net Yield	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)	Net Yield	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)
	Average Day Conditions							
WAS – Traditional Primary	1.1	1.1	4,600	49,700	1.1	1.1	7,900	86,400
WAS – Primary Filters	0.9	0.7	2,600	44,700	0.9	0.7	4,500	77,500
	Maximum Month Conditions							
WAS – Traditional Primary	1.1	1.1	6,200	67,800	1.1	1.1	10,900	119,200
WAS – Primary Filters	0.9	0.7	3,500	60,800	0.9	0.7	6,200	106,800

6.11.2.3 Thickened Sludge Production

Dewatering alternatives will be evaluated based on thickened sludge production assuming thickening performance of 4% solids for thickened primary sludge and 7% solids for thickened waste activated sludge. Additionally, a solids capture of 95% was assumed for both primary and secondary thickening. Both thickened primary and secondary sludge would then be co-stored in tanks prior to dewatering. The combined thickened sludge loading at the West Side and East Side WWTPs for initial and design year is presented in **Tables 6.11-5 and 6.11-6**.

Table 6.11-5 West Side WWTP Combined Thickened Solids Quantity: Initial and Design Year

	Initial Year(2025)			Design Year (2050)		
	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)
	Average Day Conditions					
Traditional Primary & WAS	5.2	43,500	105,100	5.0	63,400	150,700
Primary Filters & Traditional WAS	4.8	45,300	119,800	4.6	66,300	173,500
	Maximum Month Conditions					
Traditional Primary & WAS	4.9	74,900	183,200	5.1	109,300	272,100
Primary Filters & Traditional WAS	4.5	81,500	218,000	4.7	119,300	322,200

Table 6.11-6 East Side WWTP Combined Thickened Solids Quantity: Initial and Design Year

	Initial Year (2025)			Design Year (2050)		
	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)	Solids Content (%)	Loading (lbs/day)	Volume (gal/day)
	Average Day Conditions					
Traditional Primary & WAS	5.3	7,600	17,300	5.3	13,300	30,100
Primary Filters & Traditional WAS	4.7	7,200	18,400	4.7	12,100	32,000
	Maximum Month Conditions					
Traditional Primary & WAS	5.1	12,000	28,300	5.1	21,000	49,700
Primary Filters & Traditional WAS	4.6	12,000	31,700	4.6	21,100	55,700

6.11.3 Sludge Thickening

6.11.3.1 Design Criteria

Thickening Operations and Performance

Both the West Side and East Side WWTPs are operated continuously, 24-hours a day, 7 days a week, however, thickening operations are not typically continuous. Many thickening technologies are not adept at handling large fluctuations in loading. Similarly, sludge wasting from the secondary process may not be constant (or even continuous). As a result, thickening systems are typically designed to handle future maximum month loading conditions, operating only a portion of the time. Unless otherwise noted, thickening equipment will be sized to handle maximum month loads while operating approximately 72 hours per week (12 hours per day, 6 days a week) under design year conditions. Fluctuations in load or sludge quantity can be accommodated by adjusting the hours of operation. In cases where WAS thickening is not intended to be continuous, WAS storage will be provided to allow operators to waste sludge from the secondary process (to maintain SRT) when thickening equipment is not operating. In such cases, enough volume will be provided to store WAS for one day at future maximum day conditions. During the initial years of operation and under future average day conditions, it is likely that thickening could be accomplished during a single 8-hour shift.

The current contract with Synagro for the hauling and disposal of thickened sludge requires that the hauled sludge have a solids content between 2 and 6 percent. For the purposes of thickening equipment sizing and selection, a goal of 5% solids content was set for combined thickened sludge. Individual thickening performance varies depending on the feed sludge characteristics, equipment type and equipment operating parameters such as polymer usage and hours of operation. Tankage will be provided to store thickened sludge (at 5 percent solids) prior to hauling. Sufficient volume will be provided to store up to three days of sludge production at future maximum month loading conditions. This volume will ensure that the West Side and East Side WWTPs will be able to continue thickening operations, at design capacity, even if there are disruptions in sludge hauling operations, due to holidays, maintenance, or other reasons.

Equipment Redundancy

Providing means to thicken at all times is essential to maintain operation of the plants. As such, redundant units will be provided for thickening and supporting equipment (pumps, aeration, polymer, etc.) to maintain normal operation with one unit out of service.

6.11.3.2 Thickening Technologies

A summary of the thickening processes considered for the East Side and West Side WWTPs residuals management are discussed below and summary in **Table 6.11-15**.

Dissolved Air Flotation Thickeners (DAFTs)

Dissolved air floatation is a technique that can be applied to either clarify liquids or to concentrate solids. In contrast with many other thickening technologies discussed in this section, DAFTs achieve solids-liquid separation by introducing fine air bubbles into the liquid sludge. The solids attach to the bubbles and rise to the surface where the solids are removed. A DAFT process, shown in **Figure 6.11-1**, requires a tank and a water/air pressurization system. The pressurization system consists of recycle pumps, air compressors, air saturation tanks, polymer addition, and control and pressure relief valves.

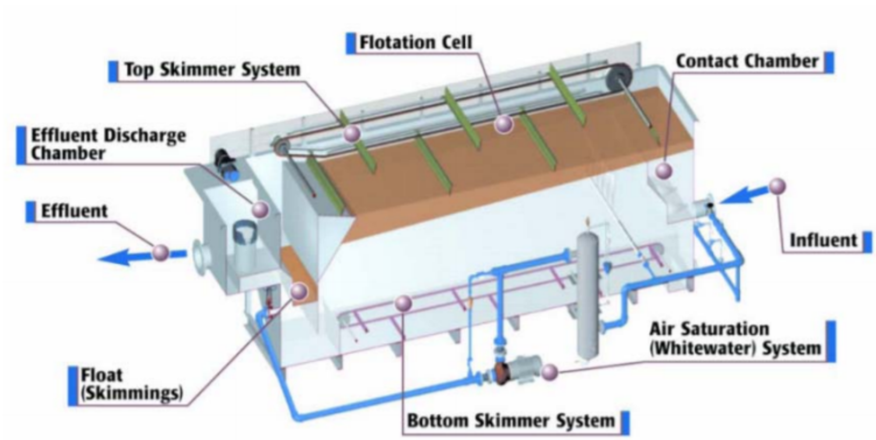


Figure 6.11-1
Dissolved Air Floatation Thickener (DAFT)

DAFTs are typically used for WAS thickening but can also be applied to co-thickening of primary and secondary solids. Thickening performance of DAFTs varies based upon the specific application, but often units can achieve 95% solids removal rates without polymer addition and can achieve up to 99% removal with polymer.

New DAFT systems are less frequently recommended today for sludge thickening applications because other technologies are available that are simpler to operate, more space efficient, and more contained than DAFTs. For these reasons, this option will not be considered further.

Gravity Belt Thickeners (GBTs)

Figure 6.11-2 shows the operation of a typical enclosed GBT. Typically, GBTs consist of one drive motor rotating a porous belt at a slow speed across several rollers. Sludge (with polymer) enters a conditioning tank on one side of the unit before being spread across the width of the porous belt. The top of the machine contains a series of plows that help spread the sludge and force the sludge to release water as it travels on the belt to the discharge end of the unit. Sludge solids are retained on top of the belt, while water (filtrate) falls to a sump or pan below the belt. Wash water is needed continuously to clean the belt while the equipment is running, and booster pumps are often needed to increase the pressure of the wash water.

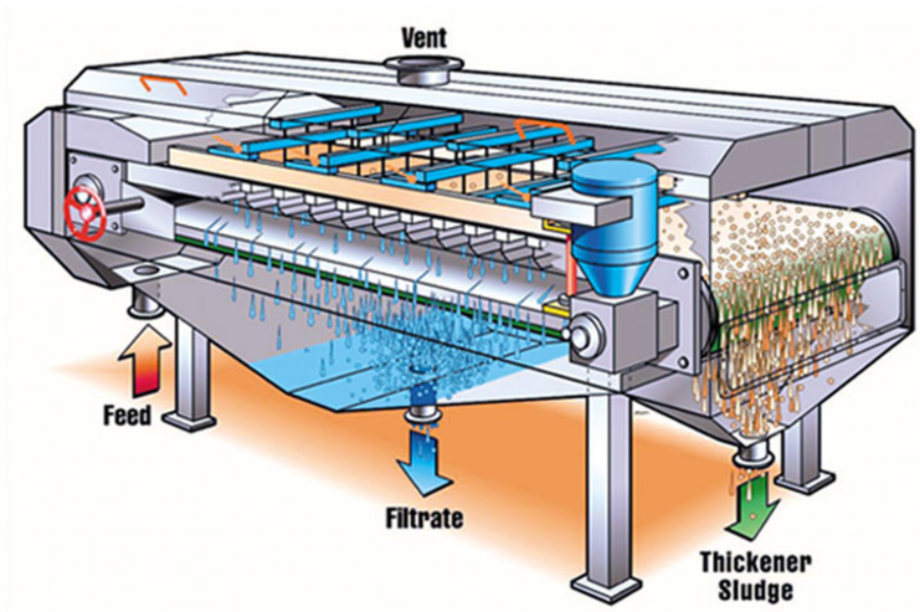


Figure 6.11-2
Gravity Belt Thickener (GBT)

GBTs are currently utilized to thicken a wide variety of feed sludge but are probably most often used to thicken WAS. Operators observe the sludge as it moves across the belt and can visually note thickening performance and make real-time adjustments to polymer addition and other control variables such as belt tension, speed and plow position. **Table 6.11-7** displays typical performance data for various feed sludges.

Table 6.11-7 Typical Gravity Belt Thickener Performance (per WEF MOP-8)

Type of Sludge	Feed Sludge Solids Concentration	Target Thickened Sludge Solids Concentration	Polymer Dose	Typical Solids Capture
Primary	1 – 4%	7 – 10%	5 – 10 active lb/dry ton	95 – 99%
Activated Sludge	0.5 – 1%	4 – 8%	5 – 10 active lb/dry ton	95 – 99%
Primary and Activated Sludge	1.5 – 3%	5 – 9%	5 – 10 active lb/dry ton	95 – 99%

GBTs are most commonly used for WAS thickening, though thickening of blended sludge is also practiced. GBTs can be susceptible to blinding of the belt if large quantities of scum are introduced to the process. Due to the modest size, cost, and simplicity of O&M, GBTs are an economical means of thickening for facilities of all sizes. GBTs can also be furnished fully enclosed. A fully enclosed unit would reduce odor issues, but access panels would need to be opened to observe thickening performance.

GBTs are typically offered in a number of different sizes, varying the belt width, typically 0.6, 1.0, 1.5, 2.0, or 3.0-meter widths. Each unit is typically sized based on maximum solids loading rate and maximum hydraulic loading rate. If the feed sludge is particularly high or low in solids content, then one loading rate will dictate the design, but it is important to consider both when selecting the equipment. As noted previously, the East Side WWTP has been operating an open 1.0-meter GBT to thicken WAS only for the last 25-years.

Typical design parameters are shown in **Tables 6.11-8** for West Side and East Side WWTPs WAS thickening for planning purposes.

Table 6.11-8 Gravity Belt Thickener Unit Data

Parameter	West Side WWTP Value		East Side WWTP Value	
	BDP	Alfa-Laval	BDP	Alfa-Laval
Belt Width	3.0m	3.0m	1.0m	0.6m
Number of Units	Two (1 Duty, 1 Standby)	Two (1 Duty, 1 Standby)	Two (1 Duty, 1 Standby)	Two (1 Duty, 1 Standby)
Hydraulic Loading Rate	800-1000 gpm	840 gpm	100-200 gpm	130 gpm
Typical Polymer Use	10 lb/DT	10 lb/DT	10 lb/DT	10 lb/DT

Rotary Drum Thickeners (RDTs)

RDTs, shown in **Figure 6.11-3**, consist of a sludge conditioning tank with mixer, rotary drum screen, internal screw or conveying flights, spray-wash system, filtrate collection pan and a variable speed drum drive. Before entering a rotating cylindrical screen, feed sludge is conditioned with polymer. The flocculated solids enter the drum where they are separated from the water. The solids continue to travel along the bottom of the rotating drum while water (filtrate) drains into a collection pan below, until sludge is collected in a hopper at the end of the unit.

