# City of New Haven Combined Sewer Overflow Long-Term Control Plan Update

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Final Draft

Greater New Haven Water Pollution Control Authority
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# Executive Summary

## Introduction

This document is the 2022 update to the City of New Haven (City) Combined Sewer Overflow (CSO) Long-Term Control Plan (LTCP). Pursuant to Consent Order WC5509, an update to the CSO LTCP is required every 5 years. The Greater New Haven Water Pollution Control Authority (GNHWPCA or Authority) is being assisted by Jacobs Engineers in this effort.

The CSO LTCP Update is a planning document used by GNHWPCA to facilitate meeting the requirements of a Consent Order entered into by Connecticut Department of Energy and Environmental Protection (CT DEEP) and the Authority. Under the terms of the Consent Order, the Authority will invest in the infrastructure necessary to comply with the U.S. Environmental Protection Agency's (EPA) CSO Control Policy. This CSO LTCP Update summarizes progress made by the Authority to implement the LTCP and the status of planned improvements still to be constructed. Periodic updates are a useful tool for modifying the philosophy and strategies of the LTCP as new information and experience is obtained. CT DEEP approval of the LTCP updates is an acceptance of these changes.

The focus of this 2022 CSO LTCP Update is on the long-term wet weather improvements to the East Shore Water Pollution Abatement Facility (ESWPAF) needed for the facility to meet discharge permit requirements over the next 20 years, and maintain the ability to remove nitrogen to the maximum extent possible while experiencing increases in flows and loads. As discussed herein, projected peak flows to the ESWPAF are expected to increase to 187 mgd because of improvements to the collection system to reduce and eliminate CSO discharge and population growth in the service area.

## Background – 2018 CSO LTCP Update

The previous CSO LTCP Update was approved by the CT DEEP in 2018. The purpose of the 2018 CSO LTCP Update (CH2M 2018) was to incorporate the findings of the 2015 Hydraulic Model Update report (CH2M 2015) and investigate potential alternatives to meet an updated 2-year, 6-hour level of service, based upon continuing flow monitoring of CSOs in the system. The updated design storm is based upon Intensity Duration Frequency (IDF) curves generated by Cornell University and the Northeast Regional Climate Center (NRCC).

The 2015 Hydraulic Model Update report documents updates the Authority's collection system hydraulic model, including the upgrade to the latest modeling platform, the expansion of the model domain, and updates to the modeling components. The completion of a comprehensive flow monitoring program enabled the calibration, validation, and system optimization of the hydraulic model.

The conclusions of the 2018 CSO LTCP Update (CH2M 2018) consisted of recommendations in three major areas:

- **Short-term Improvements –** These improvements consist of system upgrades to remove or control CSO discharge from selected CSO outfalls. These controls were developed in the 2015 Hydraulic Model Update (CH2M 2015) and generally use available in-system storage and small-scale gray and green projects to make improvements to the system at a relatively low cost. Short-term improvements were implemented between 2016 and 2020.
- **Intermediate-term Improvements –** These improvements consist of the Yale Campus/Trumbull Street Phase 2A Sewer Separation project and improvements to the remote pump stations that pump flow to the ESWPAF. The purpose of the pump station improvements is to pump the maximum wet weather flow to the plant for treatment. The improvements consist of upgrades to the East Street Pump Station, Union Pump Station, and Boulevard Pump Station. Intermediate improvement projects were initiated in and are ongoing.

- **Long-term Improvements –** The focus of the long-term improvement projects is to provide upgrades to the ESWPAF to treat a peak flow of 187 million gallons per day (mgd), complete the Fair Haven Sewer Separation Project, and incorporate necessary green and grey infrastructure to eliminate CSOs during the design storm level of service.

## CSO LTCP Improvements 2017 through 2022

The GNHWPCA continues to work on several fronts to further the CSO control program. Short-term, intermediate-term, and long-term improvements have been identified and implemented as part of the program. In the years between 2017 and 2022, the GNHWPCA has undertaken the following projects:

- 2012-04 Design and Bidding of Yale Campus Trumbull Street Area Sewer Separation Phase 2 (A&B)
- 2016-01 Infiltration and Inflow Study Phase 1 – Mill River Trunk Sewer
- 2016-02 Regulator 012 and 020 Hydraulic Improvements – Closure of Regulators 012 and 020
- 2016-03 West River CSO Improvements - New Regulators 003, 004, and 006 and Weir Adjustments
- 2016-04 Infiltration and Inflow Study, SSES, Woodbridge and East Haven Areas 15, 18, & 23
- 2016-05 Regulator 025 and 034 Improvements – Closure of Regulator 034 and Weir Adjustment at 025
- 2016-06 Design and Construction I/I Removal Middle Thorpe Drive, Hamden and East Haven 5, 9 & 13
- 2016-07 Green Infrastructure Improvements in the West River Sewershed – Installation of 66 Bioswales
- 2017-01 Design of Capacity Improvements at East Street Pump Station for CSO Reduction
- 2017-02 CEPA Environmental Impact Evaluation Study for City of New Haven CSO LTCP Update 2018
- 2018-01 Infiltration and Inflow Study, SSES, Mill River Areas 7, 11, 15 & 22
- 2019-01 Infiltration and Inflow Study, SSES, Mill River Areas 6, 13, & 19
- 2019-03 Value Engineering for the Design of Capacity Improvements at East Street Pump Station
- 2019-04 Process Air Compressor Improvements at ESWPAF for Low Level Nitrogen Reduction
- 2019-05 Planning Study, Design and Bidding of Orchard Street Area Sewer Separation
- 2021-03 City of New Haven CSO LTCP Update 2022
- 2022-01 Fair Haven Regulator Improvements Phase 1 – Adjust Regulators 009, 015, & 016

## Benefits of CSO Improvements 2017 to 2022

The completion of the Short-term Control Plan projects identified in the 2018 CSO LTCP Update was projected to reduce CSOs during a typical year from 30 million gallons per year in 2016 conditions to 19 million gallons in the post-Short-term Control Plan conditions.

Specific improvements included in the Intermediate-term Control Plan and scheduled for implementation between 2022 and 2028 include the following:

- Capacity Upgrade to the East Street Pump Station
- Yale Campus/Trumbull Street Phase 2 Sewer Separation
- Orchard Street Sewer Separation
- Phase II ESWPAF Wet Weather Treatment Improvements
- Fair Haven Regulator Improvements – Phase 2
- 2027 Update to the LTCP

**Table ES-1** lists significant physical improvements made by the GNHWPCA to the Combined Sewer System (CSS) between 2017 and 2022. These improvements typically include raising weirs to reduce the number of potential overflows per year and upgrading pump stations and piping to permit the closing of regulators and the elimination of overflows.

**Table ES-1. Improvements and Closures of CSO Regulators (2017 – 2022)**

| No. | Discharge Location | Improvement |
|---|---|---|
| ***Improvements*** | | |
| #003 | West River | Weir raised 16 inches in 2020 |
| #004 | West River | Weir raised 24 inches in 2020 |
| #006 | West River | Weir raised 42 inches in 2020 |
| #009 | Mill River | Weir raised 8 inches in 2015 and by 6 inches in 2022 |
| #015 | Quinnipiac River | Weir raised 18 inches in 2022 |
| #016 | Quinnipiac River | Weir repaired in 2014 and raised 6 inches in 2022 |
| #024 | New Haven Harbor | Weir raised 1.5 feet in 2017 |
| #025 | New Haven Harbor | Weir raised 9.15 feet in 2019 |
| ***Closures*** | | |
| #010 (A) | Mill River | Regulator closed in 2020 |
| #012 | Mill River | Closed in 2018 |
| #020 | Quinnipiac River | Closed in 2019 |
| #026 | Mill River | Closed in 2019 |
| #028 | Mill River | Closed in 2018 |
| #034 | New Haven Harbor | Closed in 2019 |

## Basis of Design for Wet Weather Upgrades to the ESWPAF

Upgrades to the ESWPAF in New Haven to accommodate CSO flows are being undertaken in at least three phases of work. Phase 1 of the planned upgrade work has been completed, and included the following components:

- Electrical Upgrades
- Odor Control Upgrades
- Nitrogen Removal Improvements (Carbon Addition, 2nd Anoxic Zone)
- Gravity Thickener and Sludge Storage
- Process Air Compressor Upgrade

The implementation plan for the wet weather upgrades to the ESWPAF assumes that two future phases of work that (that is, Phase 2 and Phase 3) will be undertaken to address increases in flows and loads and wet weather impacts on the plant.

Hydraulic flows for the current plant and the Phase 2 and Phase 3 upgrades are based on the flow conditions presented in **Table ES-2.**

**Table ES-2. Current, Phase 2, and Phase 3 Flows**

| Flow Stream | 2022 | Phase 2 | Phase 3 |
|---|---|---|---|
| ***Plant Flows (mgd)*** | | | |
| Total Plant Flow | 100 | 147 | 187 |
| Flow to Secondary | 60 | 60 | 60 |
| Wet Weather Flow | 40 | 87 | 127 |
| ***Internal Recycle Flows (mgd)*** | | | |
| RAS Recycle | 40 | 40 | 40 |
| NRCY Recycle | 80 | 80 | 80 |

NRCY = nitrified mixed liquor recycle
RAS = return activated sludge

The peak hydraulic flow for the ESWPAF was established through an evaluation described in the report *Wet Weather Capacity Improvements and Nitrogen Reduction at the East Shore Water Pollution Abatement Facility* (CH2M 2011a). The maximum conveyance to the ESWPAF is 187 mgd during a 2-year, 6-hour duration design storm event. The Phase 2 flow of 147 mgd is based on the maximum capacity of the existing primary clarifiers with all units in service. The maximum flow to secondary treatment is 60-mgd for both Phase 2 and Phase 3.

The resulting wet weather flows of 87 mgd for Phase 2 and 127 mgd for Phase 3 were determined by subtracting the flow to secondary treatment from the total plant flow. The RAS and NRCY recycles are the maximum recycle flows the plant is designed to accommodate.

## Phase 2 Wet Weather Improvements

The main focus of the Phase 2 upgrades is to construct a wet weather treatment facility downstream of the primary clarifiers at the ESWPAF. This project will include a facility to split primary effluent flow between secondary treatment and wet weather disinfection, two wet-weather chlorine contact tanks, and two new effluent lines to convey disinfected effluent to the plant outfall.

The first flow path (existing secondary treatment and disinfection), consists of four aeration basins, eight secondary clarifiers, and two chlorine contact tanks. Disinfected secondary effluent then discharges from the plant through twin 48-inch outfall pipes and a 90-inch square outfall. A maximum flow of 60 mgd is anticipated for the secondary treatment and disinfection train.

The second flow path (the wet weather treatment facility) consists of two new wet weather disinfection tanks and two 72-inch discharge pipes. The two 72-inch pipes ultimately intersect with disinfected flow from the first flow path at the 90-inch square outfall. All flow greater than 60 mgd will be conveyed to the wet weather disinfection facility.

## Phase 3 Wet Weather Improvements

The Phase 3 upgrades include a new preliminary treatment facility, a new flow-splitting facility upstream of the primary clarifiers, a fourth primary clarifier, and upgrades to the aeration basins. A hydraulic evaluation was performed to determine whether a second outfall would be needed for the Phase 3 flow condition, which has a peak flow of 40 mgd greater than that of Phase 2. Consequently, two hydraulic profiles were developed for Phase 3 for the purpose of determining whether a new outfall would be needed or beneficial for the ESWPAF, one without a new outfall and a second with the new outfall.

## Hydraulic Evaluation

An extensive hydraulic evaluation was conducted in support of this CSO LTCP Update. The Phase 2 and Phase 3 flows presented in **Table ES-2** were used as the basis for the evaluation. CSO closures, an ongoing program to make modifications to CSO regulators, and improvements to the sewer system will result in a dramatic increase in flow that will need to be treated at the ESWPAF, Consequently, the objectives of the hydraulics evaluation were to determine how these higher flows would impact the plant and to evaluate whether a new outfall would be needed. Conclusions and recommendations from the hydraulic evaluation are as follows:

- Design storm – Per TR-16 requirements, existing facilities should have uninterrupted operation of all units during 25-year flood conditions, and should be protected against the structural, process, and electrical equipment damage that might occur in a 100-year flood. A harbor elevation of 8.3 feet (NAVD 88) was selected to be representative of the 25-year design storm. The harbor elevation at the 100-year design storm is 12.0 feet (NAVD 88).

- Phase 2 flows – A new wet weather treatment facility will be constructed during Phase 2 to accommodate influent flows of up to 147 mgd at a harbor elevation of 8.3 feet (NAVD 88) during the 25-year design storm. The 25-year design storm does not negatively impact the hydraulic capacity of the existing primary and secondary treatment systems, disinfection, or the existing outfall.
- Phase 3 flows without a new outfall:
  - The hydraulic profile for Phase 3 flows of up to 187 mgd at a harbor elevation of 8.3 feet (NAVD 88) does not impact existing secondary treatment, disinfection, or existing outfall capacity during the 25-year design storm.
  - Primary clarifier weirs and scum collection equipment will need to be raised to accommodate these flow conditions.
- Limiting processes at harbor elevations between the 25-year design storm [8.3 feet (NAVD 88)] and the 100-year design storm [12.0 feet (NAVD 88)]:
  - The harbor elevation can rise to a level of 10.0 feet (NAVD 88) at Phase 3 flows without negatively impacting existing secondary treatment, disinfection, or outfall capacity.
  - The new wet weather facility can treat influent flows up to 187 mgd at a harbor elevation of 11.5 feet (NAVD 88) without impact to the wet weather disinfection facility, primary treatment, or preliminary treatment.
- Outfall recommendation:
  - The hydraulic evaluation indicates that adding a new outfall (that is, two additional outfall pipes) will not significantly improve the plant's ability to mitigate the impacts of higher harbor elevations.
  - An additional outfall is not recommended because the benefits are relatively modest compared to the potential cost and effort of constructing the outfall.

## Recommended Upgrades to the ESWPAF

**Phase 2** of the proposed improvements projects will provide a new treatment train (that is, a Wet Weather Treatment System) to allow wet weather flows of up to 147 mgd to be processed. Flow through the biological treatment system will be limited to 60 mgd. Peak flows greater than 60 mgd will be separated after primary treatment and be disinfected before being discharged through the plant outfall. Phase 2 improvements are anticipated to include the following:

1. A Wet Weather Disinfection Facility, consisting of a dedicated chlorine contact tank to disinfect wet weather flows over 60 mgd
2. Piping modifications to convey primary effluent (PE) to the facility.
3. A Flow-Splitting Facility for splitting primary effluent flows between the biological treatment system and the wet weather disinfection system.
4. Discharge piping to convey disinfected wet weather flows to the plant outfall.
5. An additional odor control train.

**Phase 3** improvements will allow the ESWPAF to process wet weather flows of up to 187 mgd. Potential projects include the following:

1. A new preliminary treatment building
   a. Influent flow metering
   b. Screening
   c. Grit removal

2. Primary treatment upgrades
    a. A new pipe gallery, or an extension to the existing pipe gallery
    d. Primary clarifier influent flow splitter
    e. Primary sludge pumps
    b. A fourth primary clarifier
    c. Rehabilitation of the three existing primary clarifiers
    d. Chemically-enhanced primary treatment
        i. Pilot testing of CEPT between Phase 2 and 3 is recommended
3. Secondary treatment improvements to incrementally increase treatment capacity
    a. Settleability improvements
        i. Hydrocylones
    b. Improvements to increase capacity
        i. MOB
        ii. MABR
    c. MOB and MABR were both evaluated for this LTCP update. A recommendation is not made at this time because both processes have relatively unproven track records.
4. Disinfection and outfall improvements
    a. Raise the baffle walls of the existing chlorine contact basin to address hydraulic restrictions at the 100-year flood. This improvement is not required for the 25-year design flood.
    b. The existing outfall has been determined to be in relatively good condition. However, the Authority should plan for a future joint rehabilitation project.
5. Solids treatment and processing
    a. A third gravity thickener

## Cost Estimates

**Table ES-3** summarizes the construction cost estimates and approximate schedules for the Phase 2 and Phase 3 long-term improvement projects discussed in this report. The estimated construction costs are itemized according to the major unit process groupings described above. The Phase 2 project (Wet Weather Treatment System) is of high priority, is the most completely defined at this time, and will most likely be the next major project to be implemented at the ESWPAF

The detailed implementation schedule for the Phase 3 projects is unknown at this time. The currently-envisioned project groupings may be further prioritized and subdivided into smaller projects as the Authority's future needs evolve.

**Table ES-4** summarizes the estimated cost for all of the major components of the LTCP, as identified in this CSO LTCP Update.

**Table ES-3. Construction Cost Summary for Wet Weather Improvements to the ESWPAF**

| CSO LTCP Components for the ESWPAF | Million (2022) | Grant % | CWF Grant | NH Loan Share | GNH Loan Share | 2022 | 2023 | 2024 | 2025 | 2026 | 2027 | 2028 | 2029 - 2040 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Phase II - Wet Weather Treatment System and Odor Control | $65.0 | 50 | 32.5 | 13.0 | 19.5 | | | | | | | | |
| Phase III - Preliminary Treatment Improvements | $69.2 | 40 | 27.7 | 16.6 | 24.9 | | | | | | | | |
| Phase III - Primary Treatment Improvements (w/ CEPT) | $58.5 | 40 | 23.4 | 14.1 | 21.1 | | | | | | | | |
| Phase III - Biological Treatment Improvements (hydrocyclones) | $4.3 | 40 | 1.7 | 1.0 | 1.5 | | | | | | | | |
| Phase III - Biological Treatment Improvements (capacity) | $50.3 | 40 | 20.3 | 12.2 | 18.3 | | | | | | | | |
| Phase III - 4th Gravity Thickener | $6.0 | 40 | 2.4 | 1.4 | 2.1 | | | | | | | | |
| Phase III – Disinfection and Outfall Improvements (Allowance) | $3.2 | 40 | 1.3 | 0.8 | 1.1 | | | | | | | | |
| Subtotal (ESWPAF Improvements) | $256.5 | | 109.3 | 59.1 | 88.6 | | | | | | | | |

## Table ES-4. CSO Long-Term Control Plan Implementation Schedule and Project Cost Estimates

| 2022 Long Term Control Plan Update | Million (2022 $) | Grant % | CWF Grant | NH Loan Share | GNH Loan Share | 2022 | 2023 | 2024 | 2025 | 2026 | 2027 | 2028 | 2029-2040 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Intermediate Term Improvements*** | ***$ 173.9*** | | ***$ 83.2*** | ***$ 31.8*** | ***$ 58.9*** | | | | | | | | |
| 2022 Long Term Control Plan Update | 0.5 | 55% | 0.3 | 0.0 | 0.2 | | | | | | | | |
| Yale Campus/Trumbull Street Phase 2A Separation (CWF 2012-04) | 20.0 | 50% | 10.0 | 4.0 | 6.0 | | | | | | | | |
| Orchard Street Sewer Separation (CWF 2019-05) | 17.0 | 50% | 8.5 | 3.4 | 5.1 | | | | | | | | |
| Capacity Upgrade of East Street Pump Station (CWF 2017-01 + VE) | 53.3 | 50% | 26.7 | 10.7 | 16.0 | | | | | | | | |
| Process Air Compressor Improvements for Low Level Nitrogen Control | 12.9 | 20% | 2.6 | 0.0 | 10.3 | | | | | | | | |
| Phase II - Wet Weather Treatment System & Odor Control (CWF 2024-01) | 65.0 | 50% | 32.5 | 13.0 | 19.5 | | | | | | | | |
| Fair Haven CSO Improvements (CWF 2023-02) | 3.5 | 50% | 1.8 | 0.7 | 1.1 | | | | | | | | |
| Wet Weather Flow Conveyance Study from West Side (CWF 2024-02) | 0.5 | 55% | 0.3 | 0.0 | 0.2 | | | | | | | | |
| 2027 Long Term Control Plan Update and Model Update (CWF-2025-01) | 1.2 | 55% | 0.7 | 0.0 | 0.5 | | | | | | | | |
| ***Long Term Improvements (ESWPAF)*** | ***$ 518.4*** | | ***$ 236.0*** | ***$ 112.5*** | ***$ 169.8*** | | | | | | | | |
| Fair Haven CSO Improvements - Phase 3 (CWF 2028-01) | 20.0 | 50% | 10.0 | 4.0 | 6.0 | | | | | | | | |
| Wet Weather Conveyanace Improvemens from West Side to Harbor | 25.5 | 50% | 12.8 | 5.1 | 7.7 | | | | | | | | |
| Capacity Upgrade of Boulevard Pump Station | 45.9 | 50% | 23.0 | 9.2 | 13.8 | | | | | | | | |
| Capacity Upgrade of Union Pump Station, Force Main, Bridge over RR | 25.2 | 50% | 12.6 | 5.0 | 7.6 | | | | | | | | |
| 2032 Long Term Control Plan Update | 0.8 | 55% | 0.4 | 0.0 | 0.4 | | | | | | | | |
| Phase III Wet Weather Improvements at the ESWPAF: (Preliminary Treatment, Primary Treatment, Gravity Thickening, Disinfection, Outfall Improvements, Biological Treatment Improvements) | 233.0 | 40% | 93.2 | 55.9 | 83.9 | | | | | | | | |
| 2037 Long Term Control Plan Update and Model Update | 1.5 | 55% | 0.8 | 0.0 | 0.7 | | | | | | | | |
| Fair Haven CSO Improvements - Phase 4 | 92.8 | 50% | 46.4 | 18.6 | 27.8 | | | | | | | | |
| CSO Storage Tanks/Separation/Green Infrasturcture | 73.7 | 50% | 36.9 | 14.7 | 22.1 | | | | | | | | |
| ***Estimated Total*** | ***$ 692.3*** | | ***$ 319.2*** | ***$ 144.3*** | ***$ 228.8*** | | | | | | | | |

# Acronyms and Abbreviations

| | |
|---|---|
| °C | degrees Celsius |
| AA | Annual Average |
| ACH | air changes per hour |
| AFD | adjustable frequency drive |
| ASI | ASI Group Ltd. |
| Authority | Greater New Haven Water Pollution Control Authority (GNHWPCA) |
| BOD | biochemical oxygen demand |
| BOD-5 | 5-day biochemical oxygen demand |
| CEPT | chemically enhanced primary treatment |
| City | City of New Haven |
| CSO | combined sewer overflow |
| CSO LTCP Update | Combined Sewer Overflow Long Term Control Plan Update |
| CSS | combined sewer system |
| CT DEEP | Connecticut Department of Energy and Environmental Protection |
| cy | cubic yard(s) |
| cy/hr | cubic yards per hour |
| DO | dissolved oxygen |
| EPA | U.S. Environmental Protection Agency |
| ESWPAF | East Shore Water Pollution Abatement Facility |
| FEMA | Federal Emergency Management Agency |
| FOG | fat, oil, and grease |
| $ft^2$ | square foot (feet) |
| gal | gallon(s) |
| gal/d/capita | gallons per day per capita |
| GBT | gravity belt thickener |
| GNHWPCA | Greater New Haven Water Pollution Control Authority (Authority) |
| gpd | gallon(s) per day |
| gpd/$ft^2$ | gallons per day per square feet |
| gph | gallon(s) per hour |
| gpm | gallon(s) per minute |

| | |
|---|---|
| HDPE | high density polyethylene |
| I/I | Inflow and Infiltration |
| IFAS | integrated fixed-film activated sludge |
| IQR | interquartile range |
| lbs | pounds |
| lbs/ft$^2$/day | pound per square foot per day |
| lbs N/d | pounds nitrogen per day |
| LTCP | Long-Term Control Plan |
| m$^2$ | square meters |
| m$^3$ | cubic meters |
| MABR | Membrane Aerated Biofilm Reactor |
| Magmeter | electromagnetic flow meter |
| MD | maximum day |
| MG | million gallon(s) |
| mg/L | milligram(s) per liter |
| mg-N/L | milligrams nitrogen per liter |
| mgd | million gallon(s) per day |
| mL/g | milliliter(s) per gram |
| MLE | Modified Ludzack-Ettinger |
| MLSS | mixed liquor suspended solids |
| mm | millimeter(s) |
| MM | maximum month |
| MOB | mobile organic biofilm |
| MW | maximum week |
| N/A | not applicable |
| NAVD 88 | North American Vertical Datum from 1988 |
| NFPA | National Fire Protection Association |
| $NH_3$-N | ammonia |
| $NH_4$-N | ammonium |
| NMC | Nine Minimum Control |
| NPDES | National Pollutant Discharge Elimination System |

| | |
|---|---|
| NRCY | nitrified mixed liquor recycle |
| ORTHO-P | orthophosphate |
| PF | peaking factor(s) |
| PE | Primary Effluent |
| PFD | process flow diagram |
| PH | Peak Hourly |
| PPD | pounds per day |
| RAS | return activated sludge |
| ROV | remotely operated vehicle |
| scfm | standard cubic feet per minute |
| SOTE | standard oxygen transfer efficiency |
| SPA | state-point analysis |
| SRT | solids retention time |
| SVI | sludge volume index |
| SWD | side water depth |
| TKN | total Kjeldahl nitrogen |
| TN | total nitrogen |
| TP | total phosphorus |
| TSS | total suspended solids |
| USACE | U.S. Army Corps of Engineers |
| UV | ultraviolet |
| WAS | waste-activated sludge |
| WLA | waste load allocation |
| WWTF | wastewater treatment facility |

# 1. Introduction

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## 1.1 Long-Term Control Plan Background

In 1997, the City commissioned the City of New Haven Long-Term CSO Control Plan (CH2M 1997) to comply with EPA's CSO Control Policy of April 1994 guidance, and as described in the New Haven Water Pollution Control Authority's National Pollutant Discharge Elimination System (NPDES) permit.

In April 2001, the City of New Haven Long-Term CSO Control Plan was developed in collaboration with the Connecticut Department of Environmental Protection (the predecessor agency to CT DEEP) and a broad group of stakeholders. The report was approved in March of 2003 and a program to eliminate all CSOs during a 2-year, 6-hour storm event was adopted.

In 2005, the GNHWPCA was created to acquire the sewer systems and provide a regional approach to sewer service for approximately 200,000 customers in the City of New Haven, and the Towns of Hamden, East Haven, and Woodbridge. Of the 555 miles of sewers in the system, the system in the City consists of approximately 70 miles of combined sewers, 70 miles of separated sewers, and 155 miles of sanitary sewers. There are approximately 260 miles of sanitary sewers in Hamden, East Haven, and Woodbridge. The Authority entered into a cost-sharing agreement with the City whereby the costs for CSO-related work would be shared 60/40, with 40 percent being the City's share.

In May 2008, a series of reports prepared for GNHWPCA documented the progress made in reducing CSOs for the 10-year period between 1997 and 2007. In July 2009, the Authority entered into a Consent Order (WC5509) with the CT DEEP for execution of the LTCP that would eliminate all CSOs during the original 2-year, 6-hour storm event.

In November 2009, the CSO LTCP Update was prepared and documented the required infrastructure improvements necessary to receive and treat the additional CSOs at the East Shore Water Pollution Abatement Facility (ESWPAF). The report also consolidated, in the form of appendices, the 2001 LTCP report and the many subsequent reports supporting the CSO LTCP Update. In March 2011, an LTCP Update was approved by CT DEEP (CH2M 2011b).

In July 2015, CT DEEP and the GNHWPCA negotiated a modification to Consent Order WC5509 to memorialize the schedule for the Authority to complete its CSO LTCP Update and subsequent updates, and require the construction of the improvements identified in such updates to achieve CSO control. The Hydraulic Model was also updated and approved by CT DEEP in 2015. The findings concluded in the 2015 Hydraulic Model Update report (CH2M 2015) were incorporated into the 2018 CSO LTCP Update.

## 1.2 2018 CSO LTCP Update

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- **Short-term Improvements –** These improvements consist of system upgrades to remove or control CSO discharge from selected CSO outfalls. These controls were developed in the 2015 Hydraulic Model Update (CH2M 2015) and generally use available in-system storage and small-scale gray and green projects to make improvements to the system at a relatively low cost. Short-term improvements were implemented between 2016 and 2020.
- **Intermediate-term Improvements –** These improvements consist of the Yale Campus/Trumbull Street Phase 2A Sewer Separation project and improvements to the remote pump stations that pump flow to the ESWPAF. The purpose of the pump station improvements is to pump the maximum wet weather flow to the plant for treatment. The improvements consist of upgrades to the East Street Pump Station, Union Pump Station, and Boulevard Pump Station. Intermediate improvement projects were initiated in and are ongoing.
- **Long-term Improvements –** The focus of the long-term improvement projects is to provide upgrades to the ESWPAF to treat a peak flow of 187 million gallons per day (mgd), complete the Fair Haven Sewer Separation Project, and incorporate necessary green and grey infrastructure to eliminate CSOs during the design storm level of service.

## 1.3 2022 CSO LTCP Update

The focus of this 2022 CSO LTCP Update is on the long-term wet weather improvements to the ESWPAF needed for the facility to meet discharge permit requirements over the next 20 years, and maintain the ability to remove nitrogen to the maximum extent possible while experiencing increases in flows and loads. As discussed herein, projected peak flows to the ESWPAF are expected to increase to 187 mgd because of improvements to the collection system to reduce and eliminate CSO discharge and population growth in the service area.

# 2. Nine Minimum Control Measures

The modification of the Consent Order WC5509 between the CT DEEP and the GNHWPCA requires the documentation of the Nine Minimum Control (NMC) measures and periodic summaries of the status of the control measures.

## 2.1 Project Background

Combined sewer systems (CSSs) carry a mixture of sanitary sewage and stormwater to a treatment facility via a single pipe. During wet weather, wastewater flows can exceed the capacity of the CSS and/or treatment facilities. In such an event, sewers are designed to overflow directly to surface water bodies, such as lakes, rivers, estuaries, or coastal waters. These overflows, CSOs, can be a source of water pollution.

As an effort to combat CSOs, EPA issued the CSO Control Policy on April 11, 1994. A key aspect of the policy is the NMCs, which are CSO-reducing measures that do not require significant engineering studies or major construction.

The NMCs are as follows:

1. Proper operation and regular maintenance programs for the sewer system and CSO outfalls.
2. Maximum use of the collection system for storage.
3. Review and modification of pretreatment requirements to ensure that CSO impacts are minimized.
4. Maximization of flow to the publicly owned treatment works for treatment.
5. Elimination of CSOs during dry weather.
6. Control of solid and floatable materials in CSOs.
7. Pollution prevention programs to reduce contaminants in CSOs.
8. Public notification of CSO occurrences and CSO impacts.
9. Monitor to effectively characterize CSO impacts and the efficacy of CSO controls.

Jacobs has completed updates to the NMCs Implementation Assessment included in the City of New Haven's previous CSO LTCPs. This updated assessment of the Authority's implementation of the NMC measures follows EPA's *Combined Sewer Overflows: Guidance for Nine Minimum Controls* (1995). For each of the NMCs, the status of control measures implemented are summarized and used to identify any deficiencies that would require future corrective action by the Authority (provided in **Appendix A**).

Based on the assessment, the Authority is in full compliance with implementation of the NMCs and no corrective action is required at this time.

Looking forward, the Authority should continue to assess and update all programs that support implementation of the NMCs, examples of which are as follows:

- CSO Flow Monitoring Program
- Monthly inspections of CSO regulators, CSO outfalls, and tidal check valves
- Hydraulic Model Updates
- Emergency Response Plans
- Regulator Improvement Program
- Capacity, Management, Operations and Maintenance Plan
- Large-diameter Sewer Cleaning Program
- Wet Weather Operational Plan at the ESWPAF
- CSO reporting in accordance with State regulations

Member communities should assess the implementation of catch basin cleaning and street cleaning programs to complement the Authority's efforts to meet the NMCs.

# 3. CSO LTCP Improvements 2017 through 2022

The GNHWPCA continues to work on several fronts to further the CSO control program. Short-term, intermediate-term, and long-term improvements have been identified and implemented as part of the program. In the years between 2017 and 2022, the GNHWPCA has undertaken the following projects:

- 2012-04 Design and Bidding of Yale Campus Trumbull Street Area Sewer Separation Phase 2 (A&B)
- 2016-01 Infiltration and Inflow Study Phase 1 – Mill River Trunk Sewer
- 2016-02 Regulator 012 and 020 Hydraulic Improvements – Closure of Regulators 012 and 020
- 2016-03 West River CSO Improvements - New Regulators 003, 004, and 006 and Weir Adjustments
- 2016-04 Infiltration and Inflow Study, SSES, Woodbridge and East Haven Areas 15, 18, & 23
- 2016-05 Regulator 025 and 034 Improvements – Closure of Regulator 034 and Weir Adjustment at 025
- 2016-06 Design and Construction I/I Removal Middle Thorpe Drive, Hamden and East Haven 5, 9 & 13
- 2016-07 Green Infrastructure Improvements in the West River Sewershed – Installation of 66 Bioswales
- 2017-01 Design of Capacity Improvements at East Street Pump Station for CSO Reduction
- 2017-02 CEPA Environmental Impact Evaluation Study for City of New Haven CSO LTCP Update 2018
- 2018-01 Infiltration and Inflow Study, SSES, Mill River Areas 7, 11, 15 & 22
- 2019-01 Infiltration and Inflow Study, SSES, Mill River Areas 6, 13, & 19
- 2019-03 Value Engineering for the Design of Capacity Improvements at East Street Pump Station
- 2019-04 Process Air Compressor Improvements at ESWPAF for Low Level Nitrogen Reduction
- 2019-05 Planning Study, Design and Bidding of Orchard Street Area Sewer Separation
- 2021-03 City of New Haven CSO LTCP Update 2022
- 2022-01 Fair Haven Regulator Improvements Phase 1 – Adjust Regulators 009, 015, & 016

## 3.1 Benefits of CSO Improvements 2017 to 2022

The completion of the Short-term Control Plan projects identified in the 2018 CSO LTCP Update was projected to reduce CSOs during a typical year from 30 million gallons per year in 2016 conditions, to 19 million gallons in the post-Short-term Control Plan conditions.

Specific improvements included in the Intermediate-term Control Plan and scheduled for implementation between 2022 and 2028 include the following:

- Capacity Upgrade to the East Street Pump Station
- Yale Campus/Trumbull Street Phase 2 Sewer Separation
- Orchard Street Sewer Separation
- Phase II ESWPAF Wet Weather Treatment Improvements
- Fair Haven Regulator Improvements – Phase 2
- 2027 Update to the LTCP

GNHWPCA maintains a CSO monitoring program and is currently seeing CSO volumes that mirror the 1967 Typical Year that is used as the baseline for comparison of improvements. Storms that exceed the intensity of storms identified during the 1967 Typical Year are excluded from the typical year projections. Examples of these most intense storms include the 25 year intensity named storm Elsa on July 8 and 9, 2021, which deposited 5.5 inches of rain in a 32-hour period. A second named storm, Ida, in September 2021, deposited 4.9 inches over 21 hours. Ida reached a peak hour rainfall of 2.23 inches, which is a return frequency of between a 50-year and 100-year event.

## 3.2 Summary

**Table 3-1** lists significant physical improvements made by the GNHWPCA to the CSS between 2017 and 2022. These improvements typically include raising weirs to reduce the number of potential overflows per year and upgrading pump stations and piping to permit the closing of regulators and the elimination of overflows.

**Table 3-1. Improvements and Closures of CSO Regulators (2017 – 2022)**

| No. | Discharge Location | Improvement |
|---|---|---|
| ***Improvements*** | | |
| #003 | West River | Weir raised 16 inches in 2020 |
| #004 | West River | Weir raised 24 inches in 2020 |
| #006 | West River | Weir raised 42 inches in 2020 |
| #009 | Mill River | Weir raised 8 inches in 2015 and by 6 inches in 2022 |
| #015 | Quinnipiac River | Weir raised 18 inches in 2022 |
| #016 | Quinnipiac River | Weir repaired in 2014 and raised 6 inches in 2022 |
| #024 | New Haven Harbor | Weir raised 1.5 feet in 2017 |
| #025 | New Haven Harbor | Weir raised 9.15 feet in 2019 |
| ***Closures*** | | |
| #010 (A) | Mill River | Closed in 2020 |
| #012 | Mill River | Closed in 2018 |
| #020 | Quinnipiac River | Closed in 2019 |
| #026 | Mill River | Closed in 2019 |
| #028 | Mill River | Closed in 2018 |
| #034 | New Haven Harbor | Closed in 2019 |

# 4. East Shore Water Pollution Abatement Facility Evaluation

## 4.1 Wastewater Flows and Loads

### 4.1.1 Introduction

Flow and load projections developed for the ESWPAF through the year 2045 are summarized in this section of the LTCP. A longer version of this section describing details of the analysis is included in **Appendix B**. The flow and loads presented herein were based on historical operator and laboratory data spanning the period from January 1, 2017, to December 31, 2021. Data for the analysis were collected at the primary influent sample collection point, and includes plant recycles and raw wastewater influent.

### 4.1.2 ESWPAF Description

**Figure 4-1** is a process flow diagram for the current wastewater treatment processes at the ESWPAF. Raw influent wastewater flow enters the plant through a gravity sewer and two force mains. The gravity sewer enters the existing headworks building, passes through coarse screens (3/4-inch openings) and grit removal before flowing by gravity to the existing pump station in the main building. The influent pump station conveys the gravity flow to the primary clarifiers. The two force mains are pumped directly to the primary clarifiers. The wastewater then flows by gravity through the remaining treatment processes including primary treatment, secondary treatment, and disinfection. Several return streams from the plant and delivered septage are combined with the influent flow before primary treatment. Influent waste and recycle streams incorporated into the influent at the sampling location are provided in the following subsections.

#### 4.1.2.1 Influent Waste Streams

- **Raw Wastewater Influent –** The service population for the ESWPAF is estimated to be approximately 233,150 people in the year 2022, resulting in an average raw wastewater flow of approximately 32.2 mgd.
- **Septage Receiving –** Septic haulers discharge their contents to the raw wastewater influent during typical business hours. An rolling average of approximately 102,000 gallons per month (approximately 5,000 gallons per weekday) are accepted by the plant.
- **Decant from Fats, Oils, and Grease (FOG) Receiving –** FOG haulers also discharge their contents at the ESWPAF. FOG is discharged to a decant tank that separates floatable and settleable material from water. Decanted water is discharged to the treatment process, while the settleable and floatable materials are sent directly to the incinerator. The plant accepts approximately 395,000 gallons of FOG materials per month.

#### 4.1.2.2 Recycle Waste Streams

- **Incinerator Scrubber Return –** Nonpotable service water (that is, ESWPAF secondary effluent) is used for wet scrubbing of the incinerator exhaust to meet air pollution control requirements. The discharge from the scrubber is recycled back to the ESWPAF upstream of the primary influent sample location.
- **Gravity Thickener Overflow –** Collected sludge from the primary clarifiers is pumped to gravity thickeners for thickening. Overflow from the gravity thickeners is recycled to the ESWPAF and enters upstream of the primary influent sample location.
- **Gravity Belt Thickener Filtrate –** The filtrate from thickening of waste-activated sludge (WAS) from the ESWPAF is recycled to the plant upstream of the primary influent sample location.

- **Dewatering –** Thickened primary sludge and WAS from the ESWPAF and a mixture of primary sludge and WAS from the Norwalk, Ansonia, Bridgeport, Branford, New Canaan, West Haven, and East Windsor wastewater treatment facilities (WWTFs) is dewatered at the ESWPAF. A combination of belt filter presses and centrifuges are used for dewatering. Filtrate from dewatering is recycled to the front end of the plant and enters upstream of the primary influent sample location.

### 4.1.3 Influent Flows

#### 4.1.3.1 Statistical Analysis

Flow data from the plant for the years 2017 through 2022, were analyzed statistically to identify average and peak flow conditions. The dataset was segregated into winter (November 1st - April 30th) and summer (May 1st - October 31st) seasons. Both 7-day and 30-day moving averages were calculated to identify peak week and maximum month (MM) flow.

The historical data were evaluated to identify and remove anomalies using the lognormal interquartile range (IQR) method. The IQR method compares the natural logarithm of the loading values to a calculated valid minimum and valid maximum for each data range. The lognormal distribution is often used to evaluate environmental statistics, which are generally positively skewed. Any value greater than the valid maximum or lower than the valid minimum is identified as a suspected outlier and removed from the data. The IQR is calculated as the difference between the 25th and 75th percentile of the historical data, which statistically represents 50 percent of the values. The valid minimum and maximum (that is, “fences”) were calculated by taking 2.5 times the IQR, which is then either added to the 75th percentile to develop the valid max or subtracted from the 25th percentile for the valid minimum. This IQR range statistically represents greater than 90 percent of the distribution. Minimum flows were not considered in the analysis.

The following conditions were identified:

- **Annual Average (AA) –** This is the average of all daily data for the entire period. A 12-month rolling average is used for ESWPAF as the basis of the 40-mgd design flow rate in the facility’s NPDES permit.
- **Maximum Month (MM) –** This is the maximum 30-day moving average during the analysis period and is the key sustained-flow design criteria. The MM value has been calculated for flow and loadings independently.
- **Maximum Week (MW) –** This is the maximum 7-day moving average during the analysis period. The MW value has been calculated for flow and loadings independently.
- **Maximum Day (MD) –** This is the maximum for flow and loadings that occurred in a single day during the analysis period.
- **Peak Hourly (PH) –** This is only determined for flow and is an important hydraulic capacity criterion for the total influent flow to the ESWPAF.

**Table 4-1** summarizes the primary influent flows and loads for the analysis period.

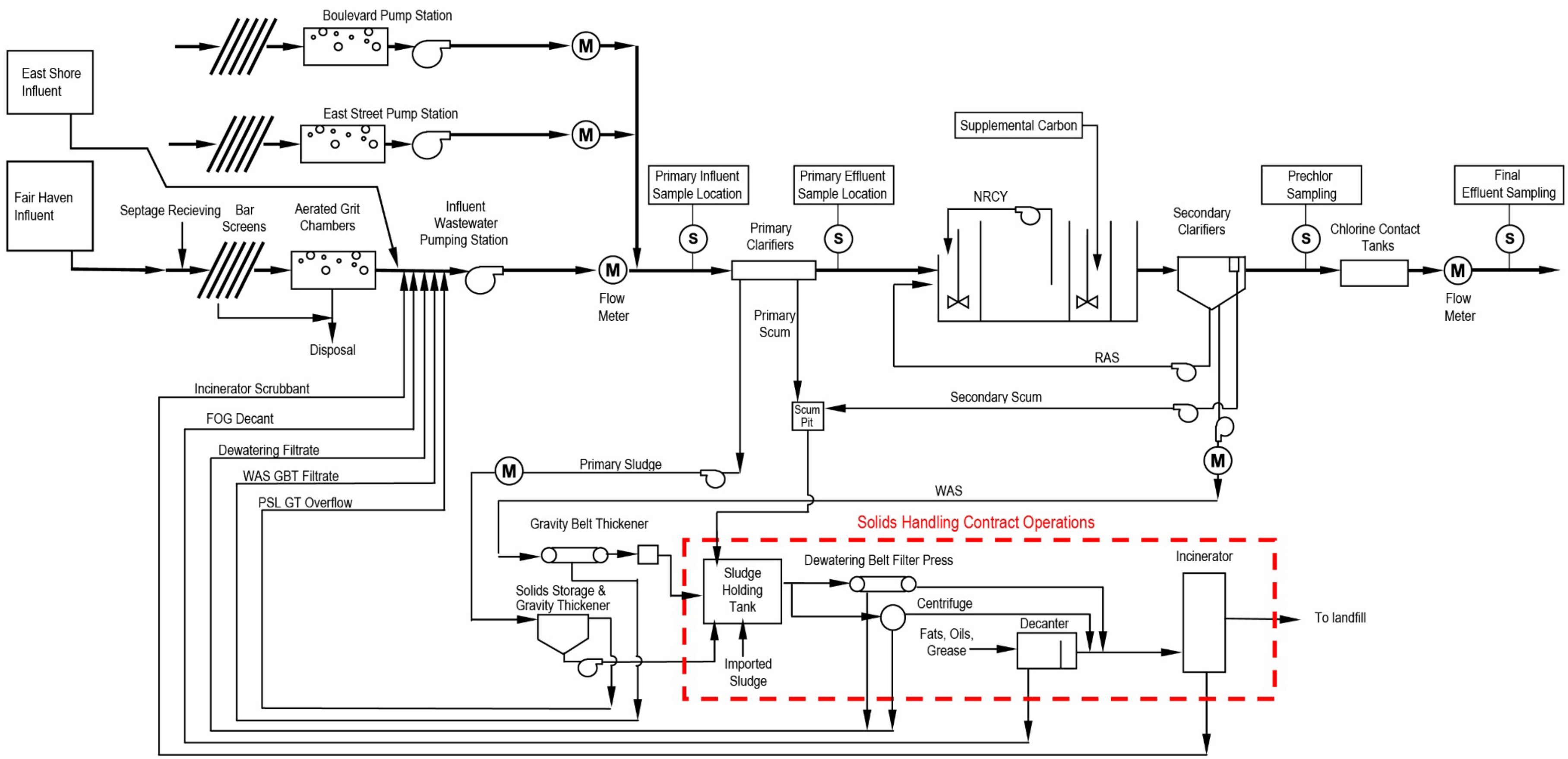



**Figure 4-1. ESWPAF Process Flow Diagram**

**Table 4-1. Summary of Historical Primary Influent Flows and Loads**

| Parameter | Flow (mgd) | Summer Flow (mgd) | Winter Flow (mgd) | TSS (PPD) | BOD-5 (PPD) | TN (PPD) | $NH_3$-N (PPD) | TP (PPD) | ORTHO-P (PPD) |
|---|---|---|---|---|---|---|---|---|---|
| AA | 29.6 | 27.9 | 31.3 | 84,300 | 93,950 | 8,846 | 5,308 | 3,033 | 1,704 |
| MM | 40.7 | 40.7 | 40.3 | 138,550 | 138,450 | 13,036 | 7,821 | 4,469 | 2,512 |
| MW | 47.8 | 45.2 | 47.8 | 176,050 | 161,500 | 15,206 | 9,124 | 5,214 | 2,930 |
| MD | 81.0 | 77.6 | 81.0 | 267,000 | 232,150 | 21,858 | 13,115 | 7,494 | 4,211 |
| PH | 110.6 | 110.6 | 105.8 | - | - | - | - | - | - |

Note: Historical flow and loading data is from the period between 2017 and 2021.
- = not applicable
BOD-5 = 5-day biochemical oxygen demand
$NH_3$-N = ammonia-nitrogen
ORTHO-P = orthophosphate
PPD = pounds per day
TN = total nitrogen
TP = total phosphorus
TSS = total suspended solids

#### 4.1.3.1.1 Historical Population

Historical population data were obtained from the U.S. Census Bureau for the 2010 and 2020 census years. A summation of the populations from New Haven, East Haven, Woodbridge, and Hamden was used as these communities all contribute flow to the ESWPAF. Using the total populations from 2010 and 2020, a growth rate of 1.5 percent per decade, or 0.15 percent per year, was calculated and used to estimate total historical population for 2017, 2018, 2019, and 2021. **Table 4-2** shows historical and estimated total populations from 2017 through 2021.

As a simplifying assumption, it was assumed that the total population of each community was serviced by the sanitary sewer collection system and not by onsite wastewater treatment systems such as residential septic tanks). For example, not all areas of Woodbridge are sewered.

**Table 4-2. Historical Population**

| Year | Census Data for Service Area[a] | Calculated Population Used for the Analysis[b] |
|---|---|---|
| 2010 | 228,986 | ----- |
| 2017 | - | 231,401 |
| 2018 | - | 231,748 |
| 2019 | - | 232,096 |
| 2020 | 232,202 | 232,202[a] |
| 2021 | - | 232,793 |

[a] 2010 and 2020 populations sourced from 2010 and 2020 U.S. Census Bureau.
[b] 2017, 2018, 2019, and 2021 populations were estimated using a population growth rate of 0.15 percent per year.

#### 4.1.3.1.2 Historical Flow Analysis

##### 4.1.3.1.2.1 Key Features of Statistical Flow Analysis

A base flow was developed to provide the means for estimating the impact of inflow and infiltration (I/I) reduction due to collection system rehabilitation and improvements. Base flows were estimated based upon daily flow data and were assumed to occur between July 1st through September 30th of the year, which tend to be the driest months of the year. Only days with no rainfall were included in the base flow estimates to minimize the contribution of I/I to the resulting base flow. Residual effects of rainfall (that is, I/I flows a day or two after rain days) was not considered.

Annual average flow is the average over the entire year, whereas seasonal average flow is the average flow over the seasons (for example, summer vs. winter). Peaking factors (PFs) were developed by dividing the flow condition (such as annual average and seasonal MM) by the historical base flow. Maximum month is typically how treatment facilities are designed since it aligns with monthly average compliance requirements. Peak-week and MD flows are used for sludge production and aeration needs.

Effluent flows were assumed to be equivalent to influent flows for the historical flow analysis. The historical influent daily average flow, winter months, and the 30-day moving average are shown on **Figure 4-2**. As can be seen on the figure, it is evident that historical peak flows tended to occur during the winter months. Winter months were classified as months that fall within the range of January 1st through April 30th and November 1st through December 31st of each calendar year. The correlation between increased peak flows and winter months is likely due to an increase in I/I into the collection system from increased precipitation during these months.

Historical flow and PFs are presented in **Table 4-3** for summer and **Table 4-4** for winter conditions. The annual baseflow, which roughly estimates the true population influenced flow with minimal influence from I/I was assumed to occur from July 1st through September 30th and only includes days when there was zero rainfall accumulation. The annual average is the true average over the entire year. Flow PFs were developed by dividing the flow condition (such as annual average and seasonal MM) by the historical base flow. Multiple summary conditions are outlined in each table.

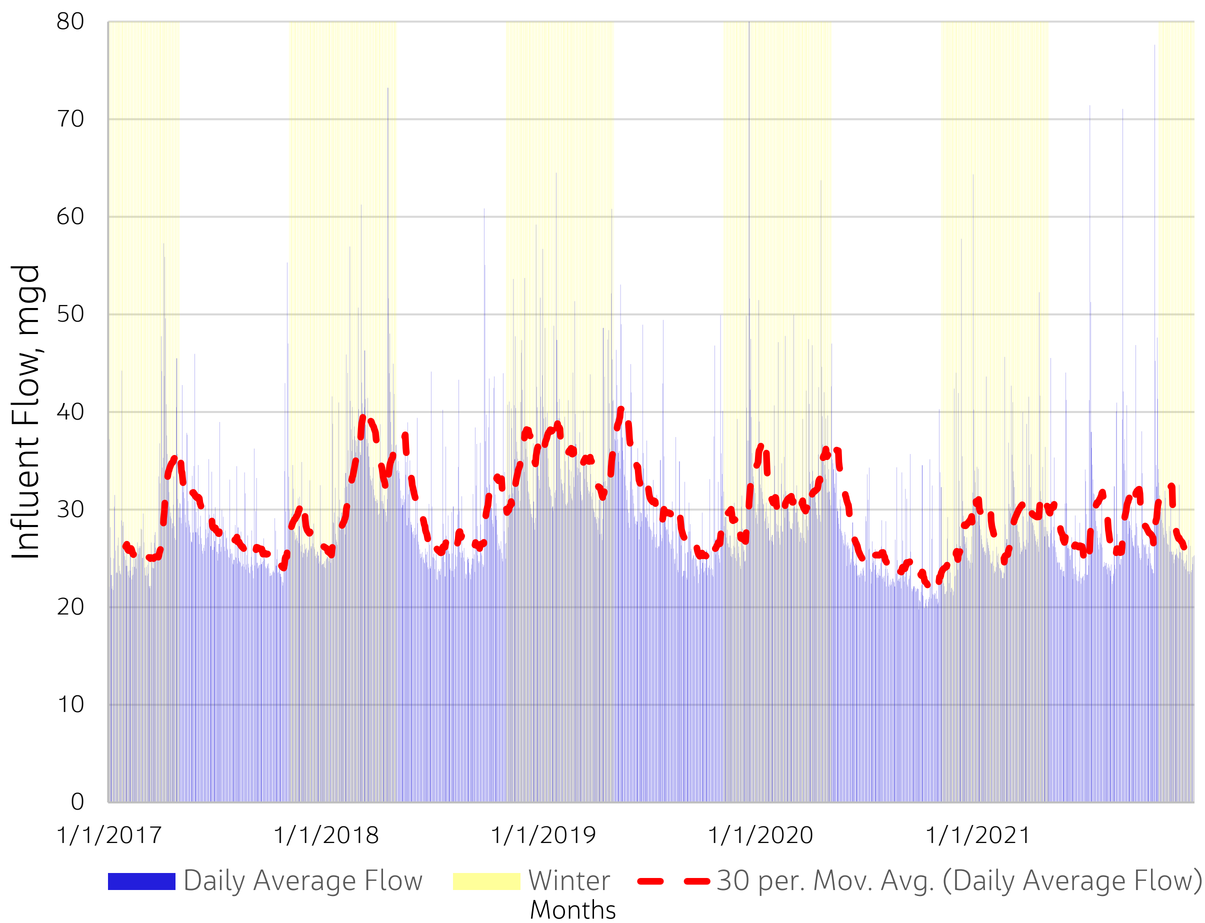



**Figure 4-2. Historical Daily Average and 30-day Moving Average Flows (2017-2021)**

**Table 4-3. Historical Flows and Peaking Factors for Average Annual and Summer Conditions**

| Year | Base Flow | All Year | | Summer (May 1 - Oct 31) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | AA | | Seasonal Average | | MM | | MW | | MD | |
| | | Flow | PF | Flow | PF | Flow | PF | Flow | PF | Flow | PF |
| 2017 | 25.0 | 27.7 | 1.11 | 27.4 | 1.10 | 35.1 | 1.40 | 36.9 | 1.48 | 55.3 | 2.21 |
| 2018 | 25.3 | 32.0 | 1.26 | 28.6 | 1.13 | 37.7 | 1.49 | 41.0 | 1.62 | 60.9 | 2.41 |
| 2019 | 26.1 | 32.0 | 1.23 | 29.8 | 1.14 | 40.7 | 1.56 | 45.2 | 1.73 | 53.1 | 2.04 |
| 2020 | 23.0 | 27.9 | 1.21 | 25.1 | 1.09 | 36.4 | 1.58 | 36.9 | 1.60 | 47.0 | 2.04 |
| 2021 | 27.5 | 28.3 | 1.03 | 28.6 | 1.04 | 32.1 | 1.17 | 44.0 | 1.60 | 77.6 | 2.83 |
| Average | 25.4 | 29.6 | 1.17 | 27.9 | 1.10 | 36.4 | 1.44 | 40.8 | 1.61 | 58.8 | 2.31 |
| Maximum | 27.5 | 32.0 | 1.26 | 29.8 | 1.14 | 40.7 | 1.58 | 45.2 | 1.73 | 77.6 | 2.83 |

Notes:
Flow units are mgd.
Data were provided for January 1, 2017 through December 31, 2021.
To determine MM and MW flows, running 30-day and 7-day averages were used.
PFs are calculated for each calendar year using the base flow for that year.