RDTs are most often used for WAS thickening, however, as the technology gains popularity, RDTs are increasingly being utilized for primary sludge and co-thickening applications. The ability of RDTs to thicken is largely dependent on the feed sludge characteristics and performance can be optimized through changes in polymer dose and drum speed. Porosity of the screen drum can only be changed by completely replacing the assembly. **Table 6.11-9** displays typical performance data for various feed sludges.

Table 6.11-9 Typical Rotary Drum Thickener Performance (per WEF MOP-8)

Type of Sludge	Feed Sludge Solids Concentration	Target Thickened Sludge Solids Concentration	Polymer Dose	Typical Solids Capture
Primary	1 – 4%	7 – 12%	5 – 15 active lb/dry ton	93 – 98%
Activated Sludge	0.5 – 1%	4 – 7%	5 – 15 active lb/dry ton	93 – 98%
Primary and Activated Sludge	1.5 – 3%	5 – 9%	5 – 15 active lb/dry ton	93 – 98%

Similar to GBTs, the drum screens are susceptible to blinding when handling excessive quantities of scum and grease that often accompanies primary sludge. Wash water is needed continuously to clean the rotating screens during equipment operation, and booster pumps are often needed to increase the water pressure. RDTs are always completely enclosed, so odor control is relatively easy to implement. RDTs are relatively simple machines with few moving parts, and those moving parts operate at a slow and safe speed.

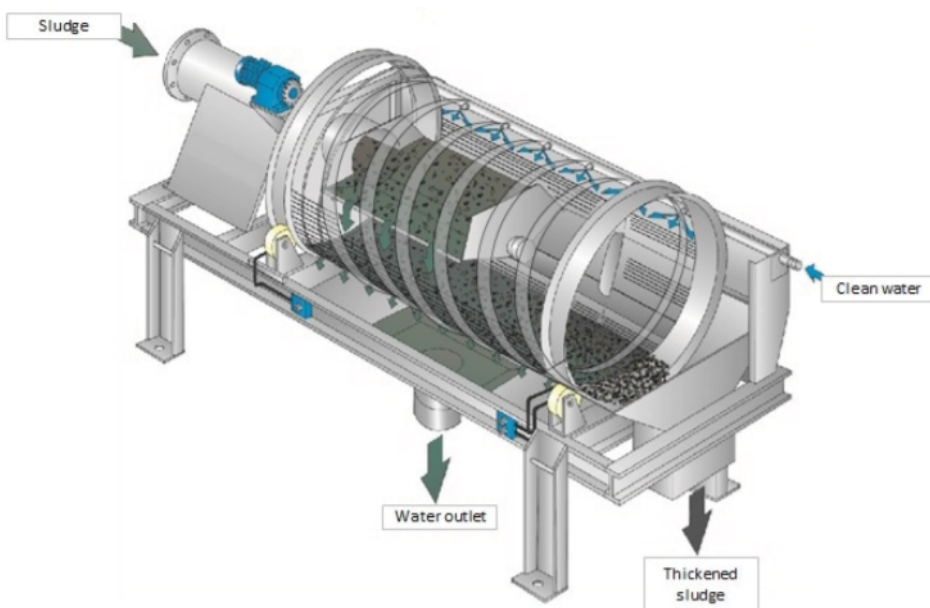


Figure 6.11-3
Rotary Drum Thickener (RDT)

RDTs are typically offered in a number of different sizes, varying the drum width, length and the number of drums in a unit. Similar to GBTs, RDTs are typically sized based on maximum solids loading rate and maximum hydraulic loading rate. If the feed sludge is particularly high or low in solids content, then one loading rate will dictate the design, but it is important to consider both when selecting the equipment.

A dual-drum RDT was installed in the lower level of the existing Solids Handling Facility at the West Side WWTP in June of 2020. Very limited data has been gathered up to this point, but initial information from the WPCA suggests that it is operating well, and the operators are pleased with the equipment. RDTs are usually best suited for small to medium size facilities such as the West Side and East Side WWTPs.

Typical design parameters are shown in **Tables 6.11-10** for West Side and East Side WAS thickening for planning purposes.

Table 6.11-10 Rotary Drum Thickener Unit Data

Parameter	West Side WWTP Value		East Side WWTP Value	
	BDP	Alfa-Laval	BDP	Alfa-Laval
Model Name	Dual 4x10	G3 Mega Duo	4x10	G3 Maxi
Description	Dual Drum	Dual Drum	Single Drum	Single Drum
Number of Units	Three (2 Duty, 1 Standby)	Three (2 Duty, 1 Standby)	Two (1 Duty, 1 Standby)	Two (1 Duty, 1 Standby)
Hydraulic Loading Rate	500 gpm	550 gpm	250 gpm	130 gpm
Typical Polymer Use	10-18 lb/DT	4-10 lb/DT	10-18 lb/DT	4-10 lb/DT

Thickening Centrifuges

Centrifuges, shown in **Figure 6.11-4**, rotate a cylindrical bowl at high speeds (greater than 1500 rpm) on a horizontal axis to force a separation of the liquid and solid phase. Sludge is fed into the bowl and centrifugal force is used to drive solids to the outer wall of the bowl. Solids are then collected by an internal scroll conveyor and brought to a solids discharge chute at the end of the centrifuge. Free water (centrate) is collected in the center of the bowl and discharged from the unit by gravity.

Centrifuges have a high degree of operational flexibility and can achieve higher thickened solids concentrations than some of the other thickening technologies discussed in this section.

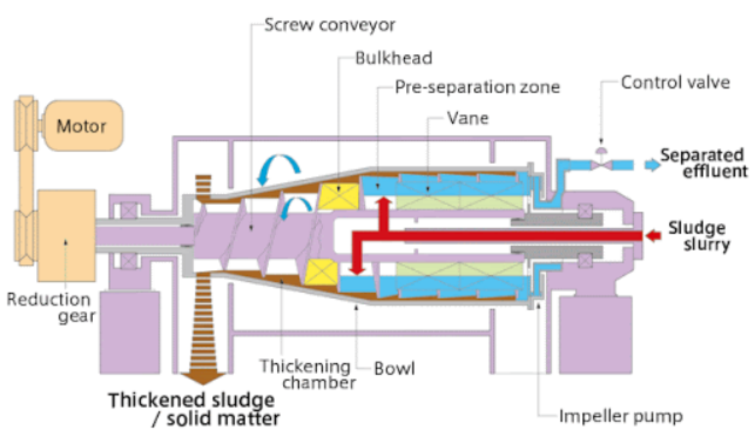


Figure 6.11-4
Thickening Centrifuge

Table 6.11-11 shows typical performance data for various feed sludges. Polymer dose and scroll differential speed (solids removal from bowl) can be adjusted to achieve desired thickening performance. While polymer may not be required it can have a significant impact on the overall solids capture of the system, reaching as high as 95 percent.

Table 6.11-11 Typical Thickening Centrifuge Performance (per WEF MOP-8)

Type of Sludge	Feed Sludge Solids Concentration	Target Thickened Sludge Solids Concentration	Polymer Dose	Target Centrate Quality
Primary	0.2 – 4%	4 – 6%	0 – 5 active lb/dry ton	<500 mg/L
Activated Sludge	0.4 – 1.5%	5 – 6%	0 – 6 active lb/dry ton	<500 mg/L
Primary and Activated Sludge	0.2 – 5%	5 – 6%	0 – 10 active lb/dry ton	<500 mg/L

Centrifuge operation is typically continuous due to long start up, time for the bowl to get up to speed, and shut down, time for the bowl to slow to a stop. Similarly, the high operating speeds produce high noise levels and often require special structural considerations to address vibration and dynamic loads. Accommodating these structural requirements can be particularly difficult in building retrofit situations. When operating in a stable condition, centrifuges require minimal operator attention. However, high operating speeds can also lead to imbalances within the unit that can result in large amounts of vibration that can damage the units. As a result, operators must undergo a robust training program to learn how to operate and maintain centrifuges and ancillary equipment. Because of their complexity and high capital and operating expense, centrifuges are generally only implemented for sludge thickening at larger facilities.

Centrifuges require a small footprint in comparison to many other thickening technologies and are entirely enclosed, so odor issues are mitigated, and cleanliness is maintained. While centrifuges are a highly proven technology for sludge thickening, they will not be considered further due to the high level of operator attention and operating expense.

Gravity Thickeners (GTs)

Gravity thickeners, seen in **Figure 6.11-5**, settle out solids in circular tanks. Similar to a clarifier, arms rotating along the tank bottom scrape thickened sludge toward the tank center, from which it is pumped out of the tank. Supernatant passes over weirs at the top of the tank's perimeter and into an effluent launder and is typically returned to the head of the plant. Typical gravity thickener performance can be seen in **Table 6.11-12**.

Table 6.11-12 Typical Gravity Thickener Performance (per TR-16)

Type of Sludge	Solids Loading Rate (lb/ft2/day)	Percent Solids (Thickened)
Primary	20 – 30	5 – 10
Trickling Filter	8 – 10	3 – 6
Activated Sludge	4 – 8	2.5 – 3
Primary and Trickling Filter	10 – 12	5 – 9
Primary and Activated Sludge	6 – 10	3 – 6

As noted in the table, gravity thickeners are most effective when settling primary sludge, as they are able to handle high and variable solids loading and achieve relatively high thickened sludge solids content. This is due to the relative density of the particles as well as the overall solids content of the sludge. Gravity thickener supernatant suspended solids concentrations can vary widely, concentrations as low as 100 mg/L and as high as 2,500 mg/L TSS have been observed. Higher solids in supernatant will result in substantial solids quantities being recycled back into the overall treatment process stream.

Typical hydraulic loading rates for primary sludge thickening range between 400-800 gpd/ft^2 and for secondary solids thickening range between 100-200 gpd/ft^2. Hydraulic loading rates must remain high for primary sludge thickening to prevent septic conditions. In many cases, plant water may be added to keep hydraulic loading within this range. Maintaining hydraulic loading rates can also help reduce scum buildup on weirs. Gravity thickeners must maintain continuous operation to provide effective thickening.

GTs can operate effectively without the need for chemical addition, especially when thickening primary sludge. However, sludge can be conditioned with polymer or inorganic coagulants to improve thickening performance. Chemical can also be added to control the formation of odor (resulting from septic conditions), but it is often more common to provide tank covers and to vent the headspace to an odor control system.

Large tank volumes (and surface areas) required for gravity thickeners provide an opportunity for flow equalization, allowing the system to handle variations in both flow and solids loading with minimal operator intervention. GTs can also provide a certain amount of thickened sludge storage, which can be beneficial if other systems fail or require maintenance. However, large volumes (and surface areas) require a large physical footprint, which may be difficult to locate at congested sites.