**Table 4-4. Historical Flows and Peaking Factors for Average Annual and Winter Conditions**

| Year | Base Flow | All Year | | Winter (November 1 – April 30) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | AA | | Seasonal Average | | MM | | MW | | MD | |
| | | Flow | PF | Flow | PF | Flow | PF | Flow | PF | Flow | PF |
| 2017 | 25.0 | 27.7 | 1.11 | 27.9 | 1.12 | 36.3 | 1.45 | 45.8 | 1.83 | 57.3 | 2.29 |
| 2018 | 25.3 | 32.0 | 1.26 | 35.4 | 1.40 | 40.3 | 1.59 | 47.8 | 1.89 | 73.2 | 2.90 |
| 2019 | 26.1 | 32.0 | 1.23 | 34.2 | 1.31 | 38.8 | 1.49 | 47.8 | 1.84 | 81.0 | 3.11 |
| 2020 | 23.0 | 27.9 | 1.21 | 30.8 | 1.34 | 37.5 | 1.63 | 42.3 | 1.84 | 64.4 | 2.80 |
| 2021 | 27.5 | 28.3 | 1.03 | 28.0 | 1.02 | 32.4 | 1.18 | 44.7 | 1.63 | 52.3 | 1.90 |
| Average | 25.4 | 29.6 | 1.17 | 31.3 | 1.24 | 37.1 | 1.47 | 45.7 | 1.80 | 65.6 | 2.60 |
| Maximum | 27.5 | 32.0 | 1.26 | 35.4 | 1.40 | 40.3 | 1.63 | 47.8 | 1.89 | 81.0 | 3.11 |

Notes:
Flow units are mgd.
Data were provided for January 1, 2017 through December 31, 2021.
To determine MM and MW flows, running 30-day and 7-day averages were used.
PFs are calculated for each calendar year using the base flow for that year.

#### 4.1.3.1.2.2 Annual Average Flows versus Precipitation

The historical annual average flows were compared to annual total precipitation to show the relation between the two. **Figure 4-3** shows total precipitation in comparison with the annual average flows from 2017 through 2021.

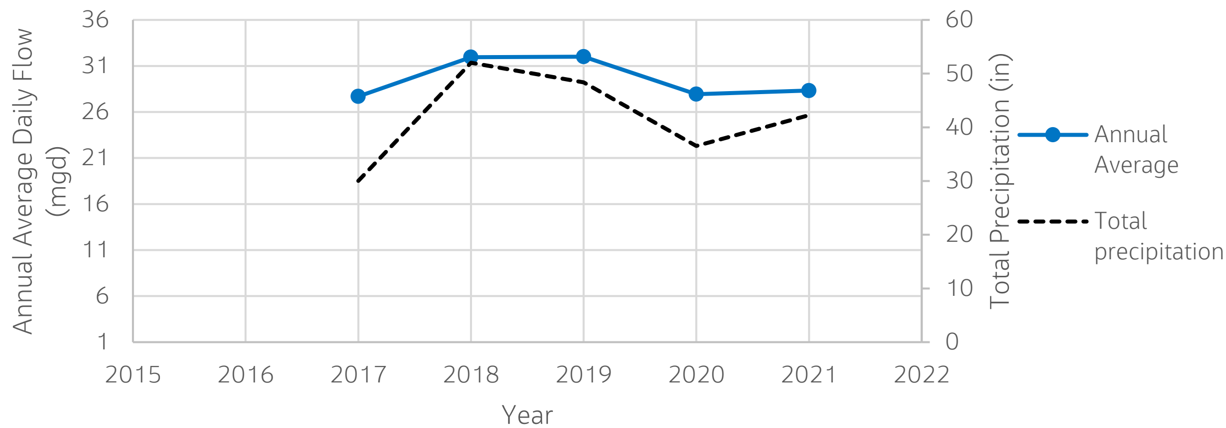



**Figure 4-3. Annual Average Flow and Total Annual Precipitation**

Precipitation data were sourced from the National Oceanic and Atmospheric Administration for the local area. **Figure 4-3** indicates that average annual flows generally mirror annual precipitation totals. However, the data for 2020 and 2021 show a significant drop in the flows relative to total precipitation during these two most recent years. The flow data do not have enough granularity to determine the causal relationship between average flow and precipitation. However, reductions in the contributions to base flow from universities and offices during the COVID outbreak may have contributed to recent reductions in annual average flow.

### 4.1.4 Historical Load Analysis

Historical loads were evaluated to determine the loading PFs to be applied for future load projections. The loading analysis was performed independently from the peak flow assessment described previously. Annual average load is the average of daily flows over the entire year, whereas seasonal average load is the average over the season (for example, summer vs. winter). Historical loads were obtained using the following procedure:

1. The influent load analysis was performed using historical data on TSS, BOD-5, TN, $NH_3$-N, TP, and ORTHO-P provided by the ESWPAF.

2. Historical mass loading rates were calculated from the flows and concentrations and the annual average loads, annual MM loads, and design loading conditions (summer and winter MM, MW, and MD) were extracted.

3. Loading data ranging from 2017 to 2021 was used for the loading analysis.

4. The average PF from 2017 to 2021 was selected for the annual average loading. The maximum PF from 2017 to 2021 was selected for the MM annual loadings and for the summer and winter weather MM, MW, and MD loadings.

5. The average annual loadings from 2017 to 2021 were calculated for each constituent.

6. The average historical loading per capita was calculated by taking the average of each year's per capita loading from 2017 to 2021.

7. Sewer serviced population projections were used to project future average annual loading rates of TSS, BOD-5, TN, $NH_3$-N, TP, and ORTHO-P.

8. PFs were then applied to the future average annual loading rates to generate future loads for the various design conditions.

#### 4.1.4.1 Loading Peaking Factors Summary

Historical data were used to develop mass load PF for all annual and seasonal conditions from 2017 to 2021. The annual averages were used to calculate the load PF A summary of the selected PFs from historical loading data is presented in **Table 4-5**. PFs were estimated when sufficient data were not available to clearly differentiate between seasonal impacts and the various flow conditions of interest. The peak loads and PF developed for each constituent are described in more detail in **Appendix B**.

**Table 4-5. Summary of Applied Peaking Factors For All Load Constituent's Projections**

| | TSS | BOD-5 | TN | $NH_3$-N | TP | ORTHO-P |
|---|---|---|---|---|---|---|
| MM Annual | 1.64 | 1.47 | 1.55 | 1.95 | 2.11 | 2.95 |
| MM Summer | 1.64 | 1.47 | 1.55 | 1.74 | 2.11 | 2.95 |
| MW Summer | 2.09 | 1.72 | 1.55 | 1.74 | 2.11 | 2.95 |
| MD Summer | 3.17 | 1.89 | 1.55 | 1.74 | 2.11 | 2.95 |
| MM Winter | 1.50 | 1.31 | 1.50 | 1.95 | 1.67 | 2.23 |
| MW Winter | 1.77 | 1.65 | 1.50 | 1.95 | 1.67 | 2.23 |
| MD Winter | 2.79 | 2.47 | 1.50 | 1.95 | 1.67 | 2.23 |

### 4.1.5 Historical Per Capita Flow and Load Estimates

#### 4.1.5.1 Per Capita Flow

Historical sewer serviced populations determined for 2017 to 2020 were used in conjunction with the previously determined average annual flows to estimate the flow per capita that is presented in **Table 4-6**. Per capita flows were calculated by dividing the annual base flow by that year's sewer serviced population. As noted above, flows appeared to drop in 2020, coincident with the pandemic.

**Table 4-6. Historical Per Capita Flows**

| Year | Sewer Serviced Population (capita) | Base Flow (mgd) | Flow per Capita (gal/d/capita) |
|---|---|---|---|
| 2017 | 231,401 | 25.0 | 108.2 |
| 2018 | 231,748 | 25.3 | 109.0 |
| 2019 | 232,096 | 26.1 | 112.2 |
| 2020 | 232,202 | 23.0 | 99.1 |
| 2021 | 232,793 | 27.5 | 118.0 |
| Average | 232,048 | 25.4 | 109.3 |

Notes:
Base flow units are mgd.
Flow per capita units are in gallons per day per capita (gal/d/capita).
2020 sewer serviced population taken from 2020 U.S. Census.
2017, 2018, 2019, and 2021 sewer serviced population calculated using a growth rate of 0.15 percent per year.
Growth rate determined using 2010 and 2020 U.S. Census data.

#### 4.1.5.2 Per Capita Loading

Historical sewer serviced populations determined for 2017 to 2021 were used in conjunction with the previously determined average annual loads to estimate the loading per capita for TSS, BOD-5, TN, $NH_3$-N, TP, and ORTHO-P. Historical data include recycles and bio-diesel, but does not include imported Sludge or FOG. Annual per capita loading for the constituents analyzed is shown in **Table 4-7** and **Table 4-8**. Per capita loads were calculated by dividing the average annual load by that year's population.

**Table 4-7. Historical Average Annual Load and Per Capita Loading for TSS, BOD-5, and TN**

| Year | Sewer Serviced Population (Capita) | TSS Loading | | BOD-5 Loading | | TN Loading | |
|---|---|---|---|---|---|---|---|
| | | Average Annual Load (PPD) | Per Capita Loading (PPD/Capita) | Average Annual Load (PPD) | Per Capita Loading (PPD/Capita) | Average Annual Load (PPD) | Per Capita Loading (PPD/Capita) |
| 2017 | 231,401 | 96,161 | 0.416 | 107,035 | 0.463 | 8,600 | 0.037 |
| 2018 | 231,748 | 85,933 | 0.371 | 108,385 | 0.468 | 9,657 | 0.042 |
| 2019 | 232,096 | 88,089 | 0.380 | 100,585 | 0.433 | 7,610 | 0.033 |
| 2020 | 232,202 | 82,607 | 0.356 | 82,553 | 0.356 | 7,615 | 0.033 |
| 2021 | 232,793 | 68,698 | 0.295 | 71,271 | 0.306 | 7,580 | 0.033 |
| Average | 232,048 | 84,298 | 0.363 | 93,966 | 0.405 | 8,212 | 0.035 |

Note:
Load units are PPD and include internal recycles.

**Table 4-8. Historical Average Annual Load and Per Capita Loading for $NH_3$-N, TP, and ORTHO-P**

| Year | Sewer Serviced Population (Capita) | $NH_3$-N Loading | | TP Loading | | ORTHO-P Loading | |
|---|---|---|---|---|---|---|---|
| | | Average Annual Load (PPD) | Per Capita Loading (PPD/Capita) | Average Annual Load (PPD) | Per Capita Loading (PPD/Capita) | Average Annual Load (PPD) | Per Capita Loading (PPD/Capita) |
| 2017 | 231,401 | 3,216 | 0.014 | 3,635 | 0.016 | 1,983 | 0.009 |
| 2018 | 231,748 | 3,424 | 0.015 | 4,220 | 0.018 | 2,852 | 0.012 |
| 2019 | 232,096 | 3,211 | 0.014 | 2,890 | 0.012 | 1,371 | 0.006 |
| 2020 | 232,202 | 3,338 | 0.014 | 2,352 | 0.010 | 1,244 | 0.005 |
| 2021 | 232,793 | 2,826 | 0.012 | 1,911 | 0.008 | 1,088 | 0.005 |
| Average | 232,048 | 3,203 | 0.014 | 3,002 | 0.013 | 1,708 | 0.007 |

Note:
Load units are PPD.

## 4.1.6 2045 Flows and Loads

### 4.1.6.1 Population Projections

Future sewer serviced populations were calculated from 2022 through 2045, using the historical annual growth factor of 0.15 percent per year. As previously discussed, a per capita growth rate of 1.5 percent occurred from 2010 to 2020 (0.15 percent annually). **Table 4-9** summarizes population projections for each 5-year increment between 2025 and 2045 used for this evaluation, based on the 0.15 percent annual growth rate.

The projected service area population appears to be consistent with population projections from the State of Connecticut. For example, the State of Connecticut projects a total population of 250,000 for the area in 2040. However, the GNHWPCA indicates that approximately 15,000 people live in areas that are not serviced by sewers. Consequently, the population of the "service area" is likely very similar to the values shown in **Table 4-9** (that is, approximately 240,000 in 2040 and 242,000 in 2045).

**Table 4-9. 2025-2045 Population Projections for the Service Area**

| Year | Population Projection (Capita) |
|---|---|
| 2025 | 234,193 |
| 2030 | 235,954 |
| 2035 | 237,729 |
| 2040 | 239,518 |
| 2045 | 241,319 |

Note:
Population projections were determined using 2010 through 2020 annual population growth rate of 0.15 percent.

### 4.1.6.2 Flow and Load Projections

#### 4.1.6.2.1 Flow Projections – Scenario-Based Peaking Factors

Two scenarios were developed for the purpose of selecting an appropriate PFs for flow. The first scenario (that is, Scenarios 1) uses an average applied PFs for calculating projected flows. The second scenario (that is, Scenario 2) uses maximum-applied PFs. Because the average PFs are smaller than the maximum PF, the projected flows in the first scenario will be smaller than those in the second scenario. **Table 4-10** summarizes the PFs for flow for each scenario, as previously determined in **Table 4-3** and **Table 4-4**.

**Table 4-10. Scenario Based Applied Peaking Factors**

| Scenario | Selection Base | Applied Peaking Factors for Flow (to Base Flow) | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | Average Annual | | Summer MM | Summer MW | Summer MD | Winter MM | Winter MW | Winter MD |
| Scenario 1 | Average | 1.17 | 1.44 | 1.61 | 2.31 | 1.24 | 1.80 | 2.60 | |
| Scenario 2 | Maximum | 1.26 | 1.58 | 1.73 | 2.83 | 1.63 | 1.89 | 3.11 | |

Using the population projection for 2045, base flows were projected by dividing the average flow per capita by the projected sewer serviced population for that year. The remaining flow parameters were projected by multiplying the selected PFs shown in **Table 4-10** by the projected base flows. The base flows and projected flows for each scenario are summarized in **Table 4-11**.

**Table 4-11. Flow Projections For Each Scenario**

| Year | Scenario | Base Flow (mgd) | Average Annual (mgd) | Summer | | | Winter | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | MM | MW | MD | MM | MW | MD |
| 2045 | 1 (Average) | 26.4 | 30.8 | 38.0 | 49.6 | 60.8 | 38.7 | 47.6 | 68.6 |
| 2045 | 2 (Maximum) | 26.4 | 33.4 | 41.7 | 45.8 | 74.5 | 43.0 | 49.9 | 82.0 |

Notes:
A service population of 241,319 persons is projected for 2045.

Jacobs recommended that Scenario 2 (based on the maximum PF) be used as the basis of design for projecting 2045 flows (CSO LTCP Update Workshop No. 2, March 10, 2022).

#### 4.1.6.2.2 Loading Projections

The average annual residential loading projections were developed for TSS, BOD-5, TN, $NH_3$-N, TP, and ORTHO-P using the calculated load per capita and the projected populations. The loading PFs previously presented in **Table 4-7** and **Table 4-8** were applied to the average annual loading to generate the future peak loading conditions presented in **Table 4-12**.

**Table 4-12. Loading Projections for 2045 (PPD)**

| | Load per Capita | Sewer Serviced Population | Average Annual | MM Annual | MM Summer | MW Summer | MD Summer | MM Winter | MW Winter | MD Winter |
|---|---|---|---|---|---|---|---|---|---|---|
| TSS | 0.363 | 241,319 | 87,650 | 144,100 | 144,100 | 183,050 | 277,650 | 131,700 | 155,200 | 244,700 |
| BOD-5 | 0.405 | 241,319 | 97,700 | 144,000 | 144,000 | 167,950 | 184,650 | 128,200 | 161,550 | 241,450 |
| TN | 0.035 | 241,319 | 9,200 | 13,550 | 13,550 | 15,800 | 17,400 | 12,050 | 15,200 | 22,750 |
| $NH_3$-N | 0.014 | 241,319 | 5,500 | 8,150 | 8,150 | 9,500 | 10,450 | 7,250 | 9,150 | 13,650 |
| TP | 0.013 | 241,319 | 3,150 | 4,650 | 4,650 | 5,400 | 5,950 | 4,150 | 5,200 | 7,800 |
| ORTHO-P | 0.007 | 241,319 | 1,750 | 2,600 | 2,600 | 3,050 | 3,350 | 2,350 | 2,950 | 4,400 |

Note:
Load units are PPD and include internal recycles.

#### 4.1.6.2.3 Basis for Design

The summary of the historical flows and loads presented in **Table 4-1** are presented here in **Table 4-13** for reference. Flow and loading data were based between 2017 and 2022.

**Table 4-13. Summary of Historical Primary Influent Flows and Loads**

| Parameter | Flow (mgd) | Summer Flow (mgd) | Winter Flow (mgd) | TSS (PPD) | BOD-5 (PPD) | TN (PPD) | $NH_3$-N (PPD) | TP (PPD) | ORTHO-P (PPD) |
|---|---|---|---|---|---|---|---|---|---|
| AA | 29.6 | 27.9 | 31.3 | 84,300 | 93,950 | 8,846 | 5,308 | 3,033 | 1,704 |
| MM | 40.7 | 40.7 | 40.3 | 138,550 | 138,450 | 13,036 | 7,821 | 4,469 | 2,512 |
| MW | 47.8 | 45.2 | 47.8 | 176,050 | 161,500 | 15,206 | 9,124 | 5,214 | 2,930 |
| MD | 81.0 | 77.6 | 81.0 | 267,000 | 232,150 | 21,858 | 13,115 | 7,494 | 4,211 |
| PH | 110.6 | 110.6 | 105.8 | - | - | - | - | | - |

Note:
Flow and loading data ranges from 2017-2021.

Scenario 2 is based on maximum PF. The flow and load projections that form the basis of design for the CSO LTCP Update and future improvements to the ESWPAF are summarized in **Table 4-14**.

**Table 4-14. Summary of 2045 Projected Primary Influent Flows and Loads**

| Condition | Flow (mgd) | 2045 | | | | | |
|---|---|---|---|---|---|---|---|
| | | TSS (PPD) | BOD-5 (PPD) | TN (PPD) | $NH_3$-N (PPD) | DP (PPD) | ORTHO-P (PPD) |
| Base Flow | 26.4 | - | - | - | - | - | |
| AA | 33.4 | 87,650 | 97,700 | 9,199 | 5,519 | 3,154 | 1,772 |
| MM Annual | - | 144,100 | 144,000 | 13,558 | 8,135 | 4,649 | 2,612 |
| MM Summer | 41.7 | 144,100 | 144,000 | 13,558 | 8,135 | 4,649 | 2,612 |
| MW Summer | 45.7 | 183,050 | 167,950 | 15,813 | 9,488 | 5,422 | 3,047 |
| MD Summer | 74.5 | 277,650 | 184,650 | 17,386 | 10,431 | 5,961 | 3,350 |
| MM Winter | 43.0 | 131,700 | 128,200 | 12,071 | 7,242 | 4,139 | 2,326 |
| MW Winter | 49.9 | 155,200 | 161,550 | 15,211 | 9,126 | 5,215 | 2,931 |
| MD Winter | 82.0 | 244,700 | 241,450 | 22,734 | 13,640 | 7,794 | 4,380 |
| PH | 187.0 | - | - | - | - | - | - |

## 4.2 Plant Hydraulic Profile

Upgrades to the ESWPAF in New Haven to accommodate CSO flows are being undertaken in at least three phases of work. Phase 1 of the planned upgrade work has been completed, and included the following components:

- Electrical Upgrades
- Odor Control Upgrades
- Nitrogen Removal Improvements (Carbon Addition, 2nd Anoxic Zone)
- Gravity Thickener and Sludge Storage
- Process Air Compressor Upgrade

This section of the LTCP discusses the impacts of the proposed Phase 2 and Phase 3 upgrades on the hydraulic profile of the ESWPAF. Hydraulic profiles for each phase of work are included in the Drawings (**Appendix D**). The assumptions and methodology made for the development of the hydraulic profile are described in the following section and the results of the evaluation are described in subsequent paragraphs.

### 4.2.1 Design Criteria

The following design criteria, including historical water level elevations and regulatory guidance, informed the development of the new hydraulic profiles.

#### 4.2.1.1 Hydraulic Profile Flows

Hydraulic evaluations of the proposed facility for 2002, Phase 2, and Phase 3 were based on the flow conditions presented in **Table 4-15.**

**Table 4-15. Hydraulic Profile Flows**

| Flow Stream | 2022 | Phase 2 | Phase 3 |
|---|---|---|---|
| ***Plant Flows (mgd)*** | | | |
| Total Plant Flow | 100 | 147 | 187 |
| Flow to Secondary | 60 | 60 | 60 |
| Wet Weather Flow | 40 | 87 | 127 |
| ***Internal Recycle Flows (mgd)*** | | | |
| RAS Recycle | 40 | 40 | 40 |
| NRCY Recycle | 80 | 80 | 80 |

NRCY = nitrified mixed liquor recycle
RAS = return activated sludge

The peak hydraulic flow for the ESWPAF was established through an evaluation described in the report *Wet Weather Capacity Improvements and Nitrogen Reduction at the East Shore Water Pollution Abatement Facility* (CH2M 2011a). The maximum conveyance to the ESWPAF is 187 mgd during a 2-year, 6-hour duration design storm event. The Phase 2 flow of 147 mgd is based on the maximum capacity of the existing primary clarifiers with all units in service. The maximum flow to secondary treatment is 60-mgd for both Phase 2 and Phase 3.

The resulting wet weather flows of 87 mgd for Phase 2 and 127 mgd for Phase 3 were determined by subtracting the flow to secondary treatment from the total plant flow. The RAS and NRCY recycles are the maximum recycle flows the plant is currently designed to accommodate.

#### 4.2.1.2 Flow Path

The implementation plan for the proposed plant upgrades for the ESWPAF assumes that two future phases of work that (that is, Phase 2 and Phase 3) will impact the hydraulic profile. These two phases of work may be further subdivided into multiple projects to facilitate their implementation. As previously noted, Phase 1 improvements are complete.

The main focus of the Phase 2 upgrades is to construct a wet weather treatment facility downstream of the primary clarifiers at the ESWPAF. This project will include a facility to split primary effluent flow between secondary treatment and wet weather disinfection, two wet-weather chlorine contact tanks, and two new effluent lines to convey disinfected effluent to the plant outfall.

The hydraulic profile for Phase 2 is calculated between the three existing primary clarifiers and the plant outfall, and assumes that all processes are online. All Primary Effluent (PE) flow will pass through the new flow-splitting facility, where flow will be split to either secondary treatment or wet weather disinfection.

The first flow path (existing secondary treatment and disinfection), consists of four aeration basins, eight secondary clarifiers, and two chlorine contact tanks. Disinfected secondary effluent then discharges from the plant through twin 48-inch outfalls and a 90-inch square overflow trough adjacent to the plant outfall. A maximum flow of 60 mgd is anticipated for the secondary treatment and disinfection train.

The second flow path (the wet weather treatment facility) consists of two new wet weather disinfection tanks and two 72-inch discharge pipes. The two 72-inch pipes ultimately intersect with disinfected flow from the first flow path at the 90-inch outfall overflow trough. All flow greater than 60 mgd will be conveyed to the wet weather disinfection facility. The flow schematic and hydraulic profiles for Phase 2 are shown in Figures 2 through 5 of **Appendix D.**

The Phase 3 upgrades include a new preliminary treatment facility, a new flow-splitting facility upstream of the primary clarifiers, a fourth primary clarifier, and upgrades to the aeration basins. A hydraulic evaluation was performed to determine whether a second outfall would be needed for the Phase 3 flow condition, which has a peak flow of 40 mgd greater than that of Phase 2. Consequently, two hydraulic profiles were developed for Phase 3 for the purpose of determining whether a new outfall would be needed or beneficial for the ESWPAF:  one without a new outfall and a second with the new outfall.

The first outfall scenario is the same as Phase 2 (that is, with higher flows but no changes to the existing outfall configuration). The second outfall scenario has the two 72-inch wet weather discharge pipes from the Phase 2 construction being extended into the harbor as a new outfall. The flow schematic and hydraulic profiles for Phase 3 are shown on Figures 6 and 10 in **Appendix D**.

The hydraulic profiles for Phase 3 assume all processes are online and runs through the new preliminary treatment facility, which consists of four screens followed by six head cells for grit removal. After preliminary treatment, the flow path is generally the same as that for Phase 2, with the exception of the two different outfall scenarios.

#### 4.2.1.3 Harbor Elevations

*Technical Release (TR)16, Guides for the Design of Wastewater Treatment Works* (TR-16; NEIWPCC 2016) were used to determine the appropriate design storm and resulting flood elevations for the evaluation. TR-16 considers two main categories of WWTFs: (1) existing facilities, and (2) new facilities. Section 1.2 of TR-16 states that existing facilities should have uninterrupted operation of all units during 25-year flood conditions, and should be protected against the structural, process, and electrical equipment damage that might occur in a 100-year flood.

New facilities are required to have uninterrupted operation of all units during a 100-year flood elevation. All noncritical equipment should be protected at a water surface elevation that is two-feet above the 100-year flood, and all critical equipment should be protected at a water surface elevation that is three-feet above the 100-year flood. Existing plants planning for expansion or upgrades should be improved to the maximum extent possible to meet the new criteria. TR-16 advises that, "until such time that FEMA or ACOE flood criteria are amended to include the impact of climate change, a greater measure of flood protection may be warranted."

Since the ESWPAF is an existing facility, the 25-year tidal flood elevation is the governing criteria for this analysis. The North American Vertical Datum (NAVD) from 1988 (NAVD 88) was used as the basis for the hydraulic profiles. Hydraulic profiles developed for previous reports were based on other vertical datum standards.

A harbor elevation of 8.3 feet (NAVD 88) for the 25-year tidal flood elevation was used for the Phase 2 scenario and the Phase 3 scenario without a new outfall. The 25-year flood elevation was calculated by using the 50-year and 10-year flood elevations as reference points. The 50-year harbor elevation of 8.8 feet (NAVD 88) and 10-year harbor elevation of 7.5 feet (NAVD 88) were determined using the 1988 U.S. Army Corps of Engineers (USACE) New England Coastline Tidal Flood Survey. The published Federal Emergency Management Authority (FEMA) Risk Insurance Report elevations are lower than the USACE elevations. Thus, the more conservative USACE values were used for this evaluation.  An elevation of 12.0 feet (NAVD 88) was assumed for the 100-year tidal flood, based on FEMA Flood Hazard Maps. This elevation is more conservative than the 1988 USACE New England Coastline Tidal Flood Survey and the FEMA Risk Insurance Reports. The additional 3.7 feet of head represented by the 100-year flood would submerge the secondary treatment weirs. The value of the 100-year flood elevation is provided here for context and reference purposes. However, no hydraulics calculations were performed at this flood elevation.

#### 4.2.1.4 Modeling

Hydraulic modeling of Phase 2 and Phase 3 was performed using the WinHYDRO computer model. WinHYDRO is a gravity flow systems program developed by Jacobs that computes energy losses beginning at the downstream end of a treatment plant. Working upstream through the plant, the energy grade and hydraulic grade line are established at the downstream and upstream end of each defined hydraulic element (for example, pipelines and weirs). The following assumptions and techniques were utilized in developing, running, and analyzing the hydraulic model and the model results:

- All units are online.
- A Manning's n value of 0.015 was used for all channels and piping.
- A Manning's n value of 0.035 was used for the aeration basins tanks.
- Flow is equally split between identical process units or equipment.
- The longest flow path was modeled.
- The model was developed using available mechanical, structural, and civil plan, and profile record drawings and the drawings included in this report for the future upgrades.
- All elevations are based on the NAVD 88 datum.
- Subcritical flow in open channels was assumed. WinHYDRO does not consider critical or supercritical flow.

### 4.2.2 Phase 2 Hydraulic Evaluation

The goal of the Phase 2 hydraulic evaluation was to identify and minimize any hydraulic constraints within the ESWPAF and set elevations for the Wet Weather Disinfection Facility. **Table 4-16** shows the various process units, their existing and proposed (Phase 2) elevations, and the hydraulic grade line elevations at various points. Figures 3 through 5 in **Appendix D** show the Phase 2 hydraulic grade line through the plant for intermediate flows of up to 147 mgd.

**Table 4-16. Phase 2 Hydraulic Evaluation (Peak Flow of 147 mgd)**

| Process Unit | Wall Elevation (feet ) | Baffle Wall Elevation (feet) | Effluent Weir Elevation (feet) | Influent Water Surface (feet) | Effluent Channel Water Surface (feet) |
|---|---|---|---|---|---|
| Primary Clarifiers | 22.0 | N/A | 17.8 | 18.9 | 17.5 |
| Splitter Box (Phase 2) | 23.0 | N/A | N/A | 16.3 | 16.3 |
| Wet Weather Disinfection (Phase 2) | 23.0 | 16.5 | 13.9 | 15.5 | 10.4 |
| Aeration Basins | 17.5 | N/A | 13.5 | 14.8 | 12.3 |
| Secondary Clarifiers | 17.5 | N/A | 11.4 | 12.3 | 10.2 |
| Chlorine Contact Tanks | 17.5 | 11.0 | 8.4 | 10.0 | 8.7 |

N/A = not applicable
All elevations are NAVD 88.