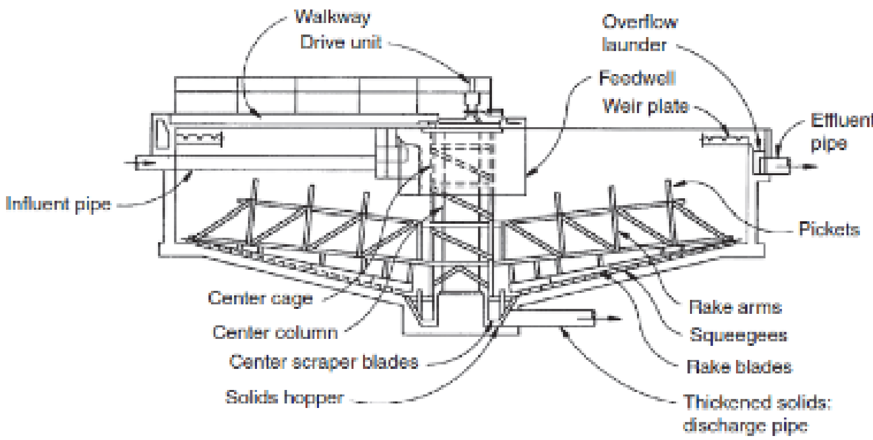


Figure 6.11-5
Gravity Thickener (GT) (Source: WEF MOP-8)

Gravity thickening is a proven technology and has been utilized at the West Side and East Side WWTPs for over 50-years and the WPCA has been using them almost exclusively for primary sludge thickening for the past 25-years.

Typical design parameters are shown in **Tables 6.11-13** for West Side and East Side WWTPs primary sludge thickening for planning purposes.

Table 6.11-13 Gravity Thickener Unit Data

Parameter	West Side WWTP Value		East Side WWTP Value	
	Traditional Primary	Primary Filtration	Traditional Primary	Primary Filtration
Tank Diameter	46'-0"	53'-0"	20'-0"	23'-0"
Tank Side Water Depth	13'-0"	13'-0"	13'-0"	13'-0"
Number of Units	Two (2 Duty)	Two (2 Duty)	Two (2 Duty)	Three (3 Duty)
Solids Loading Rate	20 lb/ft^2/day	20 lb/ft^2/day	20 lb/ft^2/day	13 lb/ft^2/day
Overflow Rate[1]	240 gpd/ft^2	610 gpd/ft^2	160 gpd/ft^2	622 gpd/ft^2

[1]-In instances where the overflow rate is below typical accepted ranges, supplemental plant water will be provided.

Emerging Thickener Technologies

Disk Thickeners – Disk thickeners utilize gravity and a slowly rotating, inclined disk filter to separate solids from the liquid phase. Conditioned sludge settles on the filter surface, allowing filtrate to pass through as the rotation carries the thickened sludge to the collection zone for removal. Disk filters are typically used for WAS thickening but can also be used for primary sludge thickening. Units can typically achieve thickened sludge solids content between 5 and 7 percent, and typically achieve capture rates exceeding 95 percent.

Volute/Screw Thickeners – Volute/screw thickeners utilize a cylindrical casing with openings and a rotating screw. As the conditioned sludge is pushed along, water drains through the casing and a thickened sludge is discharged out of the end of the unit. A volute thickener differs from a screw thickener with regards to the casing. In a volute thickener, the casing around the screw is made up of individual plates with gaps between them. These plates move continuously to ensure that the gaps between them do not clog. A screw press utilizes a perforated or wire mesh drum that requires backwashing. Volute/screw thickeners are typically used in WAS thickening application and will achieve thickened sludge concentrations between 4 and 8 percent. However, the presence of primary sludge will increase final solids output.

Membrane Thickeners – Membrane thickeners utilize either flat sheet or hollow fiber membranes to separate permeate from the WAS. Pumps draw the permeate through the membrane until a thickened sludge concentration is reached. Membrane thickening can produce thickened sludge up to 4 percent solids. Membrane thickening does not require polymer but does require an aeration system to mix the tank and to scour the membranes.

While emerging technologies may offer some potential benefits to the thickening operations at the West Side and East Side WWTPs, not enough empirical evidence exists to prove this equipment at such a large scale. Additionally, the use of proprietary equipment may have a substantial impact on the typical operations and maintenance of the systems.

6.11.3.3 Thickening Evaluation and Screening

All of the thickening technologies previously described were evaluated on a non-economic basis for a number of factors. These factors include success at other installations/reliability; site utilization; maintenance of plant operations; ease of operation; ease of maintenance; neighborhood impacts; ability to phase implementation; sludge impacts; energy efficiency; and chemical handling/hazards. **Table 6.11-14** includes the detailed non-economic evaluation of the various thickening technologies. As a result of this evaluation, DAFTs, Thickening Centrifuges and all emerging technologies were excluded from any further consideration. A summary of the remaining potential thickening technologies and their advantages and disadvantage is presented in **Table 6.11-15**.

Insert Table 6.11-14

Table 6.11-14 Thickening Alternatives Non-Economic Evaluation

Alternative Process Description	Dissolved Air Floatation Thickener	Gravity Belt Thickener	Rotary Drum Thickener	Thickening Centrifuge	Gravity Thickener
Non-Economic Criteria					
Success at Other Installations/ Reliability	Common process used at small and medium-scale facilities, often for combined sludge thickening. Units have recently started to be replaced with other mechanical thickening systems. While DAFTs have been reliable in the past, new installations are less common.	Common process used at small and large-scale facilities often for WAS thickening. Successful installations throughout New England and the U.S. with sizes comparable to East & West Side WWTPs. Currently in use at the East Side.	Slightly less common process, newer technology, used at small and medium scale facilities often for WAS thickening. Successful installations (more limited) throughout New England with sizes comparable to East & West Side WWTPs. Currently in use at the West Side.	Common process used at large-scale facilities, often for primary sludge thickening throughout New England and the U.S. with sizes comparable and larger than the West Side WWTP. Successful installations when the centrifuges are sized according to recommended industry guidelines.	Common process used at small and large-scale facilities, often for primary sludge thickening throughout New England and the U.S. with sizes comparable and larger than East & West Side WWTPs. Successful installations when the thickener units are sized according to recommended industry guidelines.
Site Utilization	Relatively compact process footprint. Systems for East and West Side WWTPs would vary from 2-3 units of varying footprint. Unit processes can be increased or decreased in size by modifying the operating hours.	Compact process footprint. Systems for East and West Side WWTPs would vary from 2-3 units of varying footprint. Unit processes can be increased or decreased in size by modifying the operating hours.	Compact process footprint. Systems for East and West Side WWTPs would vary from 2-3 units of varying footprint. Unit processes can be increased or decreased in size by modifying the operating hours.	Smallest process footprint. Unit processes can be increased or decreased in size by modifying the operating hours.	Largest process footprint with largest tank area. Systems for East and West Side WWTPs would vary from 2-3 circular tanks with separate gallery for ancillary equipment.
Maintenance of Plant Operations	Co-thickening of Primary Sludge in the existing gravity thickeners at West and East Side WWTPs and direct hauling may be required while the existing solids handling facilities are demolished and new facilities are constructed. The West Side has previously co-thickened in the gravity thickeners with some success.	Co-thickening of Primary Sludge in the existing gravity thickeners at West and East Side WWTPs and direct hauling may be required while the existing solids handling facilities are demolished and new facilities are constructed. The West Side has previously co-thickened in the gravity thickeners with some success.	Co-thickening of Primary Sludge in the existing gravity thickeners at West and East Side WWTPs and direct hauling may be required while the existing solids handling facilities are demolished and new facilities are constructed. The West Side has previously co-thickened in the gravity thickeners with some success.	Co-thickening of Primary Sludge in the existing gravity thickeners at West and East Side WWTPs and direct hauling may be required while the existing solids handling facilities are demolished and new facilities are constructed. The West Side has previously co-thickened in the gravity thickeners with some success.	East Side gravity thickeners can remain operational with minimum interruption in service due to unit redundancy. At West Side, new gravity thickeners will be constructed prior to demolishing the existing to maintain primary sludge thickening operations.
Ease of Operations	Medium complexity - Thickening can be optimized by adjusting sludge feed rate, polymer dose, air-to-solids ratio, and skimmer rake speed. Once performance is optimized, operators only need to check on the system occasionally.	Less complex - Thickening can be optimized by adjusting sludge feed rate, polymer dose and belt speed. Once performance is optimized, operators only need to check on the system occasionally.	Less complex - Thickening can be optimized by adjusting sludge feed rate, polymer dose and drum rotation speed. Once performance is optimized, operators only need to check on the system occasionally.	Operating centrifuges for thickening can be relatively simple when operations are continuous and staff is sufficiently trained. Start-up and shut-down takes a substantial amount of time due to the high rotating speeds. Centrifuges can experience unstable operation and may need to shut down to reduce excessive vibration.	Least complex - minimal equipment. Thickening is all by gravity without any chemical addition. Units operate continuously. Thickening performance is optimized by altering primary sludge pumping rate or through the addition of plant water.
Ease of Maintenance	Medium complexity - Skimmer system may require occasional cleaning and pressurized air system adds complexity.	Less complex - minimal equipment. Belt drive roller system may require infrequent maintenance. Hydraulic belt tensioning system adds complexity but rarely requires maintenance. Belt replacement is labor intensive.	Less complex - minimal equipment. Drum drive and end bearings may require infrequent maintenance.	Maintenance of centrifuges is complex and typically conducted by the manufacturer. Relying on a manufacturer can result in significant down time and expense.	Least complex - minimal equipment. Center-column drive system is identical to a circular clarifier drive, low HP and slow speed with substantial dear reduction. Bearing and gearbox lubricant level should be maintained.
Neighborhood Impacts	Thickener units will be located within the solids handling facility. All processes and thickening rooms shall be vented to odor control and should not be a major source of odor in the neighborhood.	Thickener units will be located within the solids handling facility. All processes and thickening rooms shall be vented to odor control and should not be a major source of odor in the neighborhood.	Thickener units will be located within the solids handling facility. All processes and thickening rooms shall be vented to odor control and should not be a major source of odor in the neighborhood.	Thickener units will be located within the solids handling facility. All processes and thickening rooms shall be vented to odor control and should not be a major source of odor in the neighborhood.	Thickener units should be covered and vented to an odor control system. Without these provisions, gravity thickening would be a significant source of odor in the neighborhood.
Ability to Phase Implementation	Implementation can be phased to spread out capital investment over a few years. Installation can be for initial year maximum month with space provided for an additional unit when flows/loads require in the future.	Implementation can be phased to spread out capital investment over a few years. Installation can be for initial year maximum month with space provided for an additional unit when flows/loads require in the future.	Implementation can be phased to spread out capital investment over a few years. Installation can be for initial year maximum month with space provided for an additional unit when flows/loads require in the future.	Implementation can be phased to spread out capital investment over a few years. Installation can be for initial year maximum month with space provided for an additional unit when flows/loads require in the future.	Given large unit footprints and a congested site, a gravity thickening system could not be expanded if plant flow/loads were to increase.
Sludge Impacts	Well suited for primary sludge and WAS thickening, can achieve thickened sludge solids content as high as 6%. Not well suited to large fluctuations in sludge feed quantity or solids content.	Best suited for WAS thickening, can achieve thickened sludge solids content as high as 8%. Not well suited to large fluctuations in sludge feed quantity or solids content.	Best suited for WAS thickening, can achieve thickened sludge solids content as high as 7%. Not well suited to large fluctuations in sludge feed quantity or solids content.	Well suited for primary sludge and WAS thickening, can achieve thickened sludge solids content as high as 6%. Not well suited to large fluctuations in sludge feed quantity or solids content.	Large tankage can handle significant variation in sludge quantity and solids content without operator intervention. Typically cannot provide thickened sludge as high in solids as other mechanical methods. Upsets in performance can lead to a significant recycle of solids back into the process stream.
Energy Efficiency	Medium energy requirement - slow skimmer speed requires little energy, but the pressurized air system can require a substantial amount of energy.	Low energy requirement - low speed drum drive, polymer and washwater systems.	Low energy requirement - low speed drum drive, polymer and washwater systems.	High energy requirement - high speed centrifuge drive.	Lowest energy requirement - slow speed drive mechanism.
Chemical Handling/Hazards	Requires polymer addition and a pressurized air system.	Requires polymer addition and a washwater booster system.	Requires polymer addition and a washwater booster system.	Requires polymer addition.	Chemicals are not typically required for primary sludge thickening systems. Supplemental plant water may be required to boost overflow rates.