### 4.2.3 Plant Impacts at Phase 2 Flows

The hydraulic profiles for the Phase 2 upgrade project shown on Figures 3 through 5 of **Appendix D** were calculated for the 25-year flood and a harbor elevation of 8.3 feet (NAVD 88) at a total plant flow of 147 mgd. As shown in the profiles, the chlorine contact effluent weir is submerged. The existing primary clarifier weirs in this scenario were found to be at 17.5 feet (NAVD 88), just below the weir elevation of 17.8 feet (NAVD 88). No hydraulic modifications to the existing process is required to proves the Phase 2 flows at the 25-year flood and harbor elevation.

### 4.2.4 Phase 3 Hydraulic Evaluation

The goal of the Phase 3 hydraulic evaluation was to identify and minimize hydraulic constraints within the ESWPAF, set elevations for future facilities, and determine whether a new outfall was needed for Phase 3 flows. Figures 6 and 10 in **Appendix D** shows the main process flow schematics for Phase 3, without and with a new outfall.

**Table 4-17** shows the weir and baffle wall elevations for the main unit processes and the effluent channel water elevations at the Phase 3 flow of 187 mgd and the 25-year harbor elevation of 18.3 feet (NAVD 88). Figures 7 through 9 in **Appendix D** show the hydraulic profile through the plant with the Phase 3 upgrades and without a new outfall.

The evaluation indicated that existing primary clarifier weirs would be flooded under these operating conditions. Consequently, the height of the primary clarifier weirs and the scum collection equipment will need to be raised from 17.8 feet (NAVD 88) to 18.3 feet (NAVD 88) in Phase 3 to mitigate the flooding potential and that recommendation is reflected in **Table 4-17**. The evaluation also indicated that the chlorine contact effluent weir would be flooded at Phase 3 flows. However, the basin will be able to operate properly because the baffle walls would not be flooded.

**Table 4-17. Phase 3 Hydraulic Evaluation (Peak Flow of 187 mgd)**

| Process Unit | Wall/Baffle Elevations (feet) | Effluent Weir Elevation (feet ) | Effluent Channel Elevation (feet) Without New Outfall | Effluent Channel Elevation (feet) With New Outfall |
|---|---|---|---|---|
| Screening Channel | 29.0/N/A | N/A | 23.4 | 23.4 |
| Head Cells | 27.0/N/A | N/A | 22.0 | 22.0 |
| Primary Clarifiers | 22.0/NA | 18.3 [a] | 18.1 | 18.1 |
| Splitter Box (Phase 2) | 23.0/N/A | N/A | 16.3 | 16.3 |
| Wet Weather Disinfection (Phase 2) | 23.0/16.5 | 13,9 | 10.6 | 10.7 |
| Aeration Basins | 17.5/N/A | 14.8 | 12.3 | 12.3 |
| Secondary Clarifiers | 17.5/N/A | 12.3 | 10.2 | 10.1 |
| Chlorine Contact Tanks | 17.5/11.0 | 8.4 | 8.9 | 8.4 |

[a] Primary clarifier weirs to be raised from 17.8 feet (NAVD 88) to 18.3 feet (NAVD 88) in Phase 3.

All elevations are NAVD 88.

All modeling was completed at the 25-year harbor elevation of 8.3 feet (NAVD 88).

#### 4.2.4.1 Limiting Process Units

Multiple iterations of the Phase 3 model without the new outfall were evaluated to determine the elevation at which a new outfall would be necessary. It was determined that the chlorine contact tank baffle walls would become submerged at a harbor elevation of 10.0 feet (NAVD 88). At harbor elevations greater than 10.4 feet (NAVD 88), the secondary clarifier weirs become submerged. Operations adjustments would need to be made to minimize solids and scum carryover under these conditions.

The wet weather flow-splitting facility weirs be impacted at a harbor elevation of 11.5 feet (NAVD 88), but the facility would still be operational. Preliminary indications are that the facility cannot to be set at a higher elevation because it needs to fit between the primary clarifiers and secondary treatment within the existing plant profile.

## 4.2.5 Outfall Capacity

Iterations of the Phase 3 model with a new outfall were also performed for comparison purposes. At harbor elevations greater than 10.6 feet (NAVD 88) the chlorine contact tank baffle walls become submerged. At harbor elevations greater than 10.9 feet (NAVD 88) the secondary clarifier weirs become submerged.

The comparison of the Phase 3 models indicates that there is no significant benefit to building an additional outfall. Without a new outfall, the new wet weather disinfection facility can treat influent flows up to 187 mgd at a harbor elevation of 11.5 feet (NAVD 88) before flow backs up to the effluent weirs. A new outfall would allow the facility to operate without disruption at the 100-year design storm harbor elevation of 12.0 feet (NAVD 88). However, flows between elevations 11.5 and 12.0 would not cause a major disruption to operations.

The ESWPAF is required to have uninterrupted plant processes under a 25-year design storm. Both the Phase 2 and Phase 3 facilities were able to treat the influent flows without interruptions to the plant processes without a third outfall. Therefore, a new outfall is not recommended as part of the Phase 3 upgrades.

## 4.2.6 Hydraulic Evaluation - Summary

### 4.2.6.1 Design Storm

Per TR-16 requirements, existing facilities should have uninterrupted operation of all units during 25-year flood conditions, and should be protected against the structural, process, and electrical equipment damage that might occur in a 100-year flood. A harbor elevation of 8.3 feet (NAVD 88) was selected to be representative of the 25-year design storm.

### 4.2.6.2 Phase 2 Flows

A new wet weather treatment facility will be constructed during Phase 2 to accommodate influent flows of up to 147 mgd at a harbor elevation of 8.3 feet (NAVD 88) during the 25-year design storm. The 25-year design storm does not negatively impact the hydraulic capacity of the existing primary and secondary treatment systems, disinfection, or the existing outfall.

### 4.2.6.3 Phase 3 Flows

- Phase 3 flows without a new outfall
  - The hydraulic profile for Phase 3 flows of up to 187 mgd at a harbor elevation of 8.3 feet (NAVD 88) does not impact existing secondary treatment, disinfection, or existing outfall capacity during the 25-year design storm. Primary clarifier weirs and scum collection equipment will need to be raised to accommodate these flow conditions.
  - The harbor elevation can rise to a level of 10.0 feet (NAVD 88) at Phase 3 flows without negatively impacting existing secondary treatment or outfall capacity.
  - For future reference, the new wet weather facility can treat influent flows up to 187 mgd at a harbor elevation of 11.5 feet (NAVD 88) without impact to the wet weather disinfection facility, primary treatment, or preliminary treatment.

- Phase 3 with a new outfall
  - Phase 3 flows can be accommodated without any impact to secondary treatment or outfall capacity up to a harbor elevation of 10.6 feet (NAVD 88) without impact existing secondary treatment or existing outfall capacity.
  - The new wet weather facility can treat influent flows up to 187 mgd up to a harbor elevation of 12.0 feet (NAVD 88) without impact to the wet weather facility, primary treatment or preliminary treatment.
  - The hydraulic evaluation indicates that adding a new outfall will improve the plant's ability to mitigate the impacts of an additional rise in harbor elevation of approximately 0.5 feet greater than the impact on the plant without an additional outfall.

#### 4.2.6.4 Recommendations

An additional outfall is not recommended because the benefit is relatively modest compared to the potential cost and effort of constructing the outfall.

## 4.3 Preliminary Treatment

### 4.3.1 Existing Conditions

Screening and grit removal are currently performed at major pump stations for flow pumped to the ESWPAF using rake screens with 3/4-inch openings. Gravity flow to the plant undergoes screening and grit removal in the existing inlet works building.

It was recommended that the unit processes of fine screening and grit removal are to be centralized at the ESWPAF in a new preliminary treatment building. The pump stations within the collection system and the existing headworks building will continue to provide screening and grit removal to protect equipment and minimize deposition of materials within the collection system.

### 4.3.2 Process Description

The new preliminary treatment building at the ESWPAF will house fine screens, grit-removal equipment, truck unloading, and associated truck-loading facilities. The building will be located to the east of the gravity thickeners. The new preliminary treatment building will consolidate all preliminary treatment functions in one location, provide improved screening and grit removal, and influent flow measurement without the impact of plant recycle flows.

Flow to the preliminary treatment building will be via two force mains, one 48-inch diameter force main from the collection system pump stations and one from the main pump station. The main station conveys gravity flows from incoming 42-inch and 54-inch gravity sewers into the plant. Flow from the preliminary treatment building will be conveyed via two 72-inch diameter gravity lines to flow-distribution piping in the primary sludge gallery.

The preliminary treatment building will be a two-story building with a basement. The basement will need to be flood proof, or at least three feet above the floodplain elevation. The basement area will house the two 48-inch influent force mains, flow metering, sump pumps, and the piping manifold in a pipe gallery to allow preliminary treatment trains to be taken out-of-service. The grit pumps, bottom cone portions of the grit separators, and large diameter effluent pipes will be located at the rear of the building.

The front of the first floor will house the fine screen header, flow channels, and bottom portions of the fine screens. A truck-loading bay will be located on each side of the building (each bay will accommodate two dumpsters). To the rear of the first floor will be the grit separator header, channels, and modular stacked-tray vortex grit separator units. The first floor will also house a heating, ventilation, and air conditioning room, an electrical room, and a control room. At the front of the second floor will be the screening room that includes the top portions of the fine screens, fine screen wash-presses, screenings conveyance piping, grit vortexes and classifiers, and the screenings/grit hoppers. The hoppers will be located directly above two of the four truck-loading bays. The elevation of the second floor will coincide with the top of the grit removal equipment.

Figures 13 through 21 in **Appendix D** include Process Flow Diagrams (PFDs), floor plans, and building sections for the facility. The figures are preliminary in nature. The elevation of the building and the configuration of the equipment will need to be evaluated in more detail during the design period to ensure that the mechanical and electrical equipment is protected against flood impacts.

Flows will enter the building through the basement via the two 48-inch force mains and will be pumped up to the fine screen header located between the first and second floor. Wastewater will then flow by gravity through the fine screening channels and fine screens. Once the flow has been screened, the flow will reconvene in the grit separator header, which will split the flow amongst the in-service grit separator influent channels. Six modular stacked tray vortex grit removal separators will be constructed in a rectangular configuration to allow flows to enter the units and discharge to two separate troughs leading to the two 72-inch diameter gravity lines. Screened and de-gritted flows will then leave the preliminary treatment building via the two 72-inch diameter gravity lines located in the basement of the building to be conveyed to the primary sludge gallery located to the south of the preliminary treatment building.

National Fire Protection Association (NFPA) guidelines (that is, NFPA 820) for fire and explosion protection will be incorporated into the design of the building. NFPA 820 requires portable fire extinguishers, a combustible gas detection system, and an external hose system for fine screening and grit removal areas within the building. The building areas such as the electrical room, control room, and pipe gallery will be ventilated at a minimum of 6 air changes per hour (ACH) to de-rate the areas to Division 2 spaces. Since the truck bays will have open solids containers (that is, truck roll-off dumpsters), the two duty truck bays will be ventilated at 12 ACH. All channels and equipment will be covered, and the area inside the covered spaces will be ventilated to maintain a negative pressure relative to adjoining spaces. These areas will be designated as Division 1 space.

Odorous air from the facility will be conveyed to the existing odor control system. The existing odor control facility consists of three packed towers with enough room for one additional packed tower scrubber. Each tower is capable of each treating 57,000 cubic feet per minute of odorous air. A fourth packed tower will be constructed as part of the Phase 2 plant improvements as the existing system is currently operating at capacity.

### 4.3.3 Flow Measurement

Existing flow meters measure flows from the Boulevard and East Street Pump Stations and flows from the main influent wastewater pumping station that conveys gravity sewer flows to the ESWPAF. However, additional flow-metering is recommended at the plant to improve accuracy and facilitate operations.

Because of size and space requirements, it is impractical to use Parshall flumes for flow metering in the new preliminary treatment building. Magnetic or doppler radar/ultrasonic-pulse flow meters (FLO-DAR) will be used for flow measurement. There may be a need for 3 to 5 meters of flow span to accurately measure flows across the entire potential range of flows. By using multiple meters, requirements for pipe lengths will be reduced.

Flow metering equipment will be located in the piping, leading to the preliminary treatment building because this piping provides opportunities for high flow velocities, which may reduce the amount of equipment needed. The meters will be located on straight lengths of pipe located in a pipe gallery in the basement of the preliminary treatment building, ideally before plant recycles enter the flow.

The two 48-inch influent pipes will have a cross connecting pipe between the two vertical portions of both 48-inch influent pipes located on the first floor of the pipe gallery. The connecting pipe will have two additional 48-inch vertical pipes leading to the fine screening header located above between the first and second floor. Isolation valves will be installed between the four vertical 48-inch pipes along the cross-connecting pipe to allow isolation of one of the 48-inch influent pipes.

## 4.3.4 Fine Screening

The fine screening process consists of four screening trains. Each train is nominally rated at 62.5 mgd. Two trains, one duty and one standby, are required for dry weather flows, and three duty and one standby train are required for wet weather flows of up to 187 mgd. A passive overflow provides the ability for 60 mgd, the dry weather flow, to bypass the screens and go directly to grit removal in the event that all screening units fail.

The design criteria for the fine screening unit process are summarized in **Table 4-18**.

**Table 4-18. Fine Screening Design Criteria**

| Fine Screening Design Criteria | Value |
|---|---|
| Number of Units | 4 (3 + 1 Standby at Peak Flows) |
| Peak Flow | 62.5 mgd per train |
| Technology | Self-cleaning moving media channel screen |
| Average screening volume (wet) | 30 cubic feet/mgd |
| Wet weather peaking factor | 10 |
| Channel Width | 7.5 feet |
| Screening Efficiency | 70% |
| Tilt | Yes |

### 4.3.4.1 Fine Screen Channels

The four screening trains will be fed from a common influent channel and flow to a common effluent channel. The common effluent channel will be fitted with stop log grooves to allow isolation of one side of the fine screening tray. The screening channels will have motorized sluice gates at the inlet and outlet ends to isolate each of the screens and screen channels for maintenance. The channels will also be furnished with stop log grooves to isolate flow in the case that the sluice gates need to be taken out of service for maintenance.

The screen channel width is 7.5-feet wide, and the minimum approach channel length is three times the width of the screens, or 22.5 feet. The velocity in the approach channel will vary with the plant flow rate. To re-suspend any grit that may have settled in the channels during periods of low flow, high-pressure jets will be installed in the bottom of the channels to allow the channels to be flushed periodically with non-potable plant water. The flushing system will be sized to flush one channel at a time. **Table 4-19** summarizes the screen channel design criteria.

**Table 4-19. Fine Screening Channel Design Criteria**

| Fine Screening Design Criteria | Value |
|---|---|
| Channel width | 7.5 feet |
| Channel depth | 9.5 feet |
| Channel recess for unit | 9 inches |
| Minimum approach distance | 22.5 feet |
| Channel flushing | Water jets (Non-potable Plant Water) |

#### 4.3.4.2 Fine Screens and Screenings Grinder/Washer/Compactors

Each screening train will include a continuously-cleaned fine screen with perforated plates and a combination screenings grinder/washer/compactor. The grinder/washer/compactor units will grind the screenings and flush the screenings with water to remove soluble organics. The screenings will then be mechanically compressed and conveyed to transfer screws. Wash water will drain by gravity back into the screenings channels, and the washed screenings will be compressed to 50 percent of their wet volume before being extruded through a friction cylinder to transfer screws. Screenings will fall by gravity into the transfer screws, which will convey compressed screenings to the truck loading bay. The washed and dewatered screenings material is expected to meet the municipal solid waste classification. Each wet weather screen has a dedicated transfer screw. **Table 4-20** summarizes the screens and screen washer/compactor design criteria.

**Table 4-20. Fine Screens and Washer/Compactor Design Criteria**

| Fine Screening Design Criteria | Value |
|---|---|
| Number of units | 4 |
| Flow | 62.5 mgd each |
| Type | Continuous self-cleaning |
| Opening size | 0.25 inch perforated opening |
| Angle of inclination | 75 degrees |
| Head loss | 24 inches (based on 30% blinded screen) |
| **Screenings Grinder/ Washer/Compactors** | **Value** |
| Number of units | 4 |
| Capacity | 1.96 cy/hr (wet) |
| Type | Integrated grinder/washer/compactor units with screw conveyors |
| Volume reduction | 50% |

cy/hr = cubic yards per hour

#### 4.3.4.3 Fine Screenings – Truck Loading

A total of four truck-loading bays, located in two truck-loading garages, will be provided. Two truck-loading bays will be located on the eastern or northern side of the building in the first truck-loading garage, and two will be located on the western or southern side of the building in the other truck-loading garage. In each truck-loading garage, one truck-loading bay will be used for screenings and the other will be used for grit during normal operation. An enclosed storage hopper will be located above each truck-loading bay.

The function of the enclosed storage hoppers is to contain odors until the time a truck is loaded. The enclosed hoppers are mounted on load cells to allow for weight calculations for storage and load-out of the trucks. A single leveling screw will be installed inside each storage hopper. The leveling screw inside the enclosed storage hoppers provides a means of filling each hopper at the appropriate rate and interval.

**Table 4-21** summarizes the truck loading design criteria.

**Table 4-21. Truck Loading Design Criteria**

| Leveling Screws | Value |
|---|---|
| Number of units | 4 |
| Capacity | 7 cy/hr |
| Type | Reversing screw |
| Number of drops | 3 per screw |
| **Enclosed Storage Hoppers** | **Value** |
| Number of units | 4 |
| Capacity | 15 cy each |
| Type | Enclosed rectangular bottom opening gate |

cy = cubic yard(s)

### 4.3.5 Grit Removal

The grit removal train process will consist of five duty modular stacked tray vortex grit separators and one standby, for a total of six modular stacked tray vortex grit separator units. Each modular stacked tray vortex grit separator will include 12, 12-foot stacked trays. Each unit will have a hydraulic capacity of 46.1 mgd. During wet weather events, the five duty units must be in service to treat a forward flow of 187 mgd plus recycles, allowing one standby unit to be offline.

No special provisions have been included to bypass flow around the grit removal units because one unit will remain on standby and there is no equipment failure that would prevent forward flow.

Each grit removal train will consist of a modular stacked tray vortex grit separator, a pair of grit pumps, a grit cyclone, and a classifier. The grit storage and truck loading system will be separate from that is the fine screenings unit process.

The general design criteria for the grit removal unit process are summarized in **Table 4-22**.

**Table 4-22. Grit Removal Design Criteria**

| Grit Removal | Value |
|---|---|
| Design flow (per unit) | 31.2 mgd |
| Peak hydraulic capacity | 46.1 mgd |
| Average grit loading | 5.8 cubic feet/mgd |
| Wet weather peaking factor | 6 |
| Average daily grit volume | 22 cy |

#### 4.3.5.1 Modular Stacked Tray Vortex Grit Separators

The modular stacked tray vortex grit separator trains will be fed from a common influent channel and flow to a common effluent channel. All channels will have motorized sluice gates at the inlet and outlet of the channel in order to isolate each of the head cell grit separators for maintenance. Additionally, the channels will be furnished with stop log grooves to isolate flow in case the sluice gates need to be taken out of service for maintenance.

The channel width is 4-feet wide and the minimum approach channel length is 20 feet. The velocity in the approach channel will vary with the plant flow rate. To suspend any grit that has settled in the channels, high-pressure jets will be installed in the bottom of the channels and the channels will be flushed periodically. The flushing system will be sized to flush one channel at a time. To flush out any grit that has settled in grit pump discharge lines, a W3 flush will be included to flush grit out of discharge lines so it does not settle and compact in vertical pipes.

**Table 4-23** summarizes the modular stacked tray vortex grit removal design criteria.

**Table 4-23. Modular Stacked Tray Vortex Grit Separator Design Criteria**

| Modular Stacked Tray Vortex Grit Separator | Value |
|---|---|
| Number of units | 6 |
| Channel width | 4 |
| Channel depth | 5 |
| Minimum approach distance | 20 |

### 4.3.5.2 Grit Handling and Separation

Flooded suction recessed impeller pumps will be used to pump the grit slurry out of the bottom of the grit separators and into the grit cyclones and classifiers. Recessed-impeller pumps are often used for this application because of the mechanical wear caused by the grit. To provide redundancy, a duty and standby pump will be provided for each grit separator. Grit piping will be furnished with back flushing capability.

The six grit cyclone classifiers will be located above the truck bay in the screenings and grit loading room. Each pair of grit pumps will discharge into one of six discharge pipes. Each pipe will go to an individual grit cyclone. The grit cyclones will drop directly into one of six grit classifiers located above the hoppers. Three classifiers will drop into one hopper, and the other three classifiers will drop into the other hopper. When full, both hoppers will discharge into truck roll-off containers located below in the truck-loading bays.

**Table 4-24** summarizes the grit slurry pumps design criteria.

**Table 4-24. Grit Slurry Pumps Design Criteria**

| Grit Slurry Pumps | Value |
|---|---|
| Number of units | 12 (two per separator) |
| Type of pump | Recessed impeller or progressive cavity |

**Table 4-25** summarizes the grit cyclone and classifier design criteria.

**Table 4-25. Grit Cyclone and Classifier Design Criteria**

| Grit Cyclone and Classifier | Value |
|---|---|
| Number of units | 6 |
| Performance Criteria | 95% removal of organics |
| Capacity of cyclone | Same as grit pumps |

### 4.3.5.3 Grit - Truck Loading

There will be a total of four truck-loading bays located between two truck-loading garages. Two truck-loading bays will be located on the eastern or northern side of the building in the first truck-loading garage, and two will be located on the western or southern side of the building in the other truck-loading garage. In each truck loading garage, one truck loading bay will be used for screenings and the second will be used for grit during normal operation. Above each truck loading bay will be one enclosed storage hopper. Inside each storage hopper will be a single leveling screw. The leveling screw inside the enclosed storage hoppers provides a means of filling each hopper at the appropriate rate and interval.

The grit classifiers from three of the grit trains will discharge directly into the hopper above one truck bay and the other three grit classifiers will discharge directly into the hopper above the other truck bay. By depositing directly into hoppers, there will be no need for conveying the grit, which is extremely hard on equipment.

The function of the enclosed storage hoppers is to contain odors until the time a truck is loaded. The enclosed hoppers will be mounted on load cells to allow weigh calculations for storage and load-out of trucks.

## 4.4 Primary Clarifier Flow Splitting

### 4.4.1 Existing Conditions

Raw influent wastewater flow enters the plant through a 42-inch gravity sewer, a 54-inch gravity sewer, and a 48-inch force main. The 42-inch gravity sewer directly enters the existing inlet works building for screening and grit removal, then flows by gravity to the existing pump station in the main building. Flow from the 48-inch force main discharges directly into the existing pump station in the main building. The combined flow is pumped into the inlet trough of the three existing primary clarifiers where it joins with flow from the west side pump stations (Boulevard and East Street). The three primary clarifiers have a combined capacity of 150 mgd with all units on line.

### 4.4.2 Flow Splitting

The approach to flow-splitting of PE presented herein minimizes the overall foot print of the upgraded facilities and is consistent with the approach developed for the previous LTCP update. However, the proposed facilities will be difficult to construct and will require a very detailed and well-conceived construction sequence. Alternatives to the current approach, such as construction of a stand-alone pipe gallery to the east of the clarifiers should be investigated during the early phases of design.

A new fourth primary clarifier will be constructed east of the existing primary clarifiers. For the purposes of this LTCP update, all four primary clarifiers will be shortened to a length of 242 feet long. Modifying the length of the existing primary clarifiers allows for the construction of a new primary sludge gallery at the head end of the facility.

Effluent from the preliminary treatment building will flow through two 72-inch pipes to two flow splitter headers located adjacent to the primary sludge gallery. Each header will consist of a 72-inch by 42-inch by 42-inch tee. The 72-inch pipes will transition to two 42-inch pipes and each 42-inch pipe will flow to a dedicated primary influent channel connected to their respective primary clarifiers. Each 42-inch pipe will be fitted with an isolation valve, a flow meter, and a control valve before the influent channel. Gates could also be used to split flow. However, they are generally not as accurate, take up more space, and make it more difficult to address odor control issues.

The primary sludge gallery will connect to the basement of the main building through Stair No. 7 and will house the primary sludge pumps, primary influent piping, flowmeters, isolation valves, and flow control valves needed to split flow equally among the four clarifiers. Flow metering will be performed using magnetic flowmeters.

The primary influent will feed the primary clarifier influent channel from the bottom of the channel. Three isolation gates will be provided in the primary influent channel to facilitate clarifier operation and maintenance. Figures 22 through 29 in **Appendix D** include plan views and sections of the modified primary clarifiers, the new primary sludge gallery, and the flow-splitting piping previously described.

**Figure 4-4** is an aerial view of the primary clarification facility showing the location of the pipe gallery and the fourth primary clarifier as described herein. As noted above, alternative approaches should also be investigated during the design phase. The primary clarifier upgrades are discussed in more detail in the next section of this report.

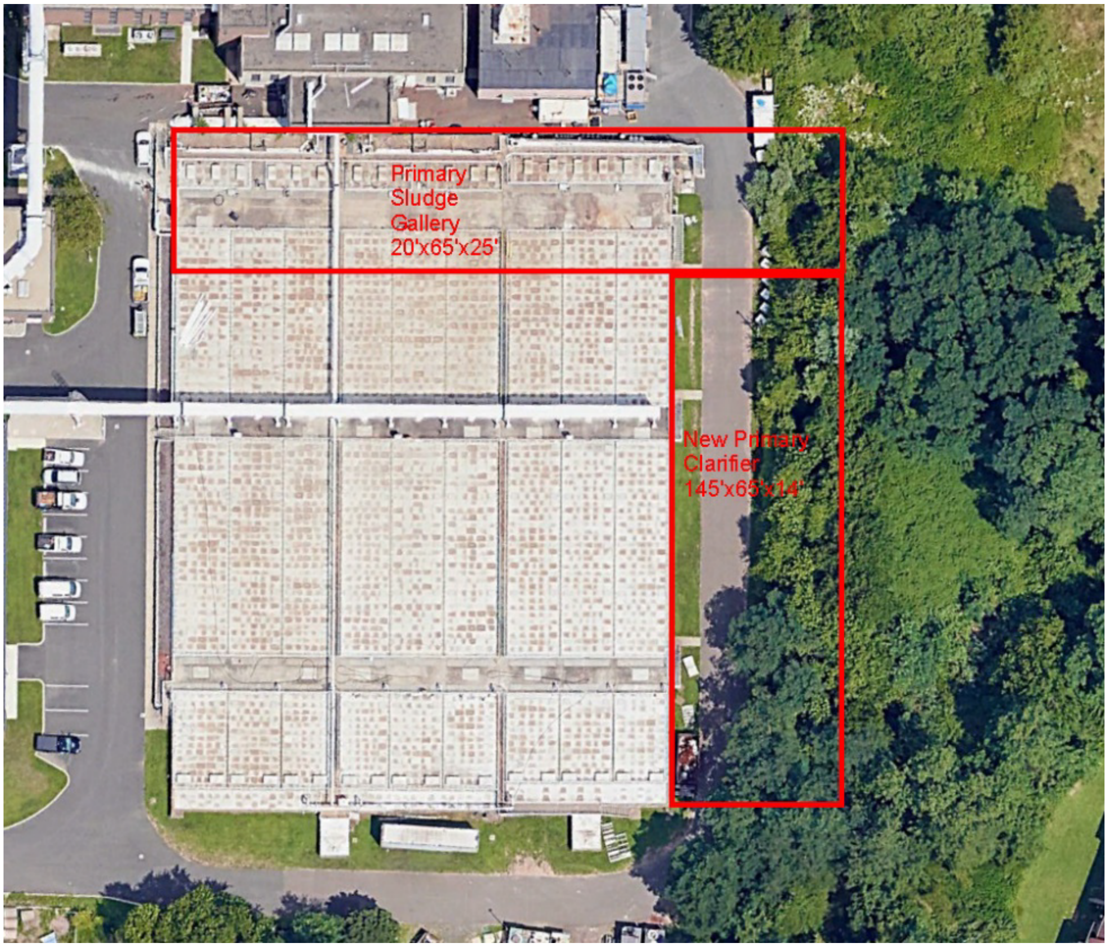



**Figure 4-4. Plan View of Modified Primary Clarifiers and New Primary Sludge Gallery**

## 4.5 Primary Treatment

The current primary clarifier facility consists of three rectangular primary clarifiers. Each primary clarifier is 265 feet in length (inclusive of the distribution channel), 65 feet in width, and operates with a side water depth (SWD) of 11 feet. While the plant can operate with only one clarifier in service during dry-weather conditions, all three primary clarifiers are maintained ready and typically are on line during both dry weather and wet weather. Thus all three clarifiers are usually in service, but any one can be taken off line for maintenance.