Table 6.11-15 Thickener Technology Advantages and Disadvantages

Advantages	Disadvantages
Gravity Belt Thickener	
▪ Proven technology ▪ Simple O&M ▪ Operator familiarity ▪ Compact footprint ▪ High degree of thickening achievable ▪ Visual performance monitoring ▪ Enclosures for Odor Control (optional)	▪ Requires more consistent sludge feed (flow and solids content) ▪ Ancillary equipment required: Polymer storage and feed system and wash water booster pump system ▪ Prone to belt blinding when handling primary/secondary scum and grease
Rotary Drum Thickener	
▪ Proven technology ▪ Simple O&M ▪ Operator familiarity ▪ Compact footprint ▪ High degree of thickening achievable ▪ Enclosures for Odor Control	▪ Requires more consistent sludge feed (flow and solids content) ▪ Ancillary equipment required: Polymer storage and feed system and wash water booster pump system ▪ Prone to belt blinding when handling primary/secondary scum and grease ▪ Typically use more polymer to achieve the same degree of thickening as a GBT
Gravity Thickener	
▪ Proven technology ▪ Simple O&M ▪ Operator familiarity ▪ Handles large variations in flow and solids content ▪ Good performance with thin sludge ▪ Minimal to no chemical use	▪ Larger footprint ▪ Thinner final solids content ▪ Higher solids content in supernatant ▪ Produces odors (needs large covers) ▪ Scum/Grease buildup ▪ May require plant water to maintain overflow rates

Discussions with the WPCA revealed a preference to maintain gravity thickening for primary sludge based on the ability to handle large fluctuations in solids and hydraulic loading expected at the plants given the combined collection system. GTs are also well suited for “thin” primary sludge that is expected from primary filter backwash operations. It was also noted that the WPCA preferred to maintain their existing RDT technology that had recently gone into operation at the West Side WWTP. Lastly, there was a strong preference to maintain similar technologies between the two plants. These thickening alternatives, gravity thickeners for primary sludge and RDTs for waste activated sludge thickening are described in greater detail in Section 7.

6.11.3.4 Summary and Recommendations

Maintain and rehabilitate existing gravity thickeners or construct new gravity thickeners for thickening of primary sludge at both the West Side and East Side WWTPs. Employ RDTs at both plants for the thickening of waste activated sludge. Provide waste activated storage for one day of storage at future maximum day sludge production and provide three days of thickened sludge storage at future maximum month production. Continue hauling thickened sludge off-site for further treatment and disposal.

6.11.4 Dewatering

6.11.4.1 Design Approach

The goal of sludge dewatering is to produce a semi-solid cake with a solids content in excess of 20% for hauling to off-site disposal facilities. This can significantly reduce the number of truck trips to the disposal site and can in some cases also result in reduced disposal fees depending on the facility. Dewatering is more effective when sludge has already been thickened, and as such, all dewatering alternatives assume thickening preceding dewatering.

Dewatering Operations and Performance

As with thickening, dewatering operations are not typically continuous at treatment plants of this size. Unless otherwise noted, thickening equipment will be sized to handle design-year maximum month loads while operating no more than 72 hours per week (12 hours per day, 6 days a week). This is intended to match the operating time for thickening processes, to allow operators to observe and maintain both processes within the same facility. Any fluctuations in sludge quantity can be accommodated by adjusting the hours of operation. During the initial years of operation and under future average day conditions, dewatering could likely occur within a single 8-hour shift.

Current contracts with Synagro for the hauling and disposal of thickened sludge requires that the hauled sludge have a solids content between 2 and 6 percent. This contract would need to be renegotiated if dewatering is to be implemented.

Equipment Redundancy

Providing means to dewater at all times may not be essential to maintain operation of the plants. Hauling contracts can be negotiated to include provisions to allow for liquid hauling during emergency situations. As such, redundant units may not be required. However, due to perceived volatility in the sludge disposal market with regards to dewatering, alternatives discussed later in this section will include full redundancy in dewatering and supporting equipment (pumps, aeration, polymer, etc.) to maintain normal operation with one unit out of service.

Site Location

Dewatered cake sludge is very difficult to pump, and conveyors can be difficult to operate and maintain. For these reasons, truck access to the dewatering facility is essential. Dewatering operations are also typically conducted on an elevated level, to allow roll-off containers to be filled from above. For these reasons, any dewatering equipment would be located on a second level of a solids handling facility, above the thickening operations.

6.11.4.2 Dewatering Technologies

A summary of the thickening processes considered for the East Side and West Side WWTPs residuals management are discussed below and summarized in **Table 6.11-19**.

Belt Filter Presses (BFPs)

Belt filter presses typically consist of three separate zones, a gravity zone, low pressure zone and a high-pressure zone. Conditioned sludge first enters a gravity zone, which is very similar thickening zone of a GBT. Sludge solids are retained on top of the belt, while water (filtrate) falls to a pan below the belt. After the gravity zone, the partially dewatered sludge drops onto another porous belt, which then begins to converge with the other belt in the low-pressure zone before entering the high-pressure zone. In the high-pressure zone, the two belts are pressed together by a series of rollers to actively squeeze more water out of the sludge. This filtrate is collected in a second pan. Wash water is needed continuously to clean both belts while the equipment is running, and booster pumps are often needed to increase the pressure of the wash water. A schematic of a typical 2-belt BFP is shown in **Figure 6.11-6**. Some manufacturers offer a 3-belt configuration that uses a single belt dedicated to the gravity zone.

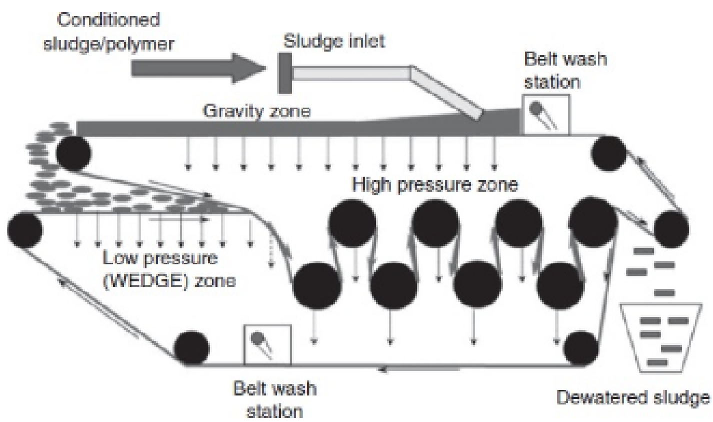


Figure 6.11-6
2-Belt Belt Filter Press (courtesy of Alfa-Laval)

Similar to a GBT, operators observe the sludge as it moves across the belt and can visually note dewatering performance and make real-time adjustments to polymer addition and other control variables such as belt tension, speed and plow position. **Table 6.11-16** displays typical performance data for various feed sludges. It should be noted that in applications similar to those proposed at the West Side and East Side WWTPs, dewatering performance will likely be greater, when handling previously thickened sludge.

Due to the modest size, cost, and simplicity of O&M, BFPs are an economical means of thickening for facilities of all sizes. Few manufacturers offer fully enclosed units so typical dewatering facilities with BFPs require robust ventilation and odor control systems.

BFPs are typically offered in a number of different sizes, varying the belt width, typically ranging from 0.5 to 3.5-meter widths. Each unit is typically sized based on maximum solids loading rate and maximum hydraulic loading rate. If the feed sludge is particularly high or low in solids content, then one loading rate will dictate the design, but it is important to consider both when selecting the equipment.

Table 6.11-16 Typical Belt Filter Press Performance (via WEF MOP-8)

Type of Sludge	Feed Sludge Solids Concentration	Target Thickened Sludge Solids Concentration	Polymer Dose	Typical Solids Capture
Primary	2 – 5%	10 – 20%	10 – 20 active lb/dry ton	95 – 98%
Activated Sludge	2 – 6%	10 – 20%	20 – 30 active lb/dry ton	92 – 96%
Primary and Activated Sludge	0.5 – 2%	10 – 20%	15 – 25 active lb/dry ton	94 – 97%

Dewatering Centrifuges

Dewatering centrifuges, shown in **Figure 6.11-7**, are nearly identical to thickening centrifuges, as previously discussed. While nearly all of the internal components and ancillary processes are identical, characteristics such as physical dimensions, loading rates, and polymer use may differ. **Table 6.11-17** shows typical performance data for various feed sludges. It should be noted that in applications similar to those proposed at the West Side and East Side WWTPs, dewatering performance will likely be greater, when handling previously thickened sludge. Polymer dose and scroll differential speed (solids removal from bowl) can be adjusted to achieve desired dewatering performance. While polymer may not be required it can have a significant impact on the overall solids capture of the system, reaching as high as 99 percent.

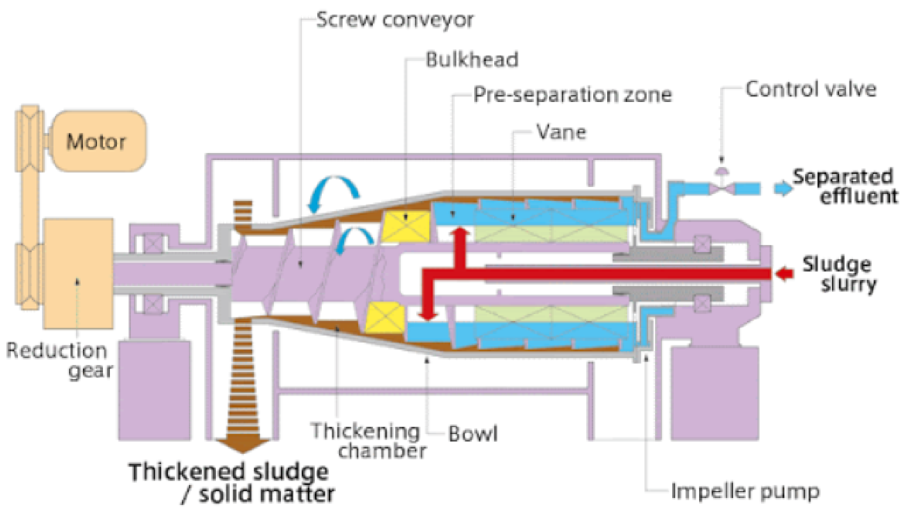


Figure 6.11-7
Dewatering Centrifuge

Table 6.11-17 Typical Dewatering Centrifuge Performance (per WEF MOP-8)

Type of Sludge	Target Thickened Sludge Solids Concentration	Solids Capture w/o Polymer	Solids Capture w/ Polymer
Primary	25 – 35%	75 – 90%	95+%
Primary and Activated Sludge	12 – 20%	55 – 65 %	92+%
Activated Sludge	5 – 15%	60 – 80%	95+%

Centrifuge operation is typically continuous due to long start up, time for the bowl to get up to speed, and shut down, time for the bowl to slow to a stop. Similarly, the high operating speeds produce high noise levels and often require special structural considerations to address vibration and dynamic loads. Accommodating these structural requirements can be particularly difficult in building retrofit situations. When operating in a stable condition, centrifuges require minimal operator attention. However, high operating speeds can also lead to imbalances within the unit that can result in large amounts of vibration that can damage the units. As a result, operators must undergo a robust training program to learn how to operate and maintain centrifuges and ancillary equipment. Because of their complexity and high capital and operating expense, centrifuges are generally only implemented for sludge thickening at larger facilities.

Centrifuges require a small footprint in comparison to many other dewatering technologies and are entirely enclosed, so odor issues are mitigated, and cleanliness is maintained. While centrifuges are a highly proven technology for sludge dewatering, they will not be considered further due to the high level of operator attention and operating expense.

Rotary Presses

Rotary presses, shown in **Figure 6.11-8**, utilize gravity, friction and pressure differential to dewater solids. Conditioned sludge enters an interior channel that is bound by screens on each side. The channel is semi-circular, making a near 180-degree turn from inlet to outlet. As the sludge is forced through the channel, free water passes thought the screens (which also rotate) and is drained out of the bottom of the unit. The outlet of the channel is constricted, forcing sludge to build up and dewater further due to the added pressure and friction. Rotary presses are a dewatering technology that is not widely used but is gaining in popularity. Typical unit performance can vary significantly depending on polymer use, influent solids characteristics and dewatering application. **Table 6.11-18** displace some performance characteristics of select installations.

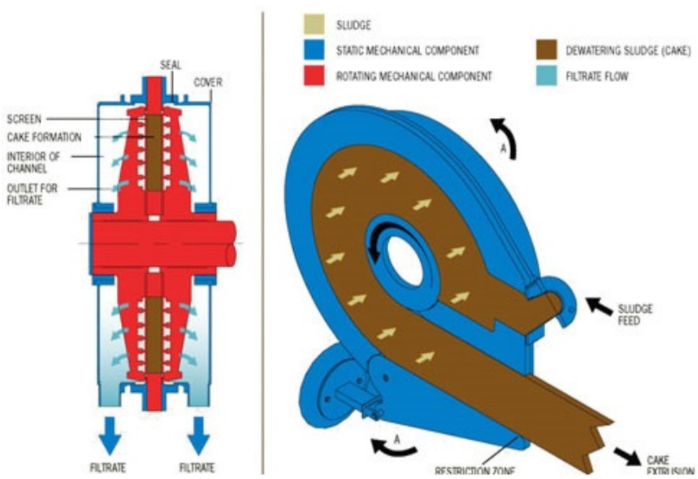


Figure 6.11-8
Rotary Press (courtesy of Fournier)

Table 6.11-18 Performance Characteristics of Rotary Presses (per WEF, 2012)

Facility Size	Solids Type	Influent Sludge Solids Content	Dewatered Sludge Solids Content
15 mgd	PS/WAS	0.8 - 1 %	12 – 14%
1.4 mgd	Thickened PS/WAS	3%	28%
19.8 mgd	Thickened PS/WAS	3 – 6%	19 – 25%
8.1 mgd	BNR Thickened	2.6%	20%
23 mgd	BNR Thickened	3.7%	19%

As seen in Table 6.11-18, rotary presses offer a substantial degree of dewatering, reducing sludge volumes to as little as 1/14 of initial liquid volume. Units are typically sized for both solids and hydraulic loading rates but are not offered in a variety of sizes. They are, however, modular. Up to eight units may be powered by a single drive motor powering a common shaft. This modularity also makes it possible to remove individual channels for servicing. Units are also fully enclosed, facilitating odor containment.

Rotary presses are typically a proprietary design and their small unit capacity would require a substantial number of units to be installed at the West Side WWTP, leading to high capital costs. For these reasons, rotary presses were not further evaluated for sludge dewatering.

Screw Presses

Screw presses are very similar to volute/screw thickeners, as previously described in thickening emerging technologies but often vary from manufacturer to manufacturer. Units can be oriented horizontally or at a slight incline (typically 10 to 20 degrees from horizontal). A typical horizontal screw press with cylindrical screen is shown in **Figure 6.11-9**. As with volute/screw thickeners, as the conditioned sludge is pushed along, water drains through the casing and dewatered sludge is discharged out of the end of the unit.

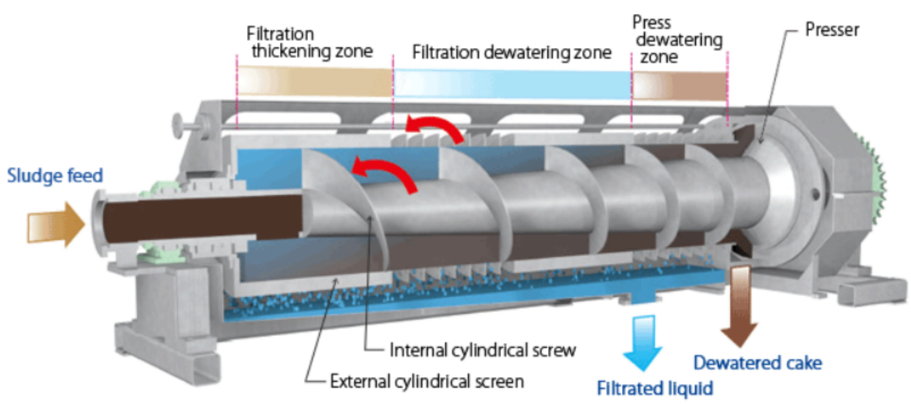


Figure 6.11-9
Screw Press

Screw presses are not widely used, and typical unit performance can vary significantly depending on polymer use, influent solids characteristics and dewatering application. **Table 6.11-19** presents some performance characteristics of select installations.

Table 6.11-19 Performance Characteristics of Screw Presses (via WEF, 2012)

Facility Size	Solids Type	Influent Sludge Solids Content	Dewatered Sludge Solids Content
1.0 mgd	PS/WAS	2.3 – 2.4%	19 – 20%
1.65 mgd	PS/WAS	1.98 – 2.4%	21.5%
1.85 mgd	WAS	0.5 – 1.5%	15 – 20%
2.0 mgd	WAS	2 - 3%	15 - 20%
4.5 mgd	WAS	0.5 - 1%	12 - 17%

As seen in Table 6.11-19, rotary presses offer a substantial degree of dewatering but are most often installed at smaller facilities. It should also be noted, that if installed at the West Side and East Side WWTPs after sludge thickening, the final solids content of the dewatered cake would likely be higher than is shown in Table 6.11-19. Units are typically sized for both solids and hydraulic loading rates but are not offered in large sizes. The fully enclosed design of a screw press provides good odor containment and is beneficial for housekeeping.

The small unit capacity of standard screw press designs would require a substantial number of units to be installed at the West Side WWTP, leading to high capital costs. For this reason, screw presses were not further evaluated for sludge dewatering.

6.11.4.3 Dewatering Technology Evaluation and Screening

All of the dewatering technologies previously described were evaluated on a non-economic basis, determining if a technology is deemed 'favorable, unfavorable, or neutral' for a number of factors. These factors include success at other installations/reliability; site utilization; maintenance of plant operations; ease of operation; ease of maintenance; neighborhood impacts; ability to phase implementation; sludge impacts; energy efficiency; and chemical handling/hazards. **Table 6.11-20** includes the detailed non-economic evaluation of the various thickening technologies.

As a result of this evaluation, dewatering centrifuges and screw presses were excluded from any further consideration. A summary of the remaining potential thickening technologies and their advantages and disadvantage is presented in **Table 6.11-21**.

Table 6.11-20 Dewatering Alternatives Non-Economic Evaluation

Alternative Process Description	Belt Filter Press	Dewatering Centrifuge	Rotary Press	Screw Press
Non-Economic Criteria				
Success at Other Installations/ Reliability	Common process used at small and large-scale facilities, often for combined sludge dewatering throughout New England and the U.S. Belt filter press technology is a reliable solution for dewatering.	Common process used at large-scale facilities, often for combined sludge dewatering throughout New England and the U.S. with sizes comparable and larger than the West Side WWTP. Successful installations when the centrifuges are sized according to recommended industry guidelines.	Less common process, newer technology, used at small and medium-scale facilities dewatering. Successful installations (more limited) throughout New England with sizes comparable to East Side WWTP.	Less common process, newer technology, used at small and medium-scale facilities dewatering. Successful installations (more limited) throughout New England with sizes comparable to East Side WWTP.
Site Utilization	Largest process footprint. Unit processes can be increased or decreased in size by modifying the operating hours. Dewatering solutions can be included in the solids handling facility as a second story.	Smallest process footprint. Unit processes can be increased or decreased in size by modifying the operating hours. Dewatering solutions can be included in the solids handling facility as a second story.	Small process footprint. Quantity of channels can be increased or decreased in size by modifying the operating hours. Dewatering solutions can be included in the solids handling facility as a second story.	Small process footprint. Unit processes can be increased or decreased in size by modifying the operating hours. Dewatering solutions can be included in the solids handling facility as a second story.
Maintenance of Plant Operations	Since neither West Side or East Side currently dewater sludge, there will be no impact to the maintenance of plant operations.	Since neither West Side or East Side currently dewater sludge, there will be no impact to the maintenance of plant operations.	Since neither West Side or East Side currently dewater sludge, there will be no impact to the maintenance of plant operations.	Since neither West Side or East Side currently dewater sludge, there will be no impact to the maintenance of plant operations.
Ease of Operations	Less complex - Dewatering can be optimized by adjusting sludge feed rate, polymer dose and belt speed. Once performance is optimized, operators only need to check on the system occasionally.	Operating centrifuges for dewatering can be relatively simple when operations are continuous and staff is sufficiently trained. Start-up and shut-down takes a substantial amount of time due to the high rotating speeds. Centrifuges can experience unstable operation and may need to shut down to reduce excessive vibration.	Less complex - Dewatering performance can be optimized by changing sludge feed, polymer dose and channel drive speed. Operation requires infrequent operator intervention.	Less complex - Dewatering performance can be optimized by changing sludge feed, polymer dose and screw drive speed. Operation requires infrequent operator intervention.
Ease of Maintenance	Less complex - minimal equipment. Belt drive roller system may require infrequent maintenance. Hydraulic belt tensioning system adds complexity but rarely requires maintenance. Belt replacement is labor intensive.	Maintenance of centrifuges is complex and typically conducted by the manufacturer. Relying on a manufacturer can result in significant down time and expense.	Less complex - minimal moving parts. Maintenance by the manufacturer may be required in come cases.	Less complex - minimal moving parts. Maintenance by the manufacturer may be required in come cases.
Neighborhood Impacts	Trucking of dewatered cake can be a significant source of odors, unless fully enclosed roll-offs are used. Dewatering units will be located within the solids handling facility. All processes and sludge areas shall be vented to odor control and should not be a major source of odor in the neighborhood.	Trucking of dewatered cake can be a significant source of odors, unless fully enclosed roll-offs are used. Dewatering units will be located within the solids handling facility. All processes and sludge areas shall be vented to odor control and should not be a major source of odor in the neighborhood.	Trucking of dewatered cake can be a significant source of odors, unless fully enclosed roll-offs are used. Dewatering units will be located within the solids handling facility. All processes and sludge areas shall be vented to odor control and should not be a major source of odor in the neighborhood.	Trucking of dewatered cake can be a significant source of odors, unless fully enclosed roll-offs are used. Dewatering units will be located within the solids handling facility. All processes and sludge areas shall be vented to odor control and should not be a major source of odor in the neighborhood.
Ability to Phase Implementation	Solids handling facility improvements can be built to accommodate thickening and retrofitted to include dewatering at a later date.	Solids handling facility improvements can be built to accommodate thickening and retrofitted to include dewatering at a later date.	Solids handling facility improvements can be built to accommodate thickening and retrofitted to include dewatering at a later date. Rotary press channels can easily be added to increase capacity with minimal impact on footprint.	Solids handling facility improvements can be built to accommodate thickening and retrofitted to include dewatering at a later date.
Sludge Impacts	Well suited to dewatering of combined thickened sludge, can typically achieve dewatered sludge with solids concentrations between 10-20%.	Well suited to dewatering of combined thickened sludge, can typically achieve dewatered sludge with solids concentrations between 12-20%.	Well suited to dewatering of combined thickened sludge, can typically achieve dewatered sludge with solids concentrations between 19-28%.	Well suited to dewatering of combined thickened sludge, can typically achieve dewatered sludge with solids concentrations between 15-20%.
Energy Efficiency	Low energy requirement - low speed belt drives.	High energy requirement - high speed centrifuge drive.	Very low energy requirement - slow speed drive mechanism.	Very low energy requirement - slow speed drive mechanism.
Chemical Handling/Hazards	Requires polymer addition and a washwater booster system.	Requires polymer addition.	Requires polymer addition.	Requires polymer addition and a washwater booster system (depending on screen or plate cylinder.