### 4.5.1 Historical Primary Clarifier Performance

#### 4.5.1.1 Historical Primary Effluent Flows and Loads

The primary clarifiers are located south of the existing main building. Wastewater enters the primary clarifiers and flows through each tank under quiescent conditions. The clarifiers act as separators, in which the waste solids settle to the bottom of the tank and the scum and grease rise to the water surface. The flights of the longitudinal collectors move slowly along the bottom and along the water surface of each tank to direct the settled solids into sludge troughs at the head end and the floating scum and grease into skimmers near the effluent end of each tank. Primary sludge is pumped to gravity thickening, and the primary scum is routed to the scum pit where it is combined with secondary scum before further processing.

Historical operations and lab data for the ESWPAF were obtained from plant staff and spanned a period from January 1, 2017 to December 31, 2021. An analysis was performed on data collected at the primary influent sample collection point, which includes plant recycles as well as raw wastewater influent. Therefore, the current flows and loads analysis is based on primary influent with recycles. **Table 4-26** summarizes the primary influent flows and loads for the analysis period.

**Table 4-26. Summary of Historical Primary Clarifier Flows and Loads**

| Parameter | Primary Flows | | Primary Influent | | Primary Effluent | |
|---|---|---|---|---|---|---|
| | Influent Flow (mgd) | Effluent Flow (mgd) | TSS (PPD) | BOD-5 (PPD) | TSS (PPD) | BOD-5 (PPD) |
| AA | 29.6 | 28.2 | 84,300 | 93,950 | 19,788 | 47,425 |
| MM | 40.7 | 39.0 | 138,550 | 138,450 | 32,395 | 62,442 |
| MW | 47.8 | 46.5 | 176,050 | 161,500 | 57,139 | 76,409 |
| MD | 81.0 | 79.6 | 267,000 | 232,150 | 90,020 | 87,586 |
| PH | 110.6 | N/A | N/A | N/A | N/A | N/A |

Notes:
Flow and loading data ranges from 2017-2021. Peak hour flows are typically measured at the plant effluent at the ESWPAF. Value shown is estimated from available primary influent data.

As previously established in the *Wet Weather Capacity Improvements and Nitrogen Reduction at the East Shore Water Pollution Abatement Facility* report (CH2M 2011c), dry weather flow conditions were defined as flows less than 60 mgd (taking small rain events into account) and wet weather flows were defined as flows in excess of 60 mgd, with a peak wet weather flow of 187 mgd. The currently permitted dry-weather design flow is 40 mgd.  However, the plant has applied for a 60-mgd permit to accommodate future and wet weather flows.

#### 4.5.1.1.1 Total Suspended Solids Removal

The TSS removal rates were analyzed during the January 2017 to December 2021 monitoring period and are presented on **Figure 4-5** As presented on **Figure 4-6**, the TSS removal percentage is plotted against the primary influent TSS. The influent TSS versus the TSS removal percent curve is typical of WWTFs and indicates the primary clarifiers at ESWPAF are performing within typical industry expectations. During this monitoring period, the primary clarifier influent average TSS loading was approximately 84,300 PPD. The average TSS removal rate was approximately 77 percent.

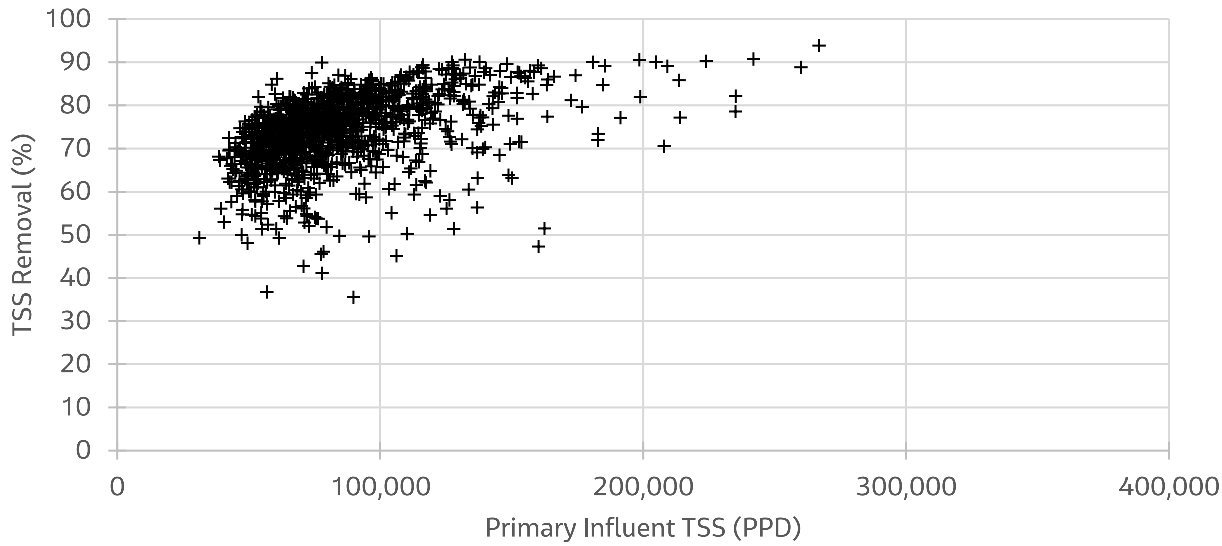



**Figure 4-5. Primary Clarifier Influent TSS vs TSS Removal Percent**

As presented on **Figure 4-6**, the TSS removal percentage is plotted against the surface overflow rate of the primary clarifiers. The surface overflow rate versus the TSS removal percent slope is similarly typical of WWTFs and indicates the primary clarifiers at ESWPAF are performing within typical industry expectations. During this monitoring period, the primary clarifier surface overflow rate average was approximately 862 gallons per day per square foot (gpd/ft$^2$). The MM, MW, and MD loadings were approximately 1,180, 1,392, and 1,567 gpd/ft$^2$, respectively. These values all fall within typical design criteria for primary clarifiers. As shown in the graph, as the surface overflow rate increases, the TSS removal performance typically decreases due to the higher hydraulic and TSS loadings.

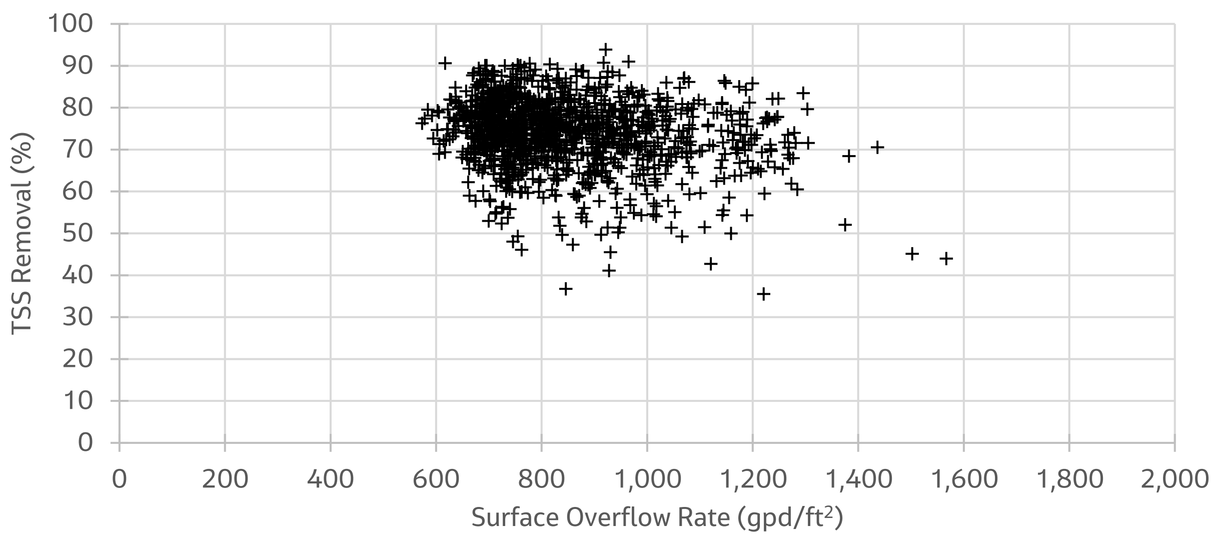



**Figure 4-6. Primary Clarifier Surface Overflow Rate vs TSS Removal Percent**

#### 4.5.1.1.2 Biochemical Oxygen Demand Removal

Biochemical oxygen demand (BOD) removal rates were analyzed during the January 2017 to December 2021 monitoring period, and are presented on **Figure 4-7** As presented on **Figure 4-7**, the BOD removal percentage is plotted against the primary influent BOD concentrations. The influent BOD versus the BOD removal percent curve is typical of WWTFs and indicates the primary clarifiers at ESWPAF are performing within typical industry expectations. During this monitoring period, the primary clarifier influent average BOD loading was approximately 93,966 PPD. The MM, MW, and MD loadings were approximately 138,468, 161,488, and 232,172 PPD, respectively. The average BOD removal rate was approximately 50 percent. The MM, MW, and MD removal rates were 55, 53, and 62 percent, respectively.

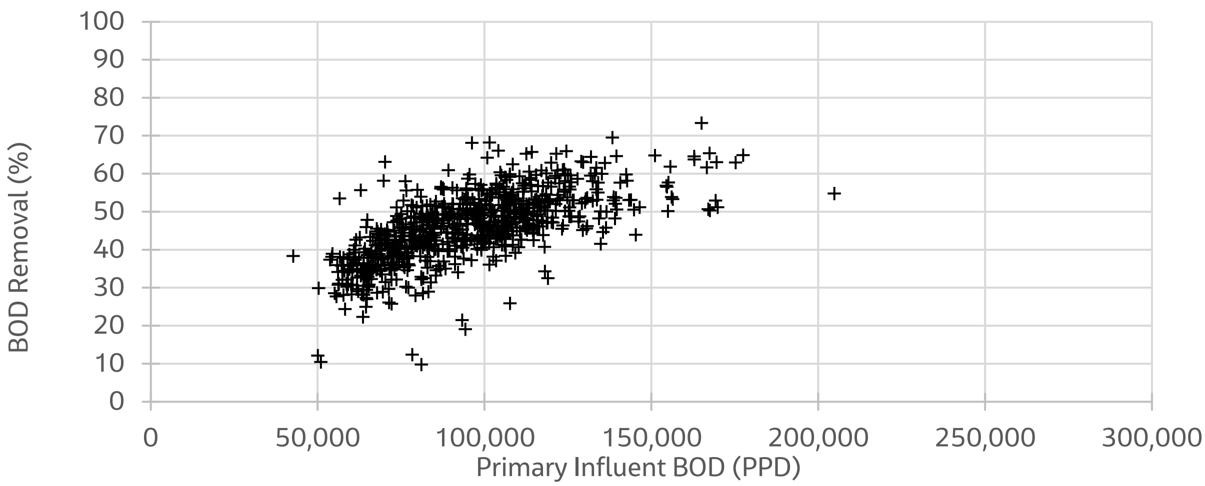



**Figure 4-7. Primary Clarifier Influent BOD vs BOD Removal Percent**

As presented on **Figure 4-8**, the BOD removal percentage is plotted against the surface overflow rate of the primary clarifiers. The surface overflow rate versus the BOD removal performance range is similarly typical of WWTFs. The data indicate that the primary clarifiers at ESWPAF are performing within typical industry expectations. As shown in the graph, typical removal is approximately 50 percent and the surface overflow rate ranges from 600 to 1,000 gpd/ft$^2$.

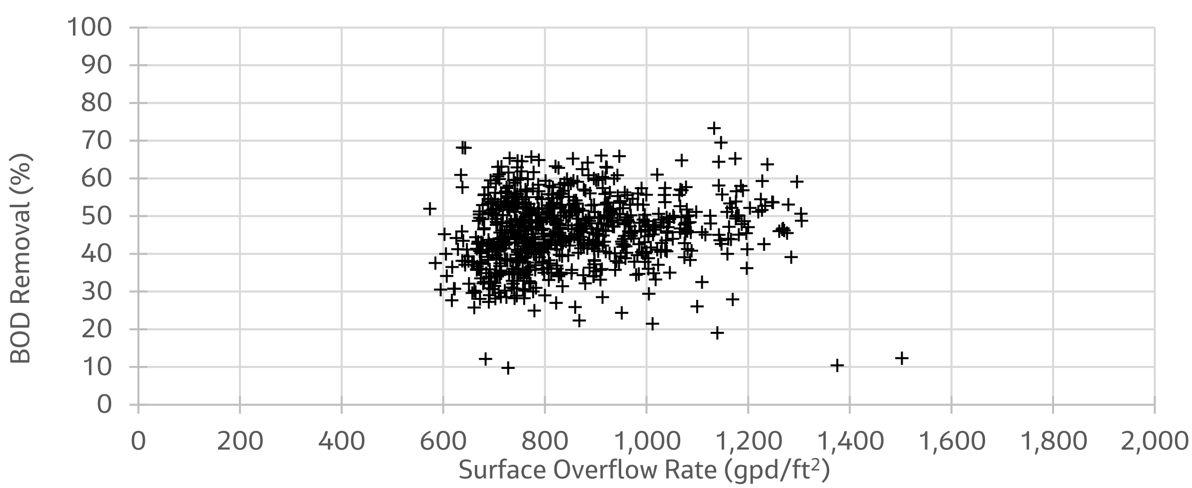



**Figure 4-8. Primary Clarifier Surface Overflow Rate vs BOD Removal Percent**

#### 4.5.1.1.3 Primary Sludge

The primary sludge flows and loads were analyzed during the January 2017 to December 2021 monitoring period. **Figure 4-9** presents the monthly moving average values for the data. The MM primary sludge flow and loading during the monitoring period were approximately 1.85 mgd and 111,000 PPD, respectively.

The average primary sludge flow and loading during the monitoring period was approximately 1.52 mgd and 64,000 PPD, respectively. The MW primary sludge flow and loading during the monitoring period were approximately 1.88 mgd and 140,000 PPD, respectively. The MD primary sludge flow and loading during the monitoring period was approximately 2.15 mgd and 203,000 PPD, respectively.

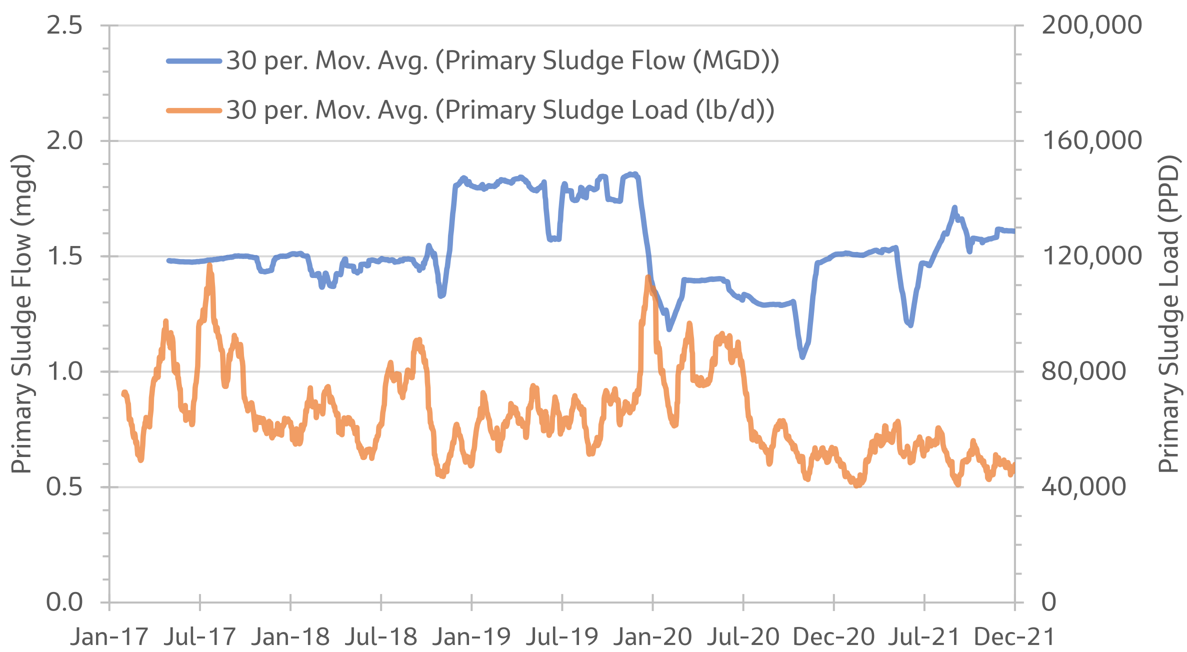



**Figure 4-9. Primary Clarifier Sludge Flows and Loading**

### 4.5.2 Primary Treatment Technology Options

The *Wet Weather Capacity Improvements and Nitrogen Reduction at the East Shore Water Pollution Abatement Facility* report (CH2M 2011c) recommended that a fourth primary clarifier be provided to meet wet-weather requirements at ESWPAF.

Several alternatives were evaluated for this CSO LTCP Update to supplement the capacity of the existing primary clarifiers or (potentially) supplant the requirement for a fourth primary clarifier. These alternatives included chemically-enhanced primary treatment (CEPT), high-rate clarification treatment and new technologies such as Proteus, a proprietary system that utilizes primary effluent filtration to reduce loadings on the downstream biological treatment system. The evaluation was discussed with the GNHWPCA during the early part of the project and will not be presented in detail here.

The implementation of CEPT during wet weather flow events will ultimately reduce solids and organic loadings to the secondary process and can improve nitrification when wet weather flow events are coupled with colder wastewater temperatures. As established in the *Wet Weather Capacity Improvements and Nitrogen Reduction at the East Shore Water Pollution Abatement Facility* report (CH2M 2011c), the implementation of CEPT resulted in improved TSS removal and BOD removal during a jar test analysis. As part of the pump station upgrades for the CSO LTCP, it is anticipated that by 2045 the peak wet weather PH flow will increase from 112 mgd to 187 mgd. **Table 4-27** presents a summary of the primary sludge flows and loads for the existing and projected MD and PH conditions, both with and without the implementation of CEPT.

**Table 4-27. Summary of the Primary Sludge Flows and Loads for the Existing and Projected Conditions**

| | Current (2017-2021) | | | | Projected 2045 | | | |
|---|---|---|---|---|---|---|---|---|
| Flow Condition | AA | MM | MD | PH | AA | MM | MD | PH |
| Primary Flow, mgd | 30 | 41 | 81 | 112 | 33 | 43 | 82 | 187 |
| Clarifiers in Service | 2 | 2 | 3 | 3 | 2 | 3 | 3 | 4 |
| PC Surface Area, $ft^2$ | 34,450 | 34,450 | 51,675 | 51,675 | 31,850[a] | 47,775[a] | 47,775[a] | 63,700[a] |
| Overflow Rate, $gpd/ft^2$ | 864 | 1,180 | 1,567 | 2,167[c] | 1,049 | 900[b] | 1,716 | 2,936[c] |
| TSS Removal, % | 76 | 75 | 66 | 75 | 76 | 75 | 66 | 75 |
| Primary Sludge, PPD | 64,000 | 104,000 | 177,000 | 277,000 | 67,000 | 109,000 | 184,000 | 296,000 |
| Primary Sludge, mgd | 1.5 | 1.6 | 2.1 | 3.0 | 1.5 | 1.7 | 2.5 | 3.2 |
| CEPT - TSS Removal, % | - | - | - | - | - | - | 85 | 85 |
| CEPT - Primary Sludge, PPD | - | - | - | - | - | - | 250,000 | 370,000 |
| CEPT - Primary Sludge, mgd | - | - | - | - | - | - | 2.5 | 3.7 |

[a] Assumes PC length will be reduced by 20 feet for construction of primary sludge gallery.
[b] Recommended Design Surface Overflow Rate $\leq$ 1,200 gpd/ $ft^2$
[c] Recommended PH Design Surface Overflow Rate ≤ 3,000 $gpd/ft^2$.

CEPT was ultimately selected as the only practical alternative for providing the increased primary clarification needed for the ESWPAF in the future. It is the easiest to implement, is the least expensive alternative, has minimal startup complexity, can be operated for short durations during wet weather flow events, has reduced energy and maintenance requirements, and reduced operational complexity. Disadvantages include increased loadings to solids-processing facilities and increased inorganic material, which can negatively impact incineration operations.

### 4.5.3 Primary Treatment Upgrades

To effectively manage the wet weather capacity and loading requirements at ESWPAF, the following Phase 3 improvements to primary treatment are recommended:

- Construction of a fourth primary clarifier treatment train.
- Construction of a new pipe gallery or an extension of the existing pipe gallery to house influent flow-splitting, primary sludge pumping, and related equipment.
- Addition of CEPT to enhance primary treatment process during wet weather flow events
- Rehabilitation of the three existing clarifiers to include installation of new clarifier mechanisms, new longitudinal sludge and scum collectors, new sludge cross collectors, and new scum and grease collection equipment, and new serpentine effluent weirs.
- The elevations of the new effluent weirs and scum collection mechanisms will be raised to minimize the impact of higher harbor elevations.

#### 4.5.3.1 New Primary Clarifier and Primary Sludge Gallery

A fourth primary clarifier will be constructed east of the existing primary clarifiers. The length of all four primary clarifiers will be 245 feet long, inclusive of the distribution channel. The existing clarifiers will be reduced in length by approximately 20 feet in length to allow a new primary sludge gallery to be constructed. The primary sludge gallery will be connecting to the basement of the main building through the existing Stair No. 7. The new primary sludge gallery will house the primary sludge pumps, primary influent piping, and the flowmeters and flow control valves required to split primary influent equally among the clarifiers, as discussed in the previous section of this report. Primary influent will be fed to the primary clarifiers' influent channel from the bottom of the channel, and will include isolation provisions to facilitate clarifier operation and maintenance.

The existing effluent weir and scum-collection system will be demolished and replacement equipment will be set at a higher elevation. The existing PE structure between Clarifiers 1 and 2 will continue to serve those two units. It is currently assumed that the existing effluent structure of Clarifier 3 will be demolished to allow for the construction of a new effluent structure to serve the existing Clarifier 3 and the new Clarifier 4.

The new primary Clarifier 4 will include a new aluminum cover and the existing clarifiers' aluminum covers will be modified to accommodate the reduced clarifier length and equipment. The stairs on the east side of the primary clarifiers will be demolished to allow for the construction of the Clarifier 4 and a new set of stairs will be constructed east of the new Clarifier 4.

**Table 4-28. Process Design Criteria for the Primary Treatment Dry and Wet Weather Flow Conditions**

| Primary Clarifiers | | |
|---|---|---|
| Length (feet) | 245 | |
| Width (feet) | 65 | |
| SWD (feet) | 11 | |
| **Parameter** | **Dry Weather (60 mgd)[a]** | **Wet Weather (187 mgd)[a]** |
| Units in service | 2 | 4 |
| Overflow rate (gpd/ft$^2$) | 1,900 | 2,950 |
| Influent TSS (PPD) | 201,000 | 395,000 |
| Ferric chloride dose (mg/L) | - | 30 |
| Polymer dose (mg/L) | - | 1 |
| TSS removal (%) | 75 | 75 to 90 |

[a] All available PCs are normally kept in operation and can be taken out line for maintenance. The number of units listed is the minimum recommended for the operating conditions listed.

mg/L = milligram(s) per liter

#### 4.5.3.2 Longitudinal Sludge and Scum Collectors

New nonmetallic chain-and-flight sludge and scum collectors with flight monitoring systems will be installed in each of the three treatment cells of each of the four primary clarifiers. To minimize corrosion, the chain drives and flights of the sludge and scum collectors will be fabricated from fiberglass and nylon-based materials. The lower idler shafts of each sludge and scum collector will be installed in an orientation that will minimize chain failure. Two drive mechanisms (one single and one dual) will provide power for the three longitudinal collectors in each of the four primary clarifiers and each drive mechanisms will have mechanical torque limiting devices. Plant staff have noted that optimization of the existing mechanisms has taken several years to achieve. Therefore, "lessons learned" by the plant should be incorporated into the design of the new mechanisms.

#### 4.5.3.3 Sludge Cross Connectors

The existing sludge hoppers in the three existing primary clarifiers tanks will be filled in, and part of the floors of those basins will be demolished to accommodate the new cross collectors. The new Clarifier 4 will also have a sludge hopper. The sludge cross collectors at each primary clarifier will be auger-type screws that convey settled sludge received from the longitudinal collector system to the sludge pumps suction piping. The cross collectors' drive mechanism will have mechanical torque limiting devices.

#### 4.5.3.4 Scum and Grease Collectors and Scum Pit

New scum and grease collectors (scum skimmers) will be provided in each of the three treatment cells of each of the four primary clarifiers. The slotted pipes will increase in diameter from primary Clarifier 4 to primary Clarifier 1, and will be located just upstream of the new effluent weirs. The scum collector at the northwest corner of the primary clarifiers has two chambers and receives, stores, decants, mixes, and pumps primary and secondary scum.

#### 4.5.3.5 Odor Control

The new primary clarifier constructed will include aluminum covers and will tie into the existing odor control treatment system  A more detailed evaluation will need to be conducted to confirm the sizing plant-wide odor control system within of the context of the additional primary clarifier and other future facilities.

#### 4.5.3.6 Primary Sludge Pumps

The new primary sludge pumps will be located in the new primary sludge gallery north of the primary clarifiers, which will ultimately improve the primary sludge pumping through the reduction of the primary sludge suction lines. Three primary sludge pumps will be dedicated to each of the four primary clarifiers, for a total of 12 pumps. The primary sludge pumps will be recessed impeller induced flow centrifugal pumps with AFD motors. One duty and one standby pump are required for dry weather flow conditions while all three pumps at each primary clarifier in service will be required for wet weather peak flows. The primary sludge will be routed to gravity thickening.

**Table 4-29. Process Design Criteria for the Primary Treatment Dry and Wet Weather Flow Conditions**

| Primary Sludge Pumps | | | |
|---|---|---|---|
| Flow at 100% speed (gpm) | 450 | | |
| Horsepower | 30 | | |
| Parameters | Dry Weather (60 mgd) | Wet Weather (187 mgd) | |
| | 75% TSS removal rate | 75% TSS removal rate | 90% TSS removal rate |
| Units in service (continuous operation) | 4 to 6 | 9 to 12 | 9 to 12 |
| Primary sludge concentration (mg/L) | 9,000 | 12,000 | 12,000 |
| Primary sludge load (PPD) | 150,000 | 505,000 | 565,000 |
| Ferric sludge (% of total primary sludge) | - | 7 | 6 |

gpm = gallons per minute

#### 4.5.3.7 Chemically Enhanced Primary Treatment

As originally conceived, the new CEPT facilities would be located at the east side of the existing inlet works building. However, it may be preferable, to locate these facilities at the new Preliminary Treatment Building. The final location will be determined during the design period.

It is anticipated that To implement CEPT at the primary clarifier facility, ferric chloride (38 percent solution) and polymer (30 percent solution will be applied to the effluent from the preliminary treatment building to implement CEPT. Pilot testing before construction of the new primary clarifier is recommended to confirm the recommended chemicals, doses, and expected performance of the system. At least two fiberglass ferric chloride feed tanks, chemical metering pumps, and an emulsion polymer system will also be required.