This page intentionally left blank.

This page intentionally left blank.

Table 6.11-21 Dewatering Technologies Advantages and Disadvantages

Advantages	Disadvantages
Belt Filter Press	
▪ Proven technology ▪ Simple O&M ▪ Compact footprint ▪ Visual performance monitoring	▪ Ancillary equipment required: Polymer storage and feed system and wash water booster pump system ▪ Requires more robust odor control due to lack of unit enclosures
Rotary Press	
▪ Compact footprint ▪ Enclosed for Odor Control	▪ Newer technology ▪ Ancillary equipment required: Polymer storage and feed system and wash water booster pump system ▪ Typically use more polymer to achieve the same degree of dewatering as a belt filter press
Screw Press	
▪ Compact footprint ▪ Enclosed for Odor Control	▪ Newer technology ▪ Ancillary equipment required: Polymer storage and feed system and wash water booster pump system ▪ Typically use more polymer to achieve the same degree of dewatering as a belt filter press

6.11.4.4 Dewatering Discussion

The addition of dewatering processes can significantly reduce the volume of solids requiring ultimate disposal. This reduction in volume can result in costs savings related to hauling and disposal but the addition of equipment adds both capital cost and additional operating expenses. A present worth analysis was conducted to determine the economic feasibility of adding dewatering at both the West Side and East Side WWTPs. There are, however, a number of other non-cost issues that require consideration prior to selecting dewatering.

Sludge Disposal Market

The ultimate disposal of wastewater treatment residual solids is often one of three alternatives: incineration (with ash to landfill), landfilling or beneficial reuse. **Figure 6-11.10** displays the sludge disposal facilities in New England. The current disposal market in Connecticut is dominated by incineration. The State of Connecticut currently has five operating biosolids incineration facilities: Hartford MDC, Mattabassett District, Waterbury, New Haven, and Naugatuck.

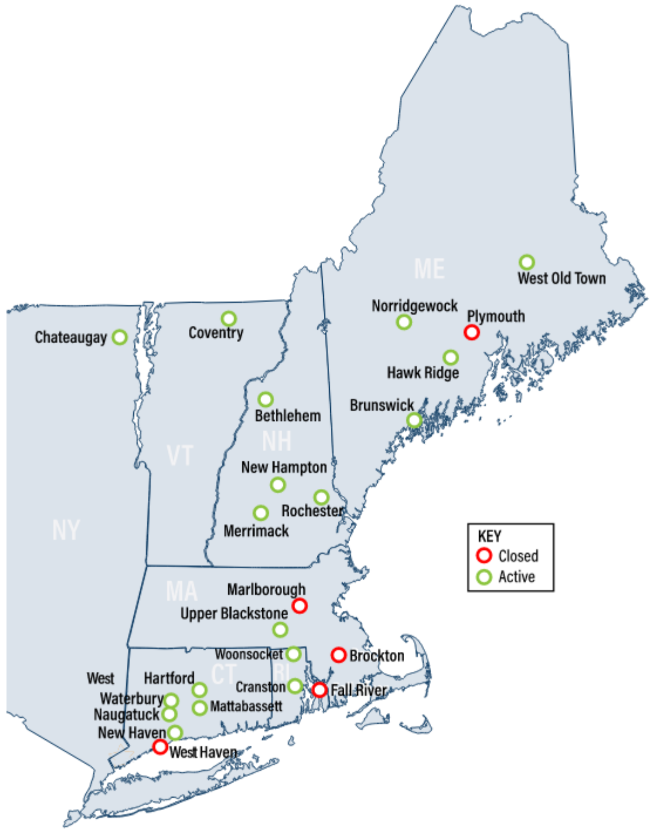


Figure 6.11-10
New England Biosolids Disposal Sites

The Metropolitan District, Hartford Connecticut (MDC) currently owns and operates a regional solids handling facility at the Hartford Water Pollution Control Facility (WPCF). The Hartford facility receives biosolids from as many as 60 customers and utilizes multiple hearth incineration to convert biosolids into inert ash. The Hartford facility has the ability to accept both liquid sludge and dewatered cake, but dewatered cake is rehydrated before addition to the sludge storage tanks. This helps to maintain uniformity in the sludge feed into the incinerator. The MDC incinerator facility has a capacity of approximately 120 dry tons per day. The Mattabassett District owns and operates a regional WPCF in Cromwell, CT. The Mattabassett WPCF operates three centrifuges to dewater sludge to a total solids content of 25 percent prior to incineration in a fluidized bed reactor, with a capacity of 36 dry tons per day. Synagro operates two of Connecticut's major merchant biosolids incinerators in Waterbury, CT and New Haven, CT. The Waterbury facility utilizes a fluidized bed reactor to incinerate up to 63 dry tons per day from as many as 40 area biosolids producers. The New Haven facility utilizes multiple hearth incineration to handle up to 40 dry tons per day. Thickened sludge from the West Side and East Side WWTPs is hauled by Synagro to their New Haven facility for dewatering and disposal. Veolia also operates a merchant facility in Naugatuck, CT. The fluidized bed reactor has a capacity of 85 dry tons per day and receives sludge from up to 30 other biosolids producers. Many of the incineration facilities in Connecticut only accept liquid sludge due to the difficulties present in the handling of dewatered cake. Incineration facilities also exist in neighboring Massachusetts and Rhode Island, but often the added trucking distance makes it economically infeasible to dispose at those sites. The existing incineration facilities in New England are currently strained, reaching capacity and continually facing pressure from stricter emissions regulations. Primarily due to increasing regulations, incinerator facilities throughout New England have been shutting down, and the construction of new facilities is not economically viable.

There are no biosolids landfills in the State of Connecticut, so sludge must be taken out-of-state for landfilling. Casella Organics and Waste Management own and operate a few landfills in Vermont, New Hampshire and Maine that accept biosolids, however it is unrealistic that the WPCA would dispose of liquid or dewatered biosolids at any of these sites due to exorbitant hauling costs.

Beneficial reuse through land application as a soil nutrient supplement is not currently allowed in the State. To meet reuse guidelines in neighboring states, additional stabilization processes must be implemented to further process biosolids. Today, state guidelines essentially dictate that biosolids be disposed of through incineration and that this sludge ash be landfilled. Given the fact that the sludge disposal market in New England is constantly changing, the WPCA may wish to consider processes to recover nutrients in the future.

Economic Evaluation

Capital improvements as included in the present worth evaluation related to thickening involve the construction of a new solids handling facility at both West Side and East Side WWTPs. Thickening of primary sludge was assumed to be achieved with gravity thickening (new units at the West Side WWTP and rehabbed units at the East Side WWTP). Thickening of secondary sludge was assumed to be achieved with rotary drum thickeners. Each Solids Handling Facility shall house RDTs and ancillary equipment, WAS and thickened sludge storage tanks and mixing systems, and sludge pumping. Dewatering, as included in the present worth analysis include: all thickening improvements previously noted; construction of a second level on the Solids Handling Facility; new belt filter presses; an additional polymer system dedicated to dewatering; larger wash water booster pump system; sludge cake screw conveyor system; and roll-off storage garage. Annual operations and maintenance costs were also developed. This included annual expected maintenance expenses, energy usage and expense, sludge hauling and disposal costs, polymer use, and costs associated with additional personnel needs. Annual maintenance was assumed to be 5% of equipment capital costs (per Section 2) and the addition of Dewatering was assumed to require one operator to dedicate at least half of their 8-hour shift to the dewatering operations.

In most cases, the costs associated with disposing of sludge is provided per dry ton of solids, and therefore the costs for disposing of thickened or dewatered sludge does not vary significantly. The savings related to providing sludge dewatering in addition to thickening is the reduced number of truck trips required to haul the sludge off-site to the disposal location. The further the disposal location the greater the cost differential.

Table 6.11-22 presents the results of the present worth analysis.

Table 6.11-22 Present Worth Analysis Results[1]

	Thickening Only	Thickening AND Dewatering
West Side WWTP		
Capital Improvements Present Worth	$20,000,000	$26,000,000
O&M Present Worth	$106,000,000	$77,000,000
Total Present Worth	$126,000,000	$103,000,000
East Side WWTP		
Capital Improvements Present Worth	$12,000,000	$16,000,000
O&M Present Worth	$29,000,000	$24,000,000
Total Present Worth	$41,000,000	$40,000,000

[1]All values are presented as 2020 dollars.

The analysis shows that the West Side WWTP could save up to $1.4 million annually by dewatering. This annual savings occurs by reducing the number of truck trips for hauling to between ¼ and 1/5 of the number of trips annually when hauling thickened sludge. This savings comes with an additional upfront cost of $8 million. While this still leads to an approximate savings of nearly $23 million over the next 20 years, there are many other non-cost factors that need to be considered before moving forward. Dewatering is not typically beneficial for smaller plants, and this analysis showed that the WPCA would not likely receive any substantial benefit if dewatering were to be implemented at the East Side WWTP. However, as discussed in Section 6.2, it could be beneficial to convey and treat the East Side plant sludge at the West Side Plant site, if not today, in the future.

Dewatering Decision

Discussions with the WPCA regarding the potential cost savings were presented alongside the potential issues with the current sludge disposal market. Much of the discussion centered around the inconsistency within the market and the potential issue of installing expensive dewatering equipment that may be mothballed before a breakeven point. It was ultimately decided that installation of dewatering processes would not be considered for the initial round of plant improvements. However, provisions will be made in the structural design of the Solids Handling Facility such that a second level for dewatering can be added in the future. This will provide the WPCA the flexibility to adapt to the changing disposal market while also slightly reducing initial capital investments.

6.11.4.5 Summary and Recommendations

Defer providing dewatering systems at this time but design a West Side WWTP solids handling facility to accommodate dewatering if desirable in the future.

6.11.5 Sludge Stabilization

6.11.5.1 Approach

After thickening or dewatering processes, many larger facilities also use sludge stabilization and volume reduction technologies such as anaerobic digestion, drying/pelletizing, gasification, or incineration. Typically, sludge stabilization technologies are implemented at larger facilities because they require a large capital investment. If the WPCA was to incorporate sludge stabilization it would be beneficial to convey the sludge produced at the East Side plant and treat in a common facility at the West Side plant. If the decision to do this was made after both plants are upgraded sludge from the West Side plant could also be conveyed to the East Side plant. In addition, third party vendors exist who are willing to design-build-operate a residuals management system, on or adjacent to a wastewater treatment plant parcel. Contracts need to be carefully crafted to ensure the interests of both parties are managed and are equitable.

6.11.5.2 Anaerobic Digestion

At some facilities, particularly at medium and large WWTFs, sludge is anaerobically digested prior to dewatering. Anaerobic digestion is a biological process in which organic matter in sludge is broken down into gases (e.g., methane and carbon dioxide), reducing volatile suspended solids by 40 to 50 percent. Digested sludge is much less odorous and contains far fewer pathogens than raw, un-stabilized sludge. The digested sludge can then be dewatered to further reduce volume. Anaerobic digestion has many advantages, such as reducing solids volume and creating a sludge that can potentially be processed further to create fertilizer. Digestion also provides for the ability to produce energy and heat by using excess biogas in a cogeneration system. This energy production can off-set the energy needs at the facility.