Chemical sizing for the new CEPT process was calculated on a biweekly basis for the worst case of the following assumptions:

- Two weeks of MW flow (50 mgd)
- Seven days of MD flow (82 mgd)
- Four days with 6-hour rain events of 2-year hourly peak flow (187 mgd)

**Table 4-30** summarizes the resulting equipment requirements.

**Table 4-30. Process Design Criteria for the Primary Treatment**

| Ferric Chloride Storage and Pumping – for Wet Weather CEPT | | |
|---|---|---|
| Number of tanks | 2 | |
| Tank diameter (feet) | 12 | |
| Working volume (gal/tank)[a] | 18,000 | |
| Metering pump capacity (gph) | 0 to 150 | |
| Parameters | MW (50 mgd) | 2-year PH (187 mgd) |
| Number of metering pumps in service | 2 | 4 |
| Polymer Storage and Pumping – for Wet Weather CEPT | | |
| Number of totes | 12 | |
| Tote capacity (gal/each) | 250 | |
| Blending unit: metering pump capacity (gph) | 0 to 15 | |
| Blending unit: dilution capacity (gph) | 0 to 900 | |
| Parameters | MW (50 mgd) | 2-year PH (187 mgd) |
| Number of blending units in service | 1 | 2 |

[a] Weekly ferric chloride delivery for MW flows.
gal = gallon(s)
gph = gallon(s) per hour

## 4.6 Secondary Treatment

Due to projected increases in future loadings and periodic wet weather flows, , increased secondary treatment capacity will be required at the ESWPAF in the future. Improvements that can retrofitted into the existing tankage are preferable because of the design constraints imposed by the existing facilities.

The future influent flows and loads as described in **Section 4.1** are summarized in **Table 4-31**.

**Table 4-31. Summary of 2045 Projected Primary Influent Flows and Loads**

| Condition | 2045 | | | | | | |
|---|---|---|---|---|---|---|---|
| | Flow (mgd) | TSS (PPD) | BOD-5 (PPD) | TN (PPD) | NH3-N (PPD) | TP (PPD) | ORTHO-P (PPD) |
| AA | 33.4 | 87,650 | 97,700 | 9,199 | 5,519 | 3,154 | 1,772 |
| MM Summer | 41.7 | 144,100 | 144,000 | 13,558 | 8,135 | 4,649 | 2,612 |
| MW Summer | 45.7 | 183,050 | 167,950 | 15,813 | 9,488 | 5,422 | 3,047 |
| MD Summer | 74.5 | 277,650 | 184,650 | 17,386 | 10,431 | 5,961 | 3,350 |
| MM Winter | 43.0 | 131,700 | 128,200 | 12,071 | 7,242 | 4,139 | 2,326 |
| MW Winter | 49.9 | 155,200 | 161,550 | 15,211 | 9,126 | 5,215 | 2,931 |
| MD Winter | 82.0 | 244,700 | 241,450 | 22,734 | 13,640 | 7,794 | 4,380 |
| PH | 187.0 | - | - | - | - | - | - |
| MD - Secondary | 60 | - | - | - | - | - | - |

It should be noted that the MM flow and load condition for summer and winter were used to evaluate the secondary system treatment capacity to remove nitrogen. This condition represents the "critical stress" condition with regards to nutrient removal when the ESWPAF will experience the MM BOD-5 and nitrogen loading, as listed in Rows 2 and 5 of **Table 4-31**.

## 4.6.1 Process Modeling

To evaluate existing performance and secondary treatment alternatives to address the need for additional capacity, a calibrated biological process model of the ESWPAF was developed using BioWin 6.2, a process modeling software package.

The process model development and calibration is discussed in **Appendix B**. The calibrated model was used to predict the biological nitrogen removal capability of the ESWPAF and to consider the potential impact of nitrification inhibition on the current process and its capabilities.

## 4.6.2 Review of Historical Nitrogen Removal Performance

The existing waste load allocation (WLA) goal for nitrogen is an annual average effluent loading of 1,568 PPD (572,320 pounds [lbs]/year). Annual limits, such as the WLA, offer a degree of flexibility by allowing for periods of suboptimal nitrogen removal performance during the more-challenging periods of time and periods of good performance that can offset each other, as long as the annual goal is achieved.

Currently, the facility is operated in a MLE configuration from approximately December 1 to May 30 and four-stage Bardenpho from approximately June 1 to November 30. Performance of the facility is summarized in **Table 4-32**. As shown in **Table 4-32**, in 2019 the effluent TN load was 752 PPD higher than the goal WLA. In 2020 and 2021, the facility produced effluent TN loads 558 PPD and 578 PPD lower than the goal WLA.

**Table 4-32. Historical Effluent Total Nitrogen (Annual)**

| Year | Average, PPD | Annual Total, PPD |
|---|---|---|
| Limit Goal WLA | 1,568 | 572,320 |
| 2017 | 780 (-788 lbs) | 284,520 (-287,800 lbs) |
| 2018 | 2,030 (+462 lbs) | 741,350 (+169,030 lbs) |
| 2019 | 2,320 (+752 lbs) | 847,970 (+275,650 lbs) |
| 2020 | 1,010 (-558 lbs) | 368,060 (-204,260 lbs) |
| 2021 | 990 (-578 lbs) | 359,840 (-212,480 lbs) |

Nitrogen removal performance of the facility from January 2017 to December 2021 is presented on **Figure 4-10** and **Figure 4-11**. As presented on **Figure 4-11**, the average effluent TN loading during the monitoring period remained below the TN goal limit (1,568 pounds per nitrogen per day [lbs N/d]) at approximately 1,425 PPD. In the first portion of 2018 and 2019, effluent TN loading exceeded the limit.

The TN excursions can be attributed to the partial loss of nitrification due to wet weather and cold temperatures resulting in high effluent ammonia ($NH_4$-N) concentrations as shown on **Figure 4-10**. Since 2020, the average effluent TN loading has been below the TN goal limit as a result of more stable nitrification as shown on **Figure 4-10** and **Figure 4-11**.

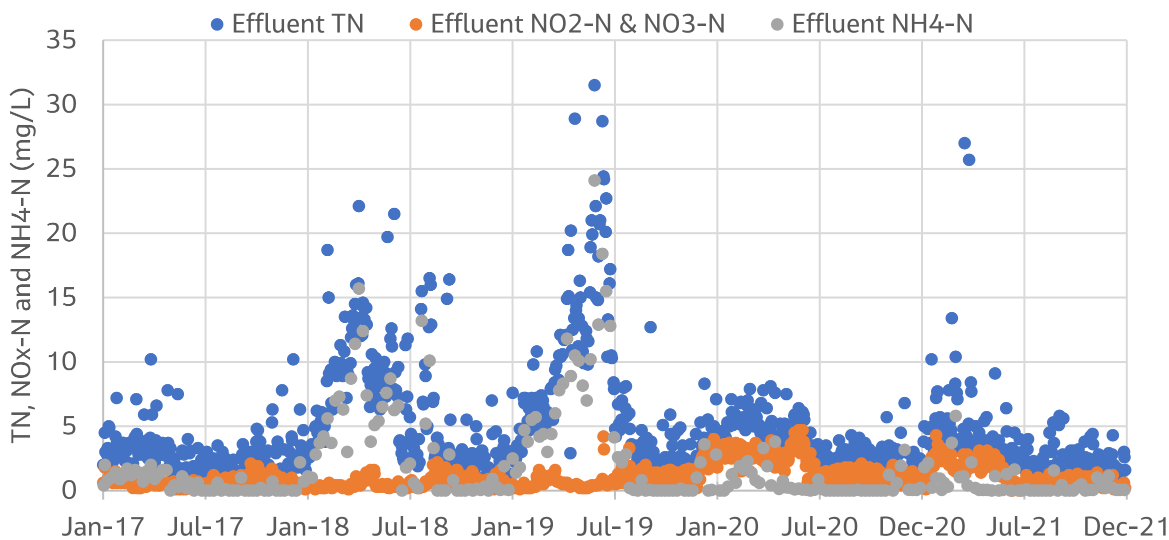



**Figure 4-10. Plant Effluent Daily TN, $NO_2$-N + $NO_3$-N and $NH_4$-N**

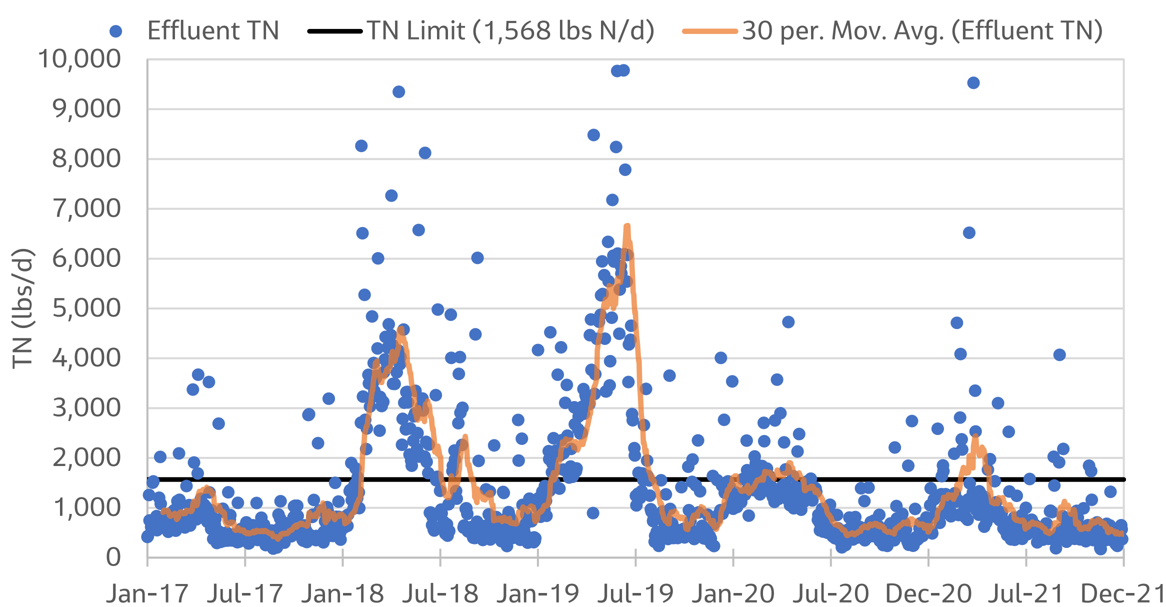



**Figure 4-11. Daily and 30-Day Moving Average TN Effluent Loading of Plant Effluent**

The principal findings from the review of the historical nitrogen removal performance is:

- Historical nitrogen removal performance has been very good. In the early months of 2018 and 2019, effluent TN loading exceeded the performance goal. These excursions can be attributed to the partial loss of nitrification due to wet weather flows, cold water temperatures during wet weather periods, low sludge volume indices (SVIs), and sludge bulking problems.

When improved nitrogen-removal is required in the future, the ESWPAF can utilize the following existing capabilities:

- Operate the facility as a four-stage Bardenpho process to help reduce nitrate levels further during warm-weather operations.
- Provide the capability to further reduce nitrate levels during by adding supplemental carbon into the swing zone or second anoxic zone.
- The recommended future improvements to primary clarification, including the addition of a fourth primary clarifier and CEPT will provide the capability to reduce TSS loadings to secondary treatment and further improve nitrogen removal performance in the future.

Previous evaluations found that the impact of trace amounts of cyanide contained in the recycle flow from incinerator scrubber water can potentially inhibit the growth of nitrifying organisms. The biological modeling evaluation completed for this LTCP Update indicates that nitrification capacity could be reduced by up to 12 percent due to the inhibition. However, the plant has determined that the biological treatment has historically been able fully nitrify year-round despite the effects of inhibition. Going forward, continuing monitoring of scrubber water and other recycles is recommended.

### 4.6.3 Secondary Treatment Capacity

Due to projected increases in future loadings, periodic wet weather flows, and certain limitations of the existing facilities, increased secondary treatment capacity will be required at the ESWPAF in the future. In response to this need, a dynamic process modeling exercise was conducted, utilizing a calibrated BioWin model of the plant, to evaluate the impact of increased loading associated with increased flow to secondary treatment.

In order to evaluate the secondary treatment capacity, dynamic modeling was conducted at various influent flows and loads to determine the maximum TN loading that the existing secondary treatment system can process while meeting the permitted effluent TN of 1,568 lbs N/d. Influent concentrations were the expected average concentrations in 2045. Total of 24 flow conditions were selected to evaluate the flows and loads associated with the current and future annual average, MM and MW conditions as shown in **Table 4-33**.

**Table 4-33. Flows and Loading Conditions for Secondary Treatment Capacity Analysis**

| Condition | Flow (mgd) | TSS (mg/L) | BOD-5 (mg/L) | TKN (mg/L) | TP (mg/L) |
|---|---|---|---|---|---|
| 1 | 18 | 280 | 350 | 30 | 8.3 |
| 2 | 20 | 280 | 350 | 30 | 8.3 |
| 24 | 64 | 280 | 350 | 30 | 8.3 |

Primary clarifier removal was based on the historical data set and were assumed in the range of 62 to 82 percent for TSS, 30 to 40 percent for BOD-5, and 25 to 27 percent for TN as shown on **Figure 4-12**.

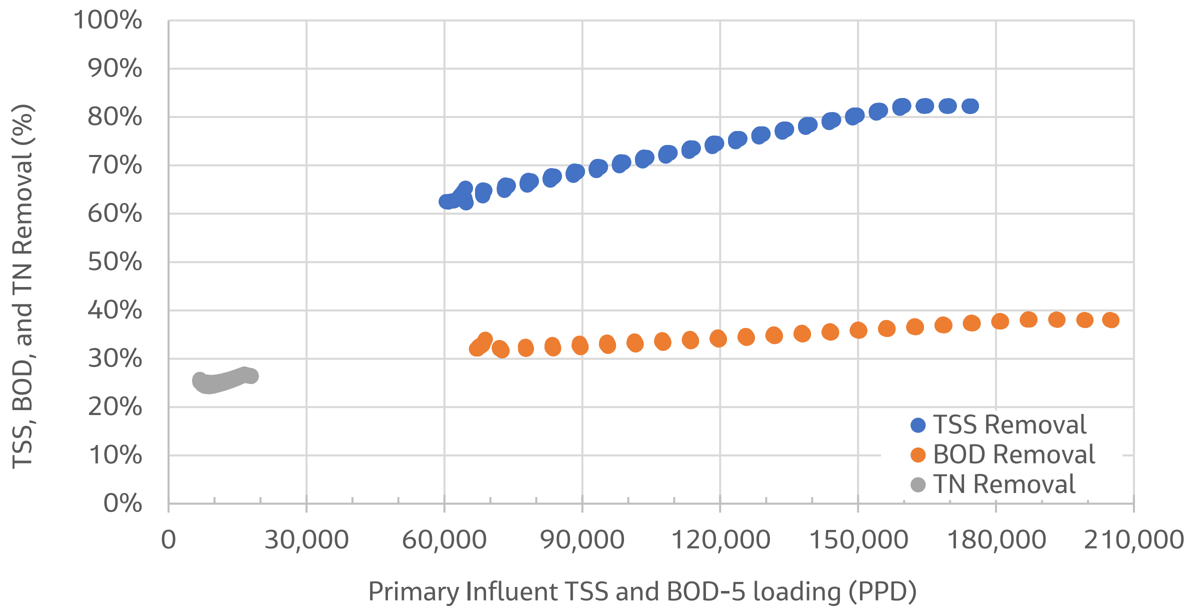



**Figure 4-12. Expected Primary Clarifier TSS, BOD-5, and TN Removal**

The dynamic modeling for the secondary treatment capacity was evaluated in two scenarios without nitrification inhibition.

<u>**Scenario 1**</u>

- Aeration basins: four aeration basins were operated in an MLE process configuration at the water temperature of 13 degrees Celsius (°C) (winter conditions) and four-stage Bardenpho at water temperature of 20°C (summer conditions).
- Internal nitrate-rich recycle (NRCY) flow was maintained at 80 mgd.
- The RAS from the clarifiers was maintained at 60 percent of the influent flow.
- MLE and four-stage Bardenpho process configurations were operated at constant SRT of 8 days and 6 days, respectively.
- No additional carbon source was used.
- Eight secondary clarifiers were in service.

Dynamic modeling results for Scenario 1 are shown on **Figure 4-13** for operation with all aeration basins in service. The modeling was done to determine the maximum TN loading the secondary treatment can handle while meeting the permitted effluent TN of 1,568 lbs N/d. The modeling shows that the maximum nitrogen loading the secondary system can treat in winter (water temperature of 13°C) and Summer (water temperature of 20°C) are approximately 9,000 lbs N/d and 14,000 lbs N/d, respectively. This is attributed to the capacity of the MLE and four-stage processes. MLE configuration can achieve effluent TN between 6 to 12 mg/L while four-stage Bardenpho can achieve effluent TN less than 5.0 mg/L.

**Figure 4-13** shows the expected secondary treatment capacity to remove nitrogen with four aeration basins in service.

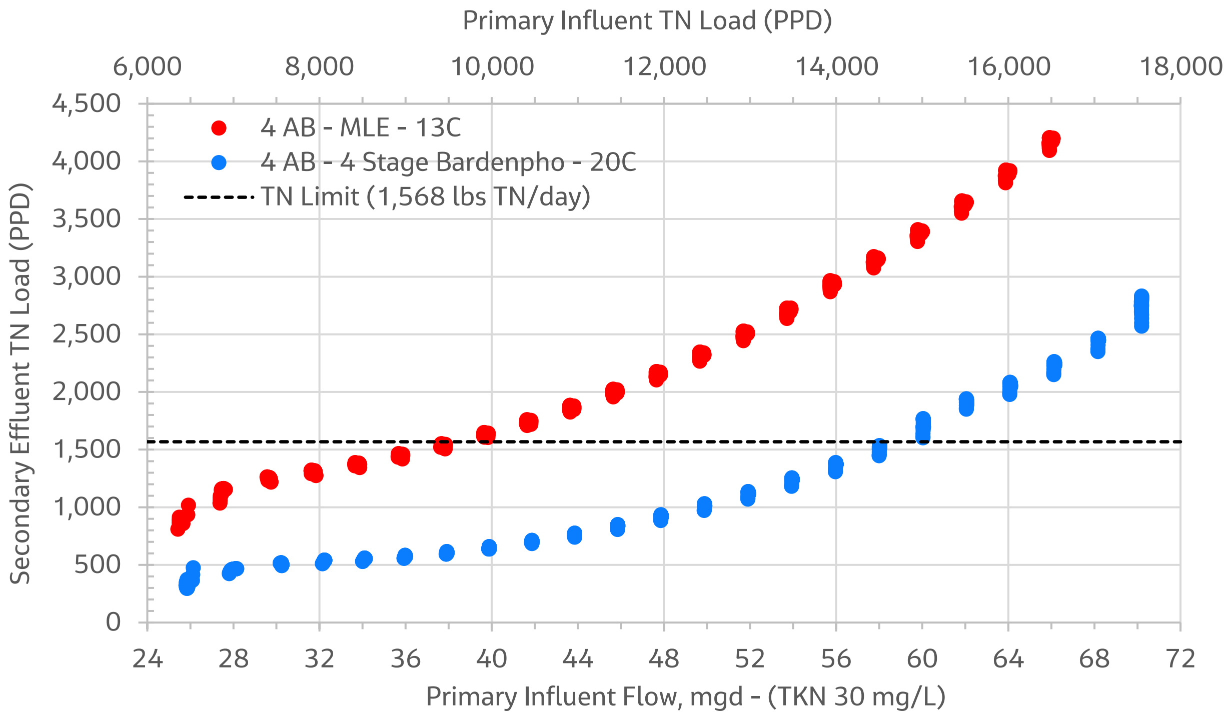



**Figure 4-13. Expected Secondary Treatment Capacity to Remove Nitrogen with Four Aeration Basins in Service**

Dynamic state-point analyses were conducted to determine if the predicted MLSS concentrations can be accommodated by the secondary clarifiers under proposed flow conditions. State-point analysis (SPA) is a graphical technique used for evaluating the performance of secondary clarifiers under peak flow conditions, MLSS concentrations and SVI (milliliters per gram [mL/g]). SVI is a test used to measure the settleability of the mixed liquor. Sludge with good settleability have SVIs values from 75 to 150. The clarifier capacity analysis was developed assuming an SVI value of 150. This value represents historical average sludge settleability observed.

The location of the state-points in relation to the settling flux curve predict the performance of the secondary clarifier. The state-point of a well operated clarifier should be located below the settling flux curve and the underflow rate line operating below the descending limb of the settling flux curve. If the state-points are located above the settling flux curve in any condition, the material will not settle in the clarifier, but will flow out of the clarifier via the effluent weir. Similarly, if the underflow rate operating line is shown above the settling flux curve in any condition, the sludge blanket is projected to rise and exit the clarifier via the effluent weir.

The state-point analyses were done for the eight existing secondary clarifiers (100-feet diameter) for the flow conditions described in **Table 4-33**. **Figure 4-14** shows the dynamic state-point analyses with eight secondary clarifiers in service. In this analysis, it was assumed a RAS rate of 60 percent of the influent flow. Based on the results, it is estimated that the limit capacity of the secondary clarifiers is at approximately 22 pounds per square feet per day (lbs/ft$^2$/day), assuming a MLSS concentration of 4,200 mg/L. The influent flow is approximately 39 mgd with a RAS flow of approximately 23.4 mgd. Flow conditions than result in solids flux rates greater than 22 lbs/ft$^2$/day may exceed the secondary clarifiers capacity to settle the MLSS resulting in effluent TSS higher than of 30 mg/L. The clarifier performance analysis was done assuming an average SVI of 150 mL/g and standard settling solid flux curve.

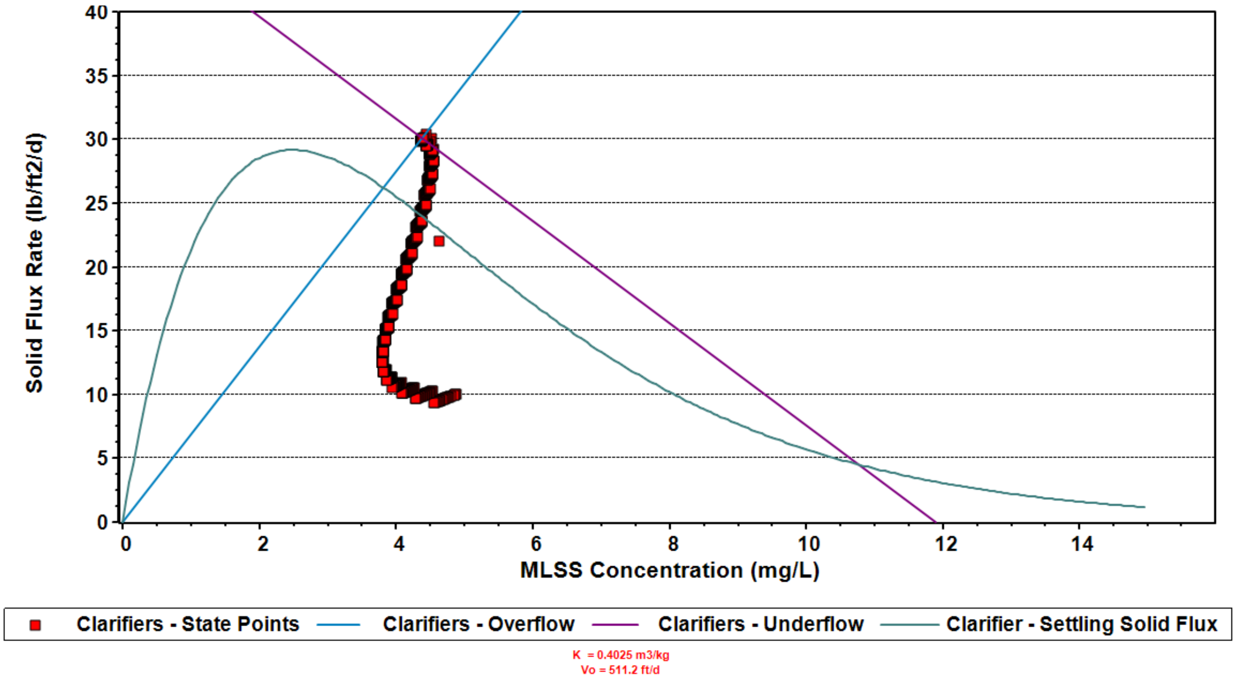



**Figure 4-14. Dynamic State-Point Analyses With Eight Secondary Clarifiers in Service**

<u>**Scenario 2**</u>

- Aeration basins: three aeration basins were operated in an MLE process configuration at the water temperature of 13°C (winter conditions) and four-stage Bardenpho at water temperature of 20°C (summer conditions).
- MLE and four-stage Bardenpho process configurations were operated at constant SRT of 8 day and 6 day, respectively.
- NRCY was maintained at 60 mgd.
- The RAS was maintained at 50 percent of the influent flow.
- Seven secondary clarifiers were in service (one is out-of-service).

Dynamic modeling results for Scenario 2 are shown on **Figure 4-15** for operation with three aeration basins in service. The modeling shows that the maximum nitrogen loading the secondary system can treat in winter (water temperature of 13°C) and Summer (water temperature of 20°C) are approximately 8,000 lbs N/d and 12,000 lbs N/d, respectively. **Figure 4-15** shows the expected secondary treatment capacity to remove nitrogen with three aeration basins in service.

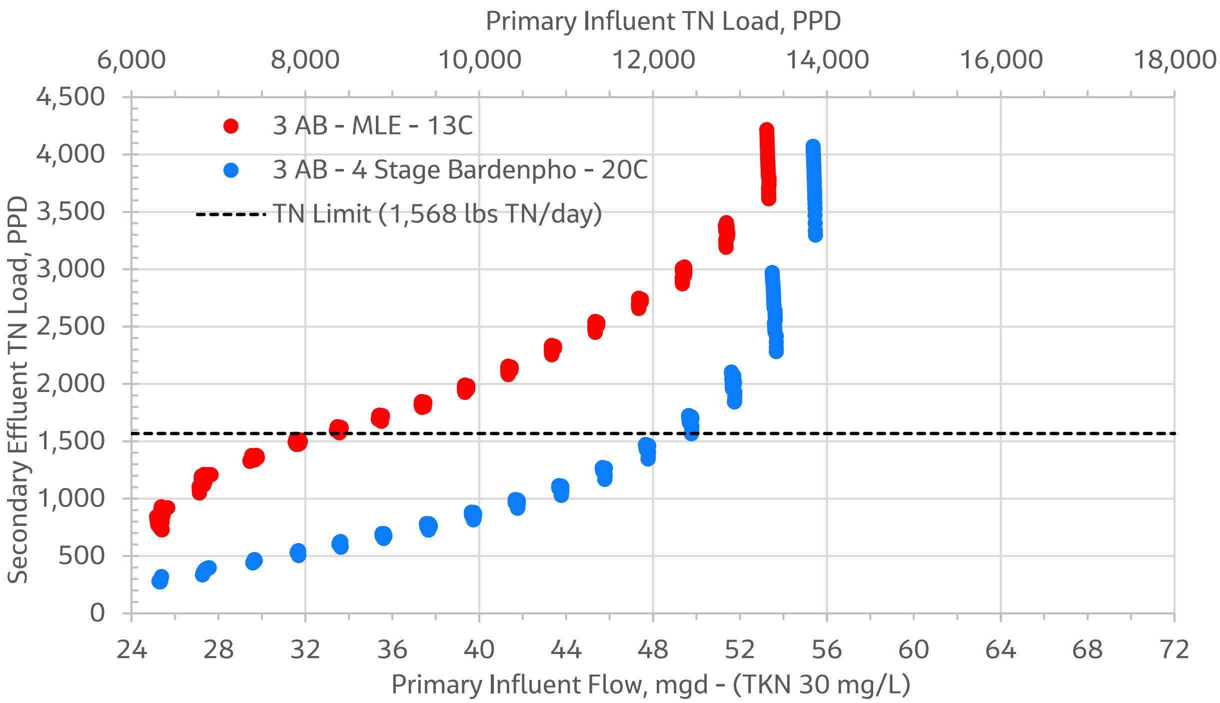



**Figure 4-15. Expected Secondary Treatment Capacity to Remove Nitrogen With Three Aeration Basins in Service**

Dynamic state-point analyses were conducted to determine if the predicted MLSS concentrations while operating with three aeration basins can be accommodated by the secondary clarifiers under proposed flow conditions. The clarifier capacity analysis was developed assuming an SVI value of 150. This value represents historical average sludge settleability observed. The state-point analyses were done with seven secondary clarifiers (100 feet diameter) for the flow conditions described in **Table 4-33**. **Figure 4-16** shows the dynamic state-point analyses with seven secondary clarifiers in service.

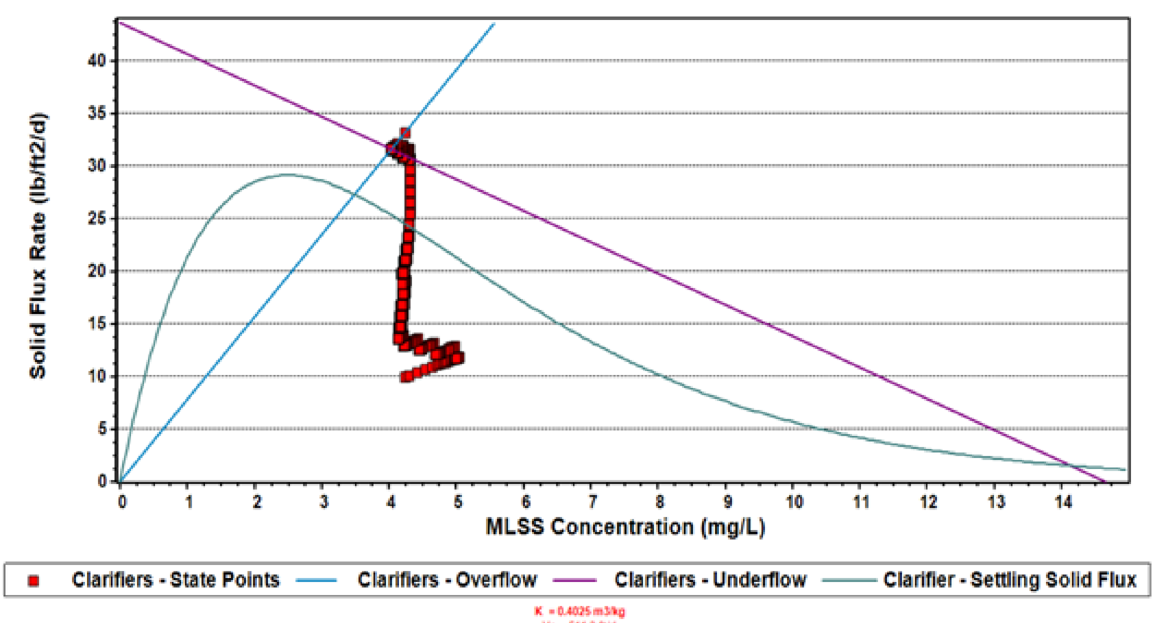



**Figure 4-16. Dynamic State-Point Analyses With Seven Secondary Clarifiers in Service**

In this analysis, it was assumed a RAS rate of 50 percent of the influent flow. Based on the results, it is estimated that the limit capacity of the secondary clarifiers is at approximately 22 $lbs/ft^2/day$ , assuming an MLSS concentration of 4,200 mg/L. The influent flow is approximately 34 mgd with a RAS flow of approximately 17 mgd. Flow conditions than result in solids flux rates greater than 22 $lbs/ft^2/day$ may exceed the secondary clarifiers capacity to settle the MLSS resulting in effluent TSS higher than of 30 mg/L. The clarifier performance analysis was done assuming an average SVI of 150 mL/g and standard settling solid flux curve.

#### 4.6.3.1 Existing Four-stage Bardenpho and MLE Processes

The capacity for these two processes were analyzed at the MM loading conditions for summer and winter for the future influent characteristics is described in **Table 4-34**. For the MLE process in winter, the historical water temperature of 13°C was selected. For the four-stage Bardenpho process in summer, water temperature of 20°C was selected. The maximum treatment capacity of the four-stage Bardenpho and MLE processes is defined as the maximum influent load that can be successfully treated over the course of 30 days without the resulting MLSS levels exceeding 4,200 mg/L (that is, the maximum MLSS level defined in the secondary clarifier analysis). The model results for both processes at MM conditions are presented in **Table 4-34**.

The modeling was done to determine the maximum TN loading the secondary treatment can handle while meeting the permitted effluent TN of 1,568 lbs N/d. The modeling results showed that MLE process configuration produced effluent TN that exceeded the nitrogen goal. This was attributed to the capacity of the MLE and four-stage processes. MLE configuration can achieve effluent TN between 6 to 12 mg/L, while four-stage Bardenpho can achieve effluent TN less than 5.0 mg/L.

**Table 4-34. Model Results for the MLE and Four-stage Bardenpho Processes for Future Maximum Month Conditions**

| Condition | MLE | Four-stage Bardenpho |
|---|---|---|
| | MM Winter 13°C | MM Summer 20°C |
| ***Aeration Basins*** | | |
| No. of Tanks Online | 4 | 4 |
| Anoxic Volume, MG | 1.76 | 1.76 |
| Aerobic Volume, MG | 4.81 | 4.81 |
| Swing/Post Anoxic, MG | 1.92 | 1.92 |
| Re-Aeration, MG | 0.66 | 0.66 |
| Total Volume, MG | 8.15 | 8.15 |
| Aerobic volume, % | 90 | 67 |
| SRT, day | 9 | 8 |
| MLSS, mg/L | 3,870 | 4,150 |
| Internal Mixed Liquor Recycle Flow, mgd | 80 | 80 |
| RAS Flow, mgd | 27.5 | 26.7 |
| Average Dissolved Oxygen, mg/L | 2.0 | 2.0 |
| Air Flow Rate, scfm | 17,570[a] | 33,260[a] |
| ***Sludge Production*** | | |
| Primary Sludge, PPD | 102,200 | 111,600 |
| Thickened Primary Sludge, PPD | 94,800 | 103,400 |
| WAS, PPD | 32,800 | 39,400 |
| Thickened WAS, PPD | 32,100 | 38,600 |
| Total Thickened Sludge, PPD | 126,900 | 142,000 |

| Condition | MLE | Four-stage Bardenpho |
|---|---|---|
| | MM Winter 13°C | MM Summer 20°C |
| ***Secondary Effluent*** | | |
| Flow Rate, mgd | 41.1 | 39.9 |
| BOD-5, mg/L | <10 | <10 |
| TSS, mg/L | <15 | <15 |
| TKN, mg/L | 1.4 | 1.5 |
| Ammonia, mg/L | 0.2 | 0.2 |
| Nitrate+ Nitrite, mg/L | 4.2 | 2.0 |
| TN, mg/L- PPD | 6.4 – 2,194 | 4.0 – 1,330 |

[a] Assumes est. alpha of 0.6 (AA) or 0.55 (MM), fouling of 0.8, beta pf 0.95, and standard oxygen transfer efficiency (SOTE) of approximately 34 percent. Current maximum blower capacity is (6) x 9,500 scfm each, (5) duty and (1) standby.
scfm = standard cubic feet per minute

The state point analysis was done to the eight existing secondary clarifiers (100 feet diameter) for the MM for winter and summer, and MD flows for 2045 as described in **Table 4-34**. At MM-13 °C the design MLSS concentration was 3,870 mg/L with a RAS flow of 27.5 mgd. At MM-20 °C and MD conditions, the design MLSS was 4,150 mg/L with RAS flows of 26.7 mgd and 30 mgd, respectively. The MD flow was 60 mgd. The results of the SPA for the MM and MD conditions are shown on **Figure 4-17**.

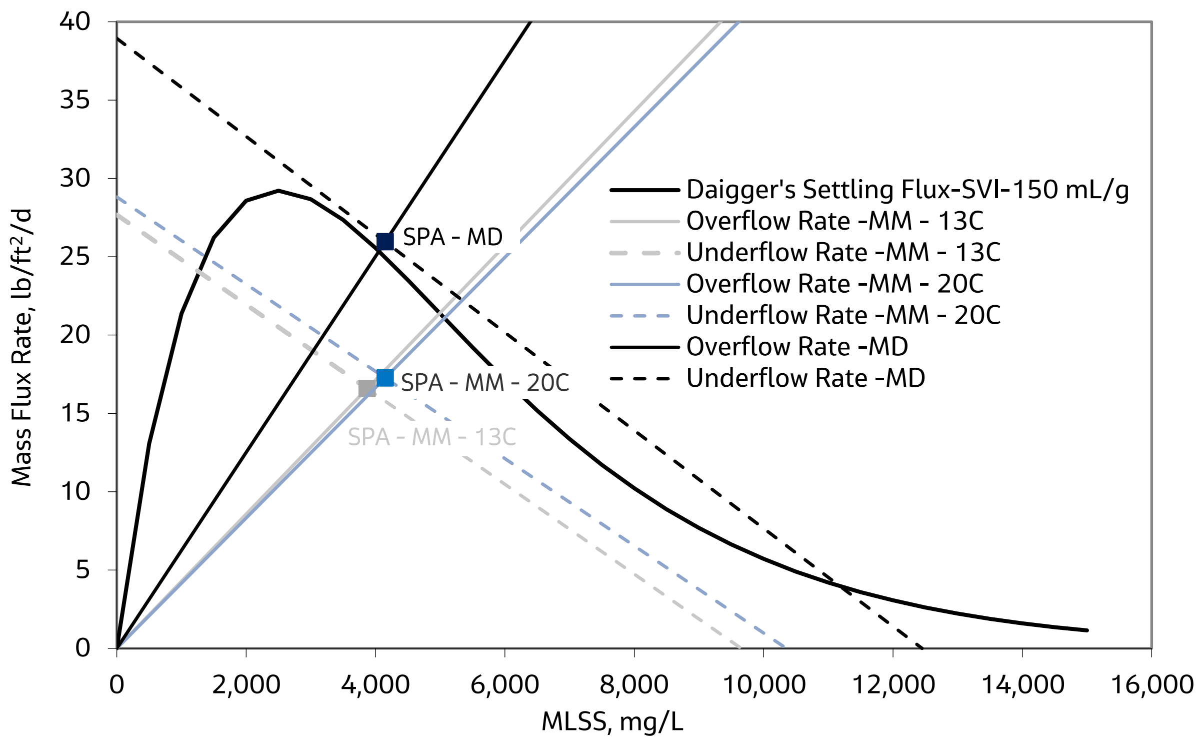



**Figure 4-17. Secondary Clarifiers State-Point Analysis**

As shown on **Figure 4-17**, at MM-13C and MM-20C conditions the secondary clarifiers will not be overloaded. However, at MD condition the secondary clarifier will not have sufficient capacity to settle the MLSS and comply with an effluent TSS limit of 30 mg/L. The clarifier performance analysis was done assuming an average SVI of 150 mL/g.

### 4.6.4 Secondary Treatment Options to Increase Capacity and Performance

The ESWPAF's current nitrogen WLA is based on an annual average daily effluent loading of 1,568 PPD (572,320 lbs/year). This annual limit means that suboptimal nitrogen removal performance during the most challenging period of the year (December through May) can be offset by better-than-average performance in the second half of the year.

Currently, the facility is operated in an MLE configuration from approximately December 1 to May 30, and four-stage Bardenpho from approximately June 1 to November 30. The MLE process configuration achieves nitrification and partial denitrification and typically produces effluent TN that slightly exceeds the treatment goal. The four-stage Bardenpho process achieves nitrification and denitrification to produce effluent TN that meets the treatment goal. The flexibility of the current seasonal operations approach has worked well and was used as the baseline operations mode for the evaluation of all alternatives.

#### 4.6.4.1 Process Improvement Evaluation

Previous studies and facility plans have made a number of recommendations to improve nitrogen removal at the ESWPAF. The following improvements have all been implemented:

- Modifications and improvements to the aeration grids for added nitrification reliability and dissolved oxygen (DO) control
- Supplemental carbon feed system for added denitrification flexibility
- Swing second anoxic zone capability to improve denitrification flexibility
- Increase mixed liquor recycle pumping capacity

Three treatment alternatives were evaluated in this LTCP for potential implementation at the ESWPAF to provide additional capacity in the future or improve nitrogen-removal performance. The three treatment alternatives selected for this evaluation include:

- Sludge Densification with Hydrocyclones
- Mobile Organic Biofilm (MOB)
- Membrane Aerated Biofilm Reactor (MABR)

Of the three options, sludge densification with hydrocyclones is recommended to improve settleability and effluent quality. The MOB and MABR options offer the potential to increase capacity in the future within the limited space available on the plant site. The evaluation of these two options assumes no nitrification inhibition and sludge densification with hydrocyclones to improve the sludge settleability.

#### 4.6.4.2 Hydrocyclones

The modeling evaluation described above indicates that the existing secondary clarifiers are solids limited. Sludge settleability can be improved by using hydrocyclones, a gravimetric selection technology that helps to retain denser biomass while wasting out the light fraction of the MLSS in the aeration basins. Increased density can lead to improved sludge settleability resulting in less solids in the effluent. The units are normally installed in the WAS line.

Hydrocyclones do not increase treatment capacity but do offer a measure protection when solids carryover is of concern. Some pilot tests have indicated that an winter-time decrease in SVI of up to 10 % can be achieved. It is recommended this technology to improve the sludge settleability be pilot tested at the ESWPAF before being implemented at full scale. **Figure 4-18** is a typical hydrocyclone skid used for sludge densification.

**Figure 4-18. Sludge Densification Hydrocyclones**

#### 4.6.4.3 Mobile Organic Biofilm

Mobile organic biofilm (MOB) is process that endeavors to increase the effective biomass in a biological treatment system by adding a mobile biofilm carrier (Kenaf) into the secondary system into the mixed liquor. Kenaf is an organic cellulosic material used as support for biofilm growth and is shown on **Figure 4-19**. This material is machined to approximately 0.5 millimeter (mm) increasing the available area for biofilm adsorption and attachment. Different than conventional plastic carriers, this organic carrier does not require sieve devices to prevent media loss as it can freely circulate throughout the aeration basin zones.

The mobile biofilm carrier adds a fixed-film component to the activated sludge process, and increases the design SRT because of the high surface area of the media. The Kenaf material enhances the settleability of the MLSS and can be returned with the RAS to the aeration basins. A screening system would be required in the WAS line to retain and return the Kenaf media to the aeration basins.

Kenaf has been used in the food processing industry to improve biological nutrient removal and treatment capacity. However, there is currently limited experience with this process in the municipal market and, as a result, it represents a process performance risk. This innovative technology would need to be pilot tested at the ESWPAF or demonstrated in full-scale facilities with a similar biological configuration to ensure that it has no negative impacts or the process (such as solids carryover) before being implemented.

**Figure 4-19. MOB Kenaf Media**

#### 4.6.4.3.1 Model Development

SUMO, a biological-treatment simulation software package developed by Dynamita, was used to model the existing ESWPAF treatment processes. A process model was developed to simulate the performance of the ESWPAF using its recent operational data supplemented with additional wastewater characterization data. The model was used to determine the facility capacity with existing infrastructure to treat future flows and loads. The ESWPAF was operated with two primary clarifiers, four aeration basins, eight secondary clarifiers and one gravity thickener. Note the number of process units were consolidated into one representative unit each for simplicity as shown on **Figure 4-20**. The SUMO models were configured in an MLE and four-stage Bardenpho processes to simulate MM conditions in winter and summer, respectively. Primary clarifier performance was based on the historical data.

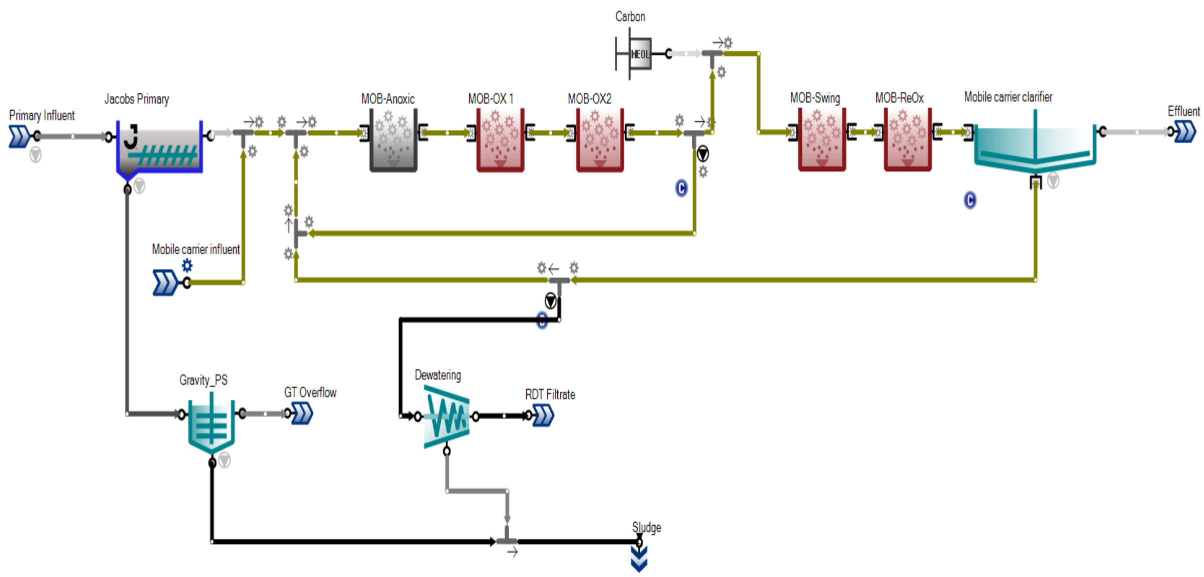



**Figure 4-20. SUMO Process Configuration Model for MOB Technology**

The capacity of the MOB technology for these two processes were analyzed at the MM loading conditions for summer and winter for the future influent characteristics described in **Table 4-34**. For the MLE process in winter, the historical water temperature of 13°C was selected. For the four-stage Bardenpho process in summer, water temperature of 20°C was selected. The maximum treatment capacity of the four-stage Bardenpho and MLE processes is defined as the maximum influent load that can be successfully treated over the course of 30 days without the resulting MLSS levels exceeding 4,200 mg/L (that is, the maximum MLSS level defined in the secondary clarifier analysis). The model results for both processes at MM conditions are presented in **Table 4-35**.

**Table 4-35. SUMO MOB Model Results for the MLE and Four-stage Bardenpho Processes for Future Maximum Month Conditions**

| Condition | MLE MM Winter 13°C | Four-stage Bardenpho MM Summer 20°C |
|---|---|---|
| ***Aeration Basins*** | | |
| No. of Tanks Online | 4 | 4 |
| Anoxic Volume, MG | 1.76 | 1.76 |
| Aerobic Volume, MG | 4.81 | 4.81 |
| Swing/Post Anoxic, MG | 1.92 | 1.92 |
| Re-Aeration, MG | 0.66 | 0.66 |
| Total Volume, MG | 8.15 | 8.15 |
| Aerobic volume, % | 90 | 67 |
| MOB Media Fill Fraction, % | 1.25 | 1.25 |
| Media Specific Surface Area, $m^2/m^3$ | 20,000 | 20,000 |
| Biofilm Thickness, mm | 0.3 | 0.3 |
| Total Surface of Biofilm Carrier, $m^2$ | $8.66\times10^6$ | $8.66\times10^6$ |
| SRT, day | 5.5 | 4.5 |
| MLSS, mg/L | 3,450 | 3,350 |
| Internal Mixed Liquor Recycle Flow, mgd | 80 | 80 |
| RAS Flow, mgd | 27.5 | 26.7 |
| Average DO, mg/L | 3.0 | 3.0 |
| Air Flow Rate, scfm | 40,250[a] | 40,060[a] |
| ***Sludge Production*** | | |
| Primary Sludge, PPD | 101,000 | 111,200 |
| Thickened Primary Sludge, PPD | 90,900 | 100,100 |
| WAS, PPD | 43,200 | 55,400 |
| Thickened WAS, PPD | 41,000 | 52,600 |
| Total Thickened Sludge, PPD | 131,900 | 152,700 |
| ***Secondary Effluent*** | | |
| Flow Rate, mgd | 40.9 | 39.7 |
| BOD-5, mg/L | <10 | <10 |
| TSS, mg/L | <15 | <15 |
| TKN, mg/L | 1.8 | 1.5 |
| Ammonia, mg/L | 0.1 | 0.1 |
| Nitrate+ Nitrite, mg/L | 4.5 | 2.0 |
| TN, mg/L- PPD | 7.0 – 2,390 | 4.5 – 1,490 |

[a] Assumes est. alpha x fouling of 0.4, and SOTE of approximately 29 percent. Current maximum blower capacity is (6) x 9,500 scfm each, (5) duty and (1) standby.
$m^2$ = square meters
$m^3$ = cubic meters

The SUMO modeling was done to determine the maximum TN loading the secondary treatment can handle while meeting the permitted effluent TN of 1,568 lbs N/d. The MOB technology operated at lower SRTs than conventional activated sludge process due to the high surface area of the media. Lower SRTs resulted in lower MLSS in the aeration basins reducing the solids loading in the secondary clarifiers. The MLSS in the aeration basins were approximately 10 percent less than the conventional activated sludge process. This result suggests that the facility could gain approximately 10 percent treatment capacity by implementing MOB technology.

The state point analysis was done to the eight existing secondary clarifiers (100 feet diameter) for the MM for winter and summer, and MD flows for 2045 as described in **Table 4-34**. At MM-13C, the design MLSS concentration was 3,450 mg/L with a RAS flow of 27.5 mgd. At MM-20C and MD conditions, the design MLSS was 3,350 mg/L with RAS flows of 26.7 mgd and 30 mgd, respectively. The MD flow was 60 mgd. A MD condition with a safety factor of 1.2 was also analyzed. At 1.2 x MD condition, the design MLSS of 3,350 mg/L with RAS flow of 36 mgd was also used. The results of the SPA for the MM, MD and 1.2 MD conditions are shown on **Figure 4-21**.

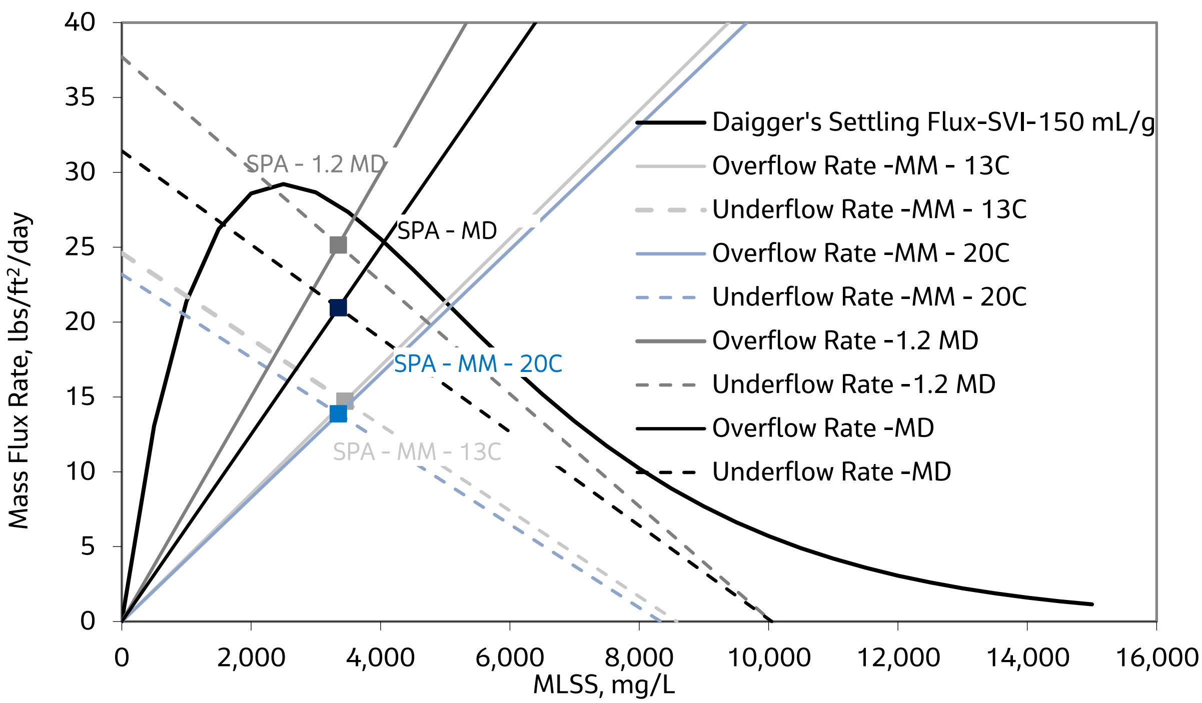



**Figure 4-21. Secondary Clarifiers State-Point Analysis for MOB Technology**

As shown on **Figure 4-21**, at MM-13C, MM-20C, MD and 1.2 MD conditions, the secondary clarifiers will not be overloaded and therefore have sufficient capacity to settle the MLSS and comply with an effluent TSS limit of 30 mg/L. The clarifier performance analysis was done assuming an average SVI of 150 mL/g. However, the MOB technology will increase the sludge settleability producing sludge with lower SVI values.

A proposed process diagram for this alternative is shown on **Figure 4-22**. The major components and improvements needed to implement the MOB technology at the ESWPAF include the following:

- WAS Line:
  - Rotary drum fine screen system
  - Sludge holding tank
  - WAS discharge piping with valves and flow meters
  - WAS return pumping system including pumps, piping, valves and flow meters

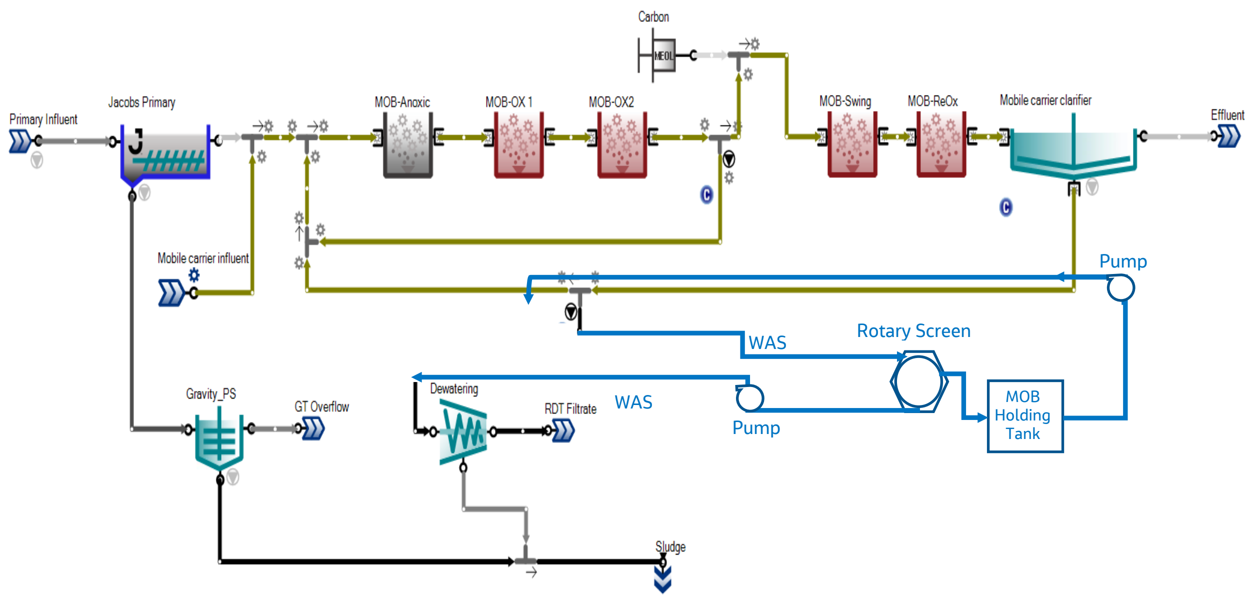



**Figure 4-22. Proposed Process Diagram for the MOB Implementation**

### 4.6.4.4 Membrane Aerated Biofilm Reactor

The membrane aerated biofilm reactor (MABR) alternative utilizes membranes as a support media for biological growth. The organisms develop a biofilm on the membrane surface to increase the biomass inventory in the secondary treatment. Implementation would be accomplished by installing MABR membranes in the first anoxic zone as shown on **Figure 4-23**. The MABR process also delivers oxygen directly to the biofilm attached to the surface of the membranes as shown on **Figure 4-23** (lower right-hand corner). Oxygen is delivered by diffusion to the biofilm with very high efficiency while substrate, such as ammonia and organics, diffuse into the biofilm from the surrounding bulk liquid. Low-pressure air is delivered to the membrane cassettes using typical aeration blowers but at a fraction of the aeration rate required by fine bubble aeration. The MABR is a hybrid process that uses both attached growth bacteria on the membrane and suspended growth bacteria in the bulk solution to remove organics and nutrients from the wastewater.