Effective anaerobic digestion requires consistent tank mixing as well as heat to maintain the proper biological activity. Tanks are typically heated to 95°F and may require supplemental heat in the winter months if biogas production is insufficient to meet heat demands. The digesters are covered and mechanical systems installed to mix the contents and to allow for gas collection and reuse. These digesters also require a reasonably sized parcel to accommodate the digesters, associated pumping and equipment, gas cleaning and storage as well as the cogeneration system. The complex system requires significant capital expenditure in both the tankage and supporting processes.

Previously, the WPCA and the City of Bridgeport evaluated adding anaerobic digester facilities in the parcel to the North of the West Side WWTP. The two-digester facility was to be owned and operated by a third party that would also accept food waste from the community. The project had progressed to a point where a site investigation was undertaken on the proposed site. Significant PCB contamination ultimately put the project on hold permanently.

The potential for anaerobic digestion at the West Side WWTP was again considered as part of this Facilities Plan. Preliminary sizing indicated that the system would require two 2.2 MG digester tanks, each with a diameter of about 85-feet, and a support building for supporting systems. Given the extent of proposed improvements required under the Administrative Orders to reduce CSOs, and improve and upgrade the liquid treatment trains at both facilities to meet NPDES permit standards it was decided that anaerobic digestion is not the highest priority investment at this time. Subsequent to the start-up and operation of the plant improvements, a reassessment of biosolids quality and quantity, anaerobic digestion and/or another residuals stabilization process could be considered at the West Side WWTP or East Side WWTP to treat biosolids from both facilities. Again, a long-term third party DBO contract may be the best mechanism for implementation. Land availability becomes the most critical aspect to move forward. The plant upgrades will be designed to easily accommodate connections to accommodate a future facility.

6.11.5.3 Gasification

Gasification involves the conversion of carbon-based materials using incomplete thermal oxidation to create a synthetic gas (syngas) that can be combusted as an energy source. There are varying configurations, but the general process includes subjecting dried sludge to high heat (1,100 to 1,800°F) and controlled oxygen levels in a reactor. The reactor produces solids (as ash) and gas. Heat from the gas can be recovered in a heat exchanger and reintroduced into the reactor or used elsewhere. Gasification is a relatively new method of sludge disposal and has not yet been proven to be economically viable. However, given the proposed delay in implementing more sophisticated residuals management alternatives, this could be a viable alternative in the future.

6.11.5.4 Incineration

There are two types of sludge incinerators: multiple-hearth and fluidized bed incinerators. Both types subject sludge cake to high levels of heat, producing gas and ash. Ash is collected at the bottom of the incinerator and gas rises to the top and leaves through a gas flue. Gas is then sent through a scrubber and discharged via an exhaust stack. Ash is collected and requires further disposal. Heat can be recovered and reintroduced to the incinerator or used to generate power.

Incinerators carry high capital and O&M costs. Sludge from various sources is often needed to make operation of an incinerator financially feasible. Additionally, evolving EPA and DEEP regulations on incinerator emissions put the long-term use of incinerators at risk. The East Side WWTP formerly incinerated sludge, but that incinerator has since been abandoned. Because of the expected regulatory hurdles on-site incineration will not be considered further.

6.11.5.5 Thermal Drying

Thermal drying involves heating biosolids to evaporate water and reduce the moisture content beyond conventional mechanical dewatering methods, producing a product that is typically referred to as pellets or granules. The drying rate is highly dependent on solids flow, initial moisture content and evaporation rates. Typically, drying is achieved in three stages. During the warm-up stage, both the solids temperature and drying rate increase. The warm-up stage is typically short, resulting in little drying. The next stage, the constant-rate stage, is where the majority of the evaporation occurs. The falling-rate stage is the final stage of drying where the drying rate decreases and the sludge has passed the point of critical moisture.

Many different drying technologies exist. These technologies use either convection, conduction or a combination of the two to transfer heat to solids. Convection systems transfer heat from hot gas to the wet solids directly. Conduction systems transfer heat from one medium, such as steam, to the solids through another material that acts as a barrier between the solids and the hot medium. Solar drying has also been implemented successfully in many cases, but depending on location, likely require additional heat to increase drying performance.

Many thermal drying processes produce biosolids with sufficient pathogen reduction to meet Class A requirements for beneficial reuse, as fertilizer. Class A products are much more marketable than dewatered cake. The transportation of dried biosolids is highly efficient due to the significant reduction in mass and volume. Given the extent of proposed improvements required under the Administrative Orders to reduce CSOs, and improve and upgrade the liquid treatment trains at both facilities to meet NPDES permit standards it was decided that thermal frying is not the highest priority investment at this time. Subsequent to the start-up and operation of the plant improvements, a reassessment of biosolids quality and quantity, solids dewatering and thermal drying could be considered at the West Side WWTP or East Side WWTP to treat biosolids from both facilities. Again, a long-term third party DBO contract may be the best mechanism for implementation. Land availability becomes the most critical aspect to move forward. The plant upgrades will be designed to easily accommodate connections to accommodate a future facility.

6.11.5.6 Recommendations

Defer recommendations for stabilization technology at this time.

6.12 Odor Control

For most cases at a wastewater treatment plant, the basic odor control strategy consists of containment, ventilation, and treatment and/or dispersion of exhaust gases. To utilize vapor-phase treatment, determining the design airflow for each source is a critical step. This section discusses the methodology used to calculate air flow, however, preliminary calculations of air flow rates is presented in Sections 7 and 8 based on the selected unit processes. For this project CDM Smith has assumed the following areas may require odor control: preliminary treatment (screening and grit removal and influent pumping station wet well), primary treatment, and sludge handling.

6.12.1 Containment and Ventilation

Approaches to contain odor source emissions will vary by process type and size.

Figure 6.12-1
Augie Dumpster by Custom Conveyor Corp

Headworks improvement alternatives include new equipment likely located indoors or contained, such as the mechanical bar screens, washer compactors, grit removal systems, grit washers, screenings and grit dumpsters, and the pumping station wet well. To reduce the quantity of air flow that must be moved and treated, it is recommended that the equipment be provided with individual enclosures that are tight fitting and easy-to-open for operational accessibility. The manufacturers of state-of-the-art screen and grit processing equipment typically provide standard enclosures for their equipment. The load out containers will likely be housed within a garage to reduce both odors and visual impacts. If the dumpsters are located outside, covers are recommended to control odorous emissions. Leveling lid type covers such as an "Augie Dumpster" or "Level Lodor™" could be used (**Figure 6.12-1**), however these

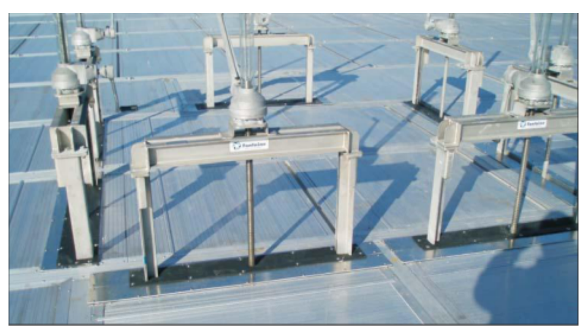


Figure 6.12-2
Aluminum Covers

systems are quite expensive and can be cumbersome to connect.

Screen channels and tanks would be fitted with flat cover plate to contain odors but provide access when needed. Aluminum covers are lightweight, durable, corrosion resistant, and constructed from rigid structured members as presented on **Figure 6.12-2**. Material of construction need to be cognizant of the coastal environment. Hatches can be designed for quick access for viewing the process below. For sources located inside a new building (depending on the alternative), additional containment could be used to minimize the amount of air conveyed to odor control with the remaining room air controlled by the HVAC system and vented to the atmosphere.

The requirements in the manual, NFPA 820, typically govern ventilation rates for odor control systems. For areas with exposed raw wastewater and for sludge holding tanks, the required NFPA 820 ventilation rates and their impact on electrical equipment classifications are generally as follows:

- 12 air changes/hour (ac/hr) provides a Class 1, Division 2 classification in the regulated/enclosed space
- < 12 ac/hr requires a Class 1, Division 1 classification in the regulated/enclosed space

Screen and grit handling and storage buildings, for example, are unclassified under NFPA with no ventilation required. However, operational comfort and corrosion potential should also be considered when designing air changes in such a building.

Another key criterion in governing ventilation rates is the need to maintain negative pressure (0.05 to 0.1-inch water column minimum). In locations where electrical components are not within or near the enclosed space, the ventilation rate can be reduced to provide only enough flow to maintain a negative pressure.

6.12.2 Candidate Odor Control Technologies

A variety of vapor-phase and liquid-phase control technologies are available to treat odors, encompassing physical, biological, and chemical treatment approaches.

The following odor reduction technologies were initially selected for investigation:

- Carbon Adsorption
- Biotrickling Filters (BTFs)
- Low Profile Biotrickling Filters (LP BTFs)
- Biofiltration
- Chemical Scrubbers
- Dispersion Fans

6.12.2.1 Carbon Adsorption

Activated carbon has been widely used to control H_2S and other odorous compounds emitted at wastewater treatment facilities. With this technology, the contaminated airstream passes through granulated carbon or other adsorptive media, providing a surface where chemical compounds in the airstream are adsorbed. Several types of adsorbents are available, including virgin carbon, catalytic carbon, and impregnated alumina.

Virgin carbon is used for the removal of VOCs. The carbon utilizes simple adsorption by trapping the contaminants to the media. As such, the media cannot be regenerated and is replaced when the media is spent. Virgin carbon has a very low capacity for H_2S and is generally used in the wastewater industry as a polishing stage in series with a treatment process that is more efficient with H_2S.

Catalytic carbon also has adsorptive properties that attract small molecules to the surface, however in contrast to virgin carbon, the adsorptive surface has catalytic effects that convert H_2S to elemental sulfur. This effect increases the overall capacity of catalytic media to two orders of magnitude greater than virgin carbon. Similar to virgin carbon catalytic carbon cannot be regenerated. Catalytic carbon is the industry standard for low (typically <10 ppm) levels of H_2S.

Impregnated adsorbents are used when H_2S is accompanied by relatively high levels of organic sulfur compounds, such as is found in solids processing odors. The adsorbent material is commonly activated alumina impregnated with permanganate for sulfur odors, however other adsorbents are available for the removal of ammonia, chlorine, and other compounds.

Figure 6.12-3
Carbon Adsorption System

Carbon adsorption systems consist of a tower filled with one or two beds containing the carbon media and a fan to push (or pull) odorous air through the media. These systems are common, although radial and horizontal flow configurations have become popular as well when sufficient space is available. Low-profile units with multiple stages have seen increased use, especially, as they allow the opportunity to "mix and match" adsorbent types for maximum odor reduction and cost-effectiveness. **Figure 6.12-3** illustrates a dual bed carbon system used in a municipal application. Regardless of their configuration, carbon systems are reliable and easy to maintain, as they have no moving parts other than fan components.

From a performance perspective, warrantees for 99.5 percent removal of H_2S and 95 percent of odor can be obtained in most cases. **Table 6.12-1** lists the advantages and disadvantages of these systems.

Table 6.12-1 Carbon Adsorption Advantages and Disadvantages

Advantages	Disadvantages
▪ Effective H_2S removal for relatively low inlet loadings ▪ Enhanced treatment for other odorants ▪ Low capital cost	▪ Not applicable for moderate to high strength exhausts due to media replacement costs

Advantages	Disadvantages
▪ Ease of operation and maintenance ▪ Small footprint ▪ Appropriate for intermittent operation	▪ Potentially higher life cycle costs, (relatively high energy costs for granulated carbons) ▪ Media replacement requirements ▪ Unpredictability of media life

Life-cycle costs are particularly sensitive to media replacement, and therefore average and peak H_2S concentrations need to be low for stand-alone carbon systems, otherwise the frequency of media replacement becomes of significant cost and very labor intensive.

6.12.2.2 Biotrickling Filters

Biological trickling filters, also known as bioscrubbers, rely on mass transfer and biological oxidation to remove odors. Odorous compounds dissolve and partition into a formed liquid biofilm on the bioscrubber media. The current standard for media is a plastic synthetic material that is inert and acts only to support the growth of microorganisms. This process allows for biological reactions to occur in the biofilm located on the media surface. An acclimation period of up to four weeks is required to develop the biofilm.

Biotrickling filters consist of cylindrical towers, as shown on **Figure 6.12-4**, filled with one or more layers of synthetic media, and irrigation control includes water and nutrient feed systems. Odorous air enters the tower from the base and flows upward, while water supplemented with nutrients trickles down across the inorganic media from the top of the tower. Depending on the manufacturer the water can be recirculated; non-potable water or potable water may be used.

The water acts as the mass transfer medium and also removes microbial metabolic byproducts such as sulfuric acid and elemental sulfur generated by the oxidation of H_2S. The production of sulfuric acid allows for removal of ammonia as a secondary pollutant by forming ammonium sulfate. The blowdown from the system typically has a low pH (about 2), and pH sampling can provide a good indication of system performance or whether to adjust irrigation controls. The current standard warrantee for synthetic media is 10 years, and in practice the media life often lasts even longer, with some warrantees extending to 15 years.

Figure 6.12-4
Biotrickling Filter

Overall, biotrickling manufacturers warrant H_2S reductions of 98 to 99 percent and odor reductions of 90 percent, with a corresponding maximum outlet concentration that can be as low as 600 dilutions to threshold (D/T).

Table 6.12-2 lists the advantages and disadvantages of biotrickling filters. Compared to other technologies, such as carbon adsorption and biofiltration (discussed subsequently), biotrickling filters can handle significantly higher H_2S concentrations.

Table 6.12-2 Biotrickling Filter Advantages and Disadvantages

Advantages	Disadvantages
Effective H_2S removal for wide range of concentrations Reacts well to brief load variations Ease of operation Small footprint Low pressure drop/energy costs	Difficulty in handling intermittent loading (i.e. intermittent dewatering schedules) Irrigation water required Nutrients may be required May require a polishing stage for odors associated with solids processing

6.12.2.3 Low Profile Biotrickling Filters

In lieu of vertical cylindrical towers, biotrickling filters can be oriented in a low-profile rectangular configuration, as shown in **Figure 6.12-5**. These systems operate in the same manner as the vertical towers, odorous air still enters from the bottom and flows upward, just over a larger surface area, and water still trickles down from the top.

Media types vary depending on the manufacturer. However, these systems lend themselves to staged media with standard biotrickling filter media in the lower layer and biofilter media, which is discussed in the next section, in the upper layer to treat other organic compounds. Others offer a carbon layer at the top of the vessel as a polishing step instead of biofilter media. Finally, these systems can be offered without any upper or polishing layers to decrease the required footprint. Consequently, loading rates and sizing vary.

Figure 6.12-5
Low Profile Biotrickling Filter (Courtesy of Bioair)

Refer to **Table 6.12-3** for advantages and disadvantages.

Table 6.12-3 Low Profile Biotrickling Filter Advantages and Disadvantages

Advantages	Disadvantages
Effective H_2S removal for wide range of concentrations Reacts well to brief load variations Ease of operation Low pressure drop/energy costs Low profile design, and with smaller footprint than a biofilter Allows staged media designs for polishing	Difficulty in handling intermittent loading (i.e. intermittent dewatering schedules) Irrigation water required Larger footprint than conventional Biotrickling filter Nutrients may be required

6.12.2.4 Biofiltration

Similar to biotrickling filters, biofilters also rely on mass transfer and biological oxidation, to remove odors. Odorous compounds dissolve into a biofilm where biological oxidation processes occur. Older designs utilized organic media consisting of composted green material, bark,

and woodchips. Modern biofilter media varies, with some manufacturers using synthetic media, lava rock, or similar inorganic high porosity material to support microbial populations. In contrast to plastic biotrickling filter media, organic biofilter media often contains nutrients and a wide variety of indigenous bacteria.

Biofilters can be constructed as in-ground concrete tanks or basins or manufactured as compact package units in fiberglass shells**, Figure 6.12-6**. Biofilters have an integrated humidification stage to saturate the incoming air stream with surface irrigation and to maintain moisture. Overall, the precise configuration of each biofiltration system varies, with key differences such as flow direction (upflow or downflow) and media type. Synthetic media and organic media have warranted lives of ten and five years respectively. Overall biofilter manufacturers warrant H_2S reductions of 98-99 percent and odor reductions of 90 percent.

Figure 6.12-6
Modular Biofilter

A key difference between biotrickling filters and biofilters is footprint. Biotrickling filters are loaded at a high rate of 50 to 120 cubic feet per minute per square foot (cfm/sf), while biofilters are typically operated at higher residence times and loaded at 3 to 5 cfm/sf. Due to the type of media used and higher residence time provided, biofilters are particularly suitable for treating other reduced sulfur compounds and complex airstreams associated with sludge processing. At the same time, biofilters cannot handle as high of H_2S concentrations as biotrickling filters. On the flip side if the H_2S concentrations are too low, the biotrickling filter may be starved and not perform well.

Table 6.12-4 lists the advantages and disadvantages for biofilters.

Table 6.12-4 Biofilter Advantages and Disadvantages

Advantages	Disadvantages
Effective H_2S removal for low to moderate concentrations Enhanced treatment of other odorants such as reduced sulfur compounds found in solids processing. Ease of operation Can be applied to certain intermittent operations	Not as effective for high H_2S concentrations (>50 ppm) Large footprint Relatively high pressure drop for organic media Organic media has short and unpredictable media life

6.12.2.5 Chemical Scrubbers

Chemical scrubbing is also used extensively for odor control at wastewater treatment plants. **Figure 6.12-7** shows a typical two stage low profile system with a chemical storage facility. This technology relies on the absorption of odorous gases into a liquid medium, accelerated through the use of added chemicals. Like biological scrubbers, odorous air is routed upward from the bottom of a vertical vessel, and modern designs use packing material in the vessel to

Figure 6.12-7
Chemical Scrubber

enhance mass transfer. The air contacts the chemicals and liquid solution that flows downward across the media surface. Treated or "scrubbed" air leaves at the top of the tower, while the liquid medium is collected in a sump at the bottom. A small percentage of the fluid is drained or "blown down" to the drain.

Typically, the system employs two chemicals for the most effective odor reduction. Single stage scrubbers use both sodium hydroxide and sodium hypochlorite in a single scrubber stage. Dual stage scrubbers use sodium hydroxide in the first stage and sodium hydroxide and sodium hypochlorite in the second stage. Sodium hydroxide increases the liquid pH to facilitate the uptake of H_2S into the liquid and is the most efficient way to remove H_2S. Sodium hypochlorite is an indiscriminate oxidizer, and while it can remove sulfide, the process can be inefficient; therefore, the second stage of a dual stage scrubber is a polishing stage to remove residual hydrogen sulfide and other odorous chemicals. Two-stage (series) scrubbers are recommended for sources with high H_2S concentrations greater than 50 ppm.

From a performance perspective, chemical scrubbers can provide predictable treatment for a wide variety of complex and intermittent odors, and as a result are regularly applied to solids handling applications with mixtures of ammonia, H_2S and organics sulfur compounds. Manufacturers can provide warrantees of 99.5 percent removal of H_2S and 95 percent of odor in most cases, as well as a warranted maximum outlet odor of 100 D/T. Notwithstanding, the design of the scrubber system or the loading rate, operations and maintenance is specialized and extensive to maintain predictable results. The operations and maintenance requirements can be mitigated somewhat by contracting the maintenance out, using a service contract to maintain the equipment; however, maintenance contracts come at a cost as well.

Refer to **Table 6.12-5** for the advantages and disadvantages for chemical scrubbers.

Table 6.12-5 Chemical Scrubber Advantages and Disadvantages

Advantages	Disadvantages
Effective and economical H_2S removal is limited to low H_2S levels (<25 ppm)	Potentially higher life cycle costs due to chemical costs at high H_2S levels
Can handle complex mixtures of chemicals	Chemical storage, delivery, and handling required
Reacts well to load variations	More complex than other systems
Small footprint	Blowdown handling required
Low capital costs	Specialized maintenance requirements (e.g. acid washing)

6.12.2.6 Dispersion Fans

The final technology discussed here is the enhanced dispersion fan, also known as a mixed flow induction fan. Enhanced dispersion fans do not provide treatment per se; the fan units "pull in" up to 70 percent of the exhaust volume, diluting the odorous air stream before discharge. They also accelerate the exhaust to provide enhanced dispersion creating an effective stack height beyond the height of the fan itself. Unlike a tall stack, however, enhanced dispersion fans offer a low profile. In addition, they can be equipped with silencers, although experience has shown them to offer quiet operation even without silencers.

Figure 6.12-8 depicts an installed dispersion fan system. The use of these units is increasing due to their low cost, effectiveness, and ease of operation. These beltless fans are very easy to maintain, requiring only lubrication once every 18 months to three years.

Dispersion fans are typically applied:

Figure 6.12-8
Enhanced Dispersion Fan

- In place of treatment technologies for low strength and high-volume exhaust
- As a supplement to treatment technologies, to both dilute outlet odor concentrations and improve dispersion characteristics of treated exhausts in lieu of a tall exhaust stack
- As redundancy for a treatment technology that is offline for maintenance or other reasons.

Advantages and Disadvantages are listed in **Table 6.12-6**. This technology could be appropriate for managing odorous air off of the primary treatment system.

Table 6.12-6 Dispersion Fan Advantages and Disadvantages

Advantages	Disadvantages
Effective dilution/dispersion of exhaust Low capital cost compared to treatment Ease of operation and maintenance Small footprint	Does not remove odors through treatment (dilutes and disperses odors) Effective for low strength exhausts May require silencer for noise control

6.12.3 Screening of Odor Control Technologies

Three significant screening criteria will drive the ultimate odor control technology selected for use at the West Side and East Side WWTPs: (1) the anticipated concentrations of H_2S and other odorous compounds to be treated, (2) the limited space available, (3) the desire to minimize chemical use on-site. To date, no odor monitoring has been performed to determine expected concentrations of odorous air in the designated areas. It is expected that this testing will be performed as a part of preliminary design.

At this point it is anticipated H_2S concentrations in the headworks will be in the low to medium range, given the northern climate and the combined collection system. It is also presumed that much of the equipment, channels and tanks will be enclosed or covered to reduce air flow. The options for primary treatment, either high rate clarification or primary filtration will likely warrant odor control, however, limited data is available on operational systems to assess expected H_2S concentrations. This will be further evaluated in preliminary design. With respect to solids processing, it is expected that the gravity thickeners, sludge holding tanks, thickened sludge holding tanks will all be covered and odorous air conveyed to a treatment system. It is also expected that the thickening equipment will be self-contained reducing the air flow requiring treatment.

Based on the available information, two odor control treatment alternatives will be carried forward: Biofiltration and Chemical Scrubbers. The ultimate site layout will determine if the odorous air emissions can be treated in a single unit, two units or three units (one for each area defined).

6.13 References

Doneker, R.L. and Jirka, G.H. 2007. *CORMIX User Manual: A Hydrodynamic Mixing Zone Model and Decision Support System for Pollutant Discharges into Surface Waters*. Prepared for U.S. Environmental Protection Agency.

Lawler, Matusky & Skelly Engineers L.L.P (LMS). 2002*. Effluent Dilution Study of Bridgeport's East Side and West Side Wastewater Treatment Plant Discharges: Subtask 5.3 Model Effluent Dilution of Existing Outfalls and Alternative Outfalls/Diffusers*. 2 Volumes. Prepared for Kasper Group, Inc., Bridgeport, CT.