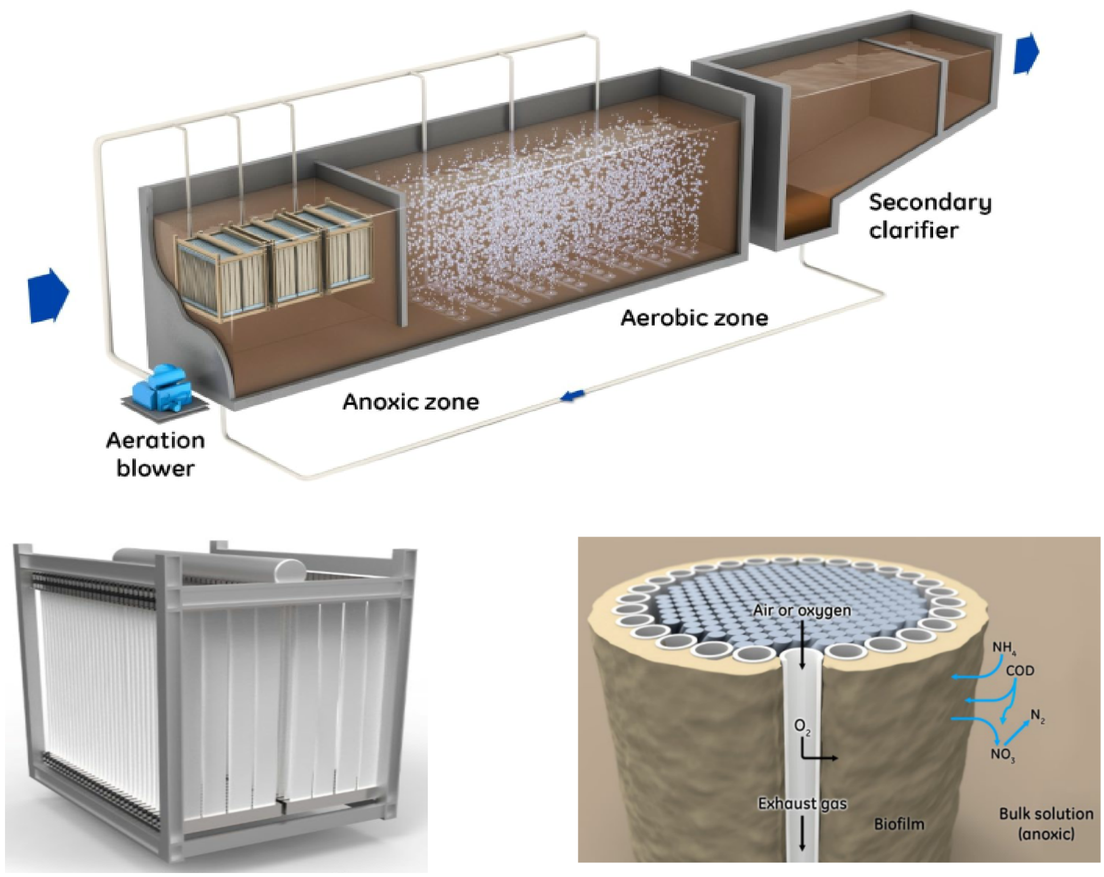



**Figure 4-23. MABR Membrane**

*Up: Installation Concept, Down Left: Membrane Cassette, and Down Right: Operation Principal*

### 4.6.4.4.1 Model Development

The calibrated BioWin model was used to determine capacity of secondary treatment with MABR technology installed within the existing infrastructure to treat future flows and loads. The ESWPAF was operated with two primary clarifiers, four aeration basins, eight secondary clarifiers, and one gravity thickeners. Note the number of process units were consolidated into one representative unit each for simplicity as shown on **Figure 4-24**. The BioWin models were configured in an MLE and four-stage Bardenpho processes with MABR technology to simulate MM conditions in winter and summer, respectively. Primary clarifier performance was based on the historical data.

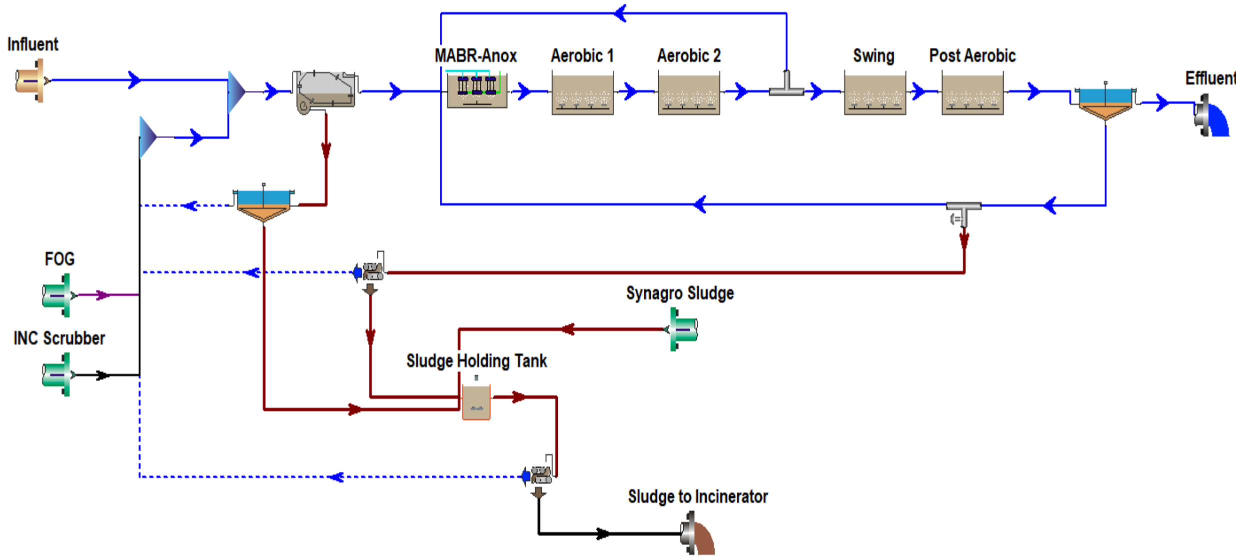



**Figure 4-24. BioWin Process Configuration Model for MABR Technology**

The capacity of the MABR technology for these two processes were analyzed at the MM loading conditions for summer and winter for the future influent characteristics described in **Table 4-36**. For the MLE process in winter, the historical water temperature of 13°C was selected. For the four-stage Bardenpho process in summer, water temperature of 20°C was selected. The maximum treatment capacity of the four-stage Bardenpho and MLE processes is defined as the maximum influent load that can be successfully treated over the course of 30 days without the resulting MLSS levels exceeding 4,200 mg/L (that is, the maximum MLSS level defined in the secondary clarifier analysis). The model results for both processes at MM conditions are presented in **Table 4-36**.

**Table 4-36. BioWin MABR Model Results for the MLE and Four-stage Bardenpho Processes for Future Maximum Month Conditions**

| Condition | MLE | Four -stage Bardenpho |
|---|---|---|
| | MM Winter 13ºC | MM Summer 20ºC |
| ***Aeration Basins*** | | |
| No. of Tanks Online | 4 | 4 |
| MABR Anoxic Volume, MG | 1.76 | 1.76 |
| Aerobic Volume, MG | 4.81 | 4.81 |
| Swing/Post Anoxic, MG | 1.92 | 1.92 |
| Re-Aeration, MG | 0.66 | 0.66 |
| Total Volume, MG | 8.15 | 8.15 |
| Aerobic Volume, % | 90 | 67 |
| MABR Cassettes | 320 | 320 |
| Membrane Surface Area/Cassette, $m^2$ | 2,340 | 2,340 |
| Total Membrane Surface Area, $m^2$ | 748,000 | 748,000 |
| Biofilm Thickness, mm | 0.2 | 0.2 |
| SRT, day | 9.0 | 7.5 |
| MLSS, mg/L | 3,450 | 3,500 |
| Internal Mixed Liquor Recycle Flow, mgd | 80 | 80 |
| RAS Flow, mgd | 27.5 | 26.7 |
| Average DO, mg/L | 2.0 | 2.0 |
| Air Flow Rate, scfm | 24,800[a] | 29,415[a] |
| ***Sludge Production*** | | |
| Primary Sludge, PPD | 102,200 | 111,600 |
| Thickened Primary Sludge, PPD | 94,800 | 103,400 |
| WAS, PPD | 33,600 | 41,000 |
| Thickened WAS, PPD | 32,950 | 40,200 |
| Total Thickened Sludge, PPD | 127,700 | 143,600 |
| ***Secondary Effluent*** | | |
| Flow Rate, mgd | 41.1 | 39.9 |
| BOD-5, mg/L | <10 | <10 |
| TSS, mg/L | <15 | <15 |
| TKN, mg/L | 2.0 | 1.8 |
| Ammonia, mg/L | 0.5 | 0.6 |
| Nitrate+ Nitrite, mg/L | 3.1 | 1.0 |
| TN, mg/L- PPD | 5.9 – 2,020 | 3.5 – 1,160 |

[a] Assumes est. alpha of 0.6 (AA) or 0.55 (MM), fouling of 0.8, beta pf 0.95, and SOTE of approximately 34 percent. Current maximum blower capacity is (6) x 9,500 scfm each, (5) duty and (1) standby.

The BioWin modeling was done to determine the maximum TN loading the secondary treatment can handle while meeting the permitted effluent TN of 1,568 lbs N/d. The modeling results showed that MLE process configuration produced effluent TN that exceeded the permitted limit. This was attributed to the capacity of the MLE and four-stage processes. MLE configuration can achieve effluent TN between 6 to 12 mg/L, while four-stage Bardenpho can achieve effluent TN less than 5.0 mg/L. The MABR technology operated at lower SRTs than conventional activated sludge process due to the high surface area of the membrane. Lower SRTs resulted in lower MLSS in the aeration basins reducing the solids loading in the secondary clarifiers. The MLSS in the aeration basins were approximately 10 percent less than the conventional activated sludge process. This result suggests that the facility could gain approximately 10 percent treatment capacity by implementing MABR technology.

The state point analysis was completed for the eight existing secondary clarifiers (100 feet diameter) for the MM in winter and summer, and with MD flows for 2045 as described in **Table 4-34**. At MM-13C the design MLSS concentration was 3,450 mg/L with a RAS flow of 27.5 mgd. At MM-20C and MD conditions, the design MLSS was 3,500 mg/L with RAS flows of 26.7 mgd and 30 mgd, respectively. The MD flow was 60 mgd. A MD condition with a safety factor of 1.2 was also analyzed. At 1.2 x MD condition, the design MLSS of 3,500 mg/L with RAS flow of 36 mgd was also used. The results of the SPA for the MM, MD and 1.2 MD conditions are shown on **Figure 4-25**.

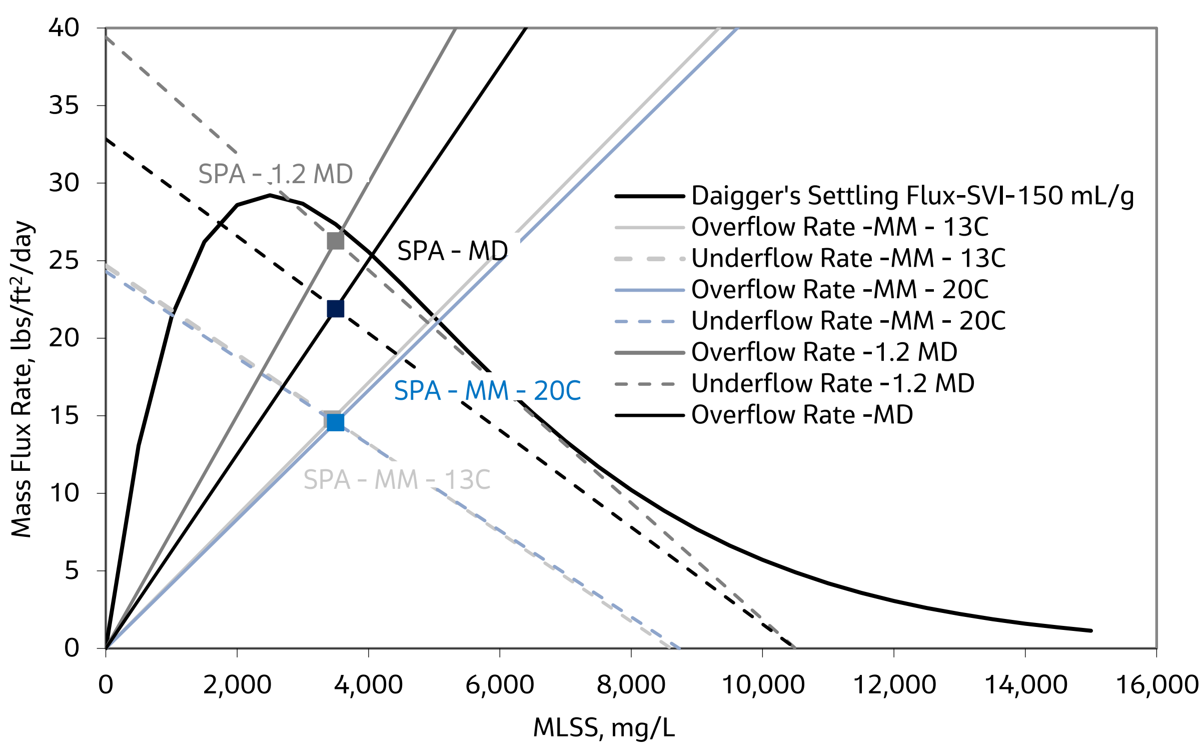



**Figure 4-25. Secondary Clarifiers State-Point Analysis for the MABR Technology**

As shown on **Figure 4-25**, at MM-13C, MM-20C, MD and 1.2 MD conditions the secondary clarifiers will not be overloaded and therefore have sufficient capacity to settle the MLSS and comply with an effluent TSS limit of 30 mg/L. The clarifier performance analysis was done assuming an average SVI of 150 mL/g. However, hydrocyclones can be installed in the WAS line to increase the sludge settleability resulting in sludge with lower SVI values.

A proposed layout for this alternative is shown on **Figure 4-26**. The major components and improvements needed to implement the MABR technology at the ESWPAF include the following:

- MABR membrane cassettes and support
- Membrane blower system including piping, valves and flowmeters
- Oxygen monitoring system

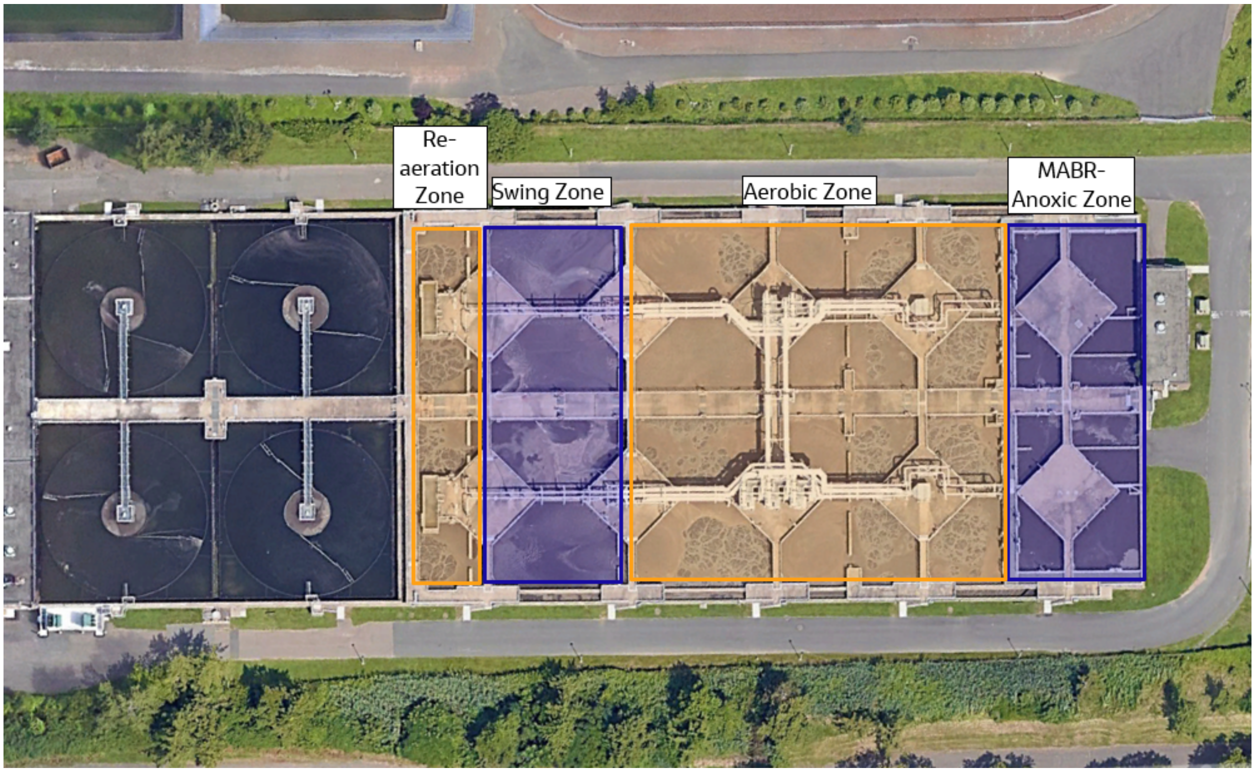



**Figure 4-26. Proposed Layout for the MABR Implementation**

### 4.6.5 Secondary Treatment Capacity Evaluation - Conclusions

The above evaluation of biological treatment technologies that can be added to the existing treatment process to provide additional capacity for flow and load increases to 2045, if needed, yielded the conclusions provided herein.

### 4.6.6 Existing Facilities and Loads

- Modeling results indicate that fully active nitrifying bacteria can increase the nitrification rate reducing the effluent ammonia concentrations. A system with fully active nitrifying bacteria requires reduced SRTs to meet the similar effluent TN in comparison to nitrification inhibition conditions. This can result in a gain of approximately 12 percent treatment capacity at the ESWPAF by removing nitrification inhibition.
- Dynamic modeling results for operation with all aeration basins and secondary clarifiers in service showed that the maximum nitrogen loading the secondary system can treat in winter (water temperature of 13°C) and Summer (water temperature of 20°C) are approximately 9,000 lbs N/d and 14,000 lbs N/d, respectively. This attributed to the capacity of the MLE and four-stage processes. MLE configuration can achieve effluent TN between 6 to 12 mg/L while four-stage Bardenpho can achieve effluent TN less than 5.0 mg/L.

- Dynamic modeling results for operation with three aeration basins and seven secondary clarifiers in service showed that the maximum nitrogen loading the secondary system can treat in winter (water temperature of 13°C) and summer (water temperature of 20°C) are approximately 8,000 lbs N/d and 12,000 lbs N/d, respectively.
- Dynamic modeling results indicated that the capacity limit of the secondary clarifiers is at approximately 22 lbs/ft$^2$/day, assuming a MLSS concentration of 4,200 mg/L, this results in an influent flow of approximately 50 mgd with a RAS flow of approximately 30 mgd. Flow conditions than result in solids flux rates greater than 22 lbs/ft$^2$/day can exceed the secondary clarifiers capacity to settle the MLSS resulting in effluent TSS higher than of 30 mg/L. The clarifier performance analysis was done assuming an average SVI of 150 mL/g and standard settling solid flux curve.

### 4.6.7 Future Loadings and Alternative Technologies

- Future flows and loads were modeled to determine the maximum TN loading the secondary treatment can handle while meeting the permitted effluent TN of 1,568 lbs N/d. The modeling results showed that MLE process configuration produced effluent TN that exceeded the permitted limit. This was attributed to the capacity of the MLE and four-stage processes. MLE configuration can achieve effluent TN between 6-12 mg/L while four-stage Bardenpho can achieve effluent TN less than 5.0 mg/L.
- The evaluation of secondary clarifier capacity for future flows and loads indicated that the secondary clarifiers will not have sufficient capacity to settle the MLSS and comply with an effluent TSS limit of 30 mg/L at the MD condition (60 mgd) . The clarifier performance analysis was done assuming an average SVI of 150 mL/g.
- Sludge settleability can be improved using hydrocyclones in the WAS line resulting in sludge with lower SVI values. Hydrocyclones are recommended as a future improvement to the ESWPAF. However, pilot testing of this technology is recommended before implementation. This technology can be easily implemented and is relatively low cost.
- Future flows and loads were modeled for MOB and MABR technologies. The modeling results showed that MLE process configuration produced effluent TN that exceeded the permitted limit. This was attributed to the capacity of the MLE and four-stage processes. MOB and MABR technologies operated at lower SRTs than conventional activated sludge process because of the high surface area of the media. The MLSS in the aeration basins were approximately 10 percent less than the conventional activated sludge process. This result suggests that the facility could gain approximately 10 percent treatment capacity by implementing either MOB or MABR technology. However, due to the high capital cost associated to these technologies, pilot testing is recommended to verify that they are suitable for increasing capacity in the secondary system at the ESWPAF.

## 4.7 Disinfection

### 4.7.1 Existing Conditions

Flow enters the existing disinfection basins through a common influent channel after passing through the secondary clarifiers. The design flow through the secondary flow path is 60 mgd. The plant currently has two disinfection tanks/cells that are each 136 feet long by 45 feet wide. Both the north and south tanks are divided lengthwise by two baffle walls each, which are at an elevation of 11 feet (NAVD 88). After completing three lengthwise passes through the tanks, the chlorine-treated wastewater flows to the common effluent channel and out through the existing plant outfall pipes.

### 4.7.2 Capacity Evaluation

Disinfection currently requires two rectangular disinfection tanks with a capacity of 100 mgd. These existing disinfection tanks will not provide the required chlorine contact time for peak flows of 187 mgd. Consequently, separate disinfection for dry and wet weather flows is recommended. The existing disinfection tanks will remain for treating up to 60 mgd of dry weather flow. Flows up to 87 mgd (Phase 2) and 127 mgd (Phase 3) will be treated by a new wet weather treatment facility. The wet weather treatment facility is discussed in detail in a later section of this report.

In previous evaluations (the 2011 Facility Plan, for example), conversion to ultraviolet (UV) disinfection was considered. However, new information on the potential for higher water elevations in New Haven Harbor indicate that sufficient head may not be available in the future to allow for UV disinfection. Consequently, Jacobs recommends that the ESWPAF retain chlorine-based disinfection into the future.

Chlorine disinfection has the following distinct advantages:

- The existing chlorine contact basins provide effective disinfection at low head loss.
- Keeping the existing infrastructure in place will prevent disruption to plant operation caused by construction.
- Chlorine disinfection is currently in place, will be needed for the wet weather disinfection facility, and is well understood by plant operators and staff.
- Replacement of chlorine disinfection with UV will increase head loss by approximately 2 feet. The hydraulics evaluation completed for this CSO LTCP Update indicates that the plant cannot accommodate this increase in head loss.

In summary, chlorine disinfection provides more flexibility when highly variable flow rates and fluctuating discharge elevations must be addressed. The existing baffle walls can be raised, when needed, to prevent overtopping due increases in harbor elevations. Other repairs can be made to combat any deterioration of the tank walls. Making such modifications is a cost-effective and minimally disruptive approach to disinfection at current and future flows.

Chlorine disinfection is cost-effective for the wet weather flows as well. Therefore, new disinfection tanks are recommended for the wet weather flows.

## 4.8 Plant Outfall

### 4.8.1 Existing Conditions

Flow is normally discharged to the New Haven Harbor through two 48-inch diameter outfall pipes made of polyethylene. The two existing outfall pipes were constructed in 1972. These pipes begin at an elevation of -1.04 feet (NAVD 88), pass through a riprap barrier, and slope down at 1.43 percent to an outlet structure at an elevation of -12.54 feet (NAVD 88). The two pipes each have an inside diameter of approximately 47 inches and length of approximately 1,100 feet. The outfall pipes exit to the harbor through a concrete headwall with wingwalls on either side.

When the outfalls cannot overcome the harbor elevation, a motorized gate is opened so that flow can also be discharged to the New Haven Harbor via the 90-inch overflow trough. This square box culvert begins at elevation -1.04 feet (NAVD 88), slopes down at 1.43 percent through riprap, and ends at an elevation of -3.04 feet (NAVD 88). The 90-inch box culvert has a length of approximately 100 feet. The outfalls and the overflow trough share a common effluent channel at the end of the existing chlorine contact tanks.

### 4.8.2 Capacity Evaluation

The capacity of the outfall was discussed in the Hydraulic Profile section of this report.

### 4.8.3 Outfall Inspection

An inspection of the existing dry weather flow outfall was conducted on July 12 and 13, 2022. During the inspection, ASI Group Ltd. (ASI) deployed an remotely operated vehicle (ROV) into both the north and south 48-inch polyethylene outfall pipes to evaluate their condition. The ROV used both video and sonar to capture any abnormalities in the pipes, including cracks, diameter changes, and debris. Following the internal inspection, a boat equipped with sonar was sent into New Haven Harbor to perform a side-scan inspection of the external condition of the twin 48-inch outfall pipes and outlet structure.

Additionally, a dive inspection was performed by A. DiCesare Associates, P.C. (ADI) on July 21, 2022. The inspection occurred along the length of the outfalls and the head wall. The results of the outfall inspection were as follows:

- North Outfall – A total of 19 joints were inspected. Most joints were spaced at 60-foot intervals with two 30-foot pipe sections. There were no leaking joints but one joint was noted to be separating from the crown. There was deformation of the pipe into an egg shape from 23 feet to around 200 feet. The deformation of the pipe is attributed to the fact that the pipe slope is not uniform. The pipe slopes down from the discharge location at the treatment plant to the bottom of the harbor in an "S-shape". This is believed to cause the pipe to deform.
- South Outfall – A total of 20 joints were inspected. Most joints were spaced at 60-foot intervals with one 30-foot section. There were no leaking joints but three joints were noted to be separated from the crown. The same pipe deformation was noted in the South Outfall as the North Outfall. The cause of the deformation is believed to be the same.
- The dive inspection showed no anomalies in the pipe, headwall, or wingwalls. There was no exposed pipe in the harbor bed. There is a large washout approximately 49 feet by 45 feet in front of the headwall from the flow scouring the harbor bottom.
- The outfalls should be inspected every 5 to 10 years to continue to evaluate the condition of the pipes and headwalls.

The full description of the inspection methods and results is documented in the August 8, 2022 Report from ADI. This report contains the results of the ADI dive inspection and the ASI ROV and Side Scan Sonar inspection.

## 4.9 Wet Weather Treatment

### 4.9.1 Wet Weather Flow Splitting

As noted previously, flows of up to 60 mgd will be disinfected through the existing chlorine contact facility after secondary treatment. Flows greater than 60 mgd will be disinfected through a new wet-weather treatment facility. Flows of 60 mgd or less will be directed to secondary treatment through two 60-inch pipes equipped with electromagnetic flow meters (magmeters) to split flow between the two facilities. The magmeters will measure the dry weather flow before it reaches a junction chamber and is directed to secondary treatment via an 84-inch pipe. Overflow of more than 60 mgd will bypass secondary treatment and be disinfected through the wet weather treatment facility.

### 4.9.2 Wet Weather Disinfection

Wet weather flows will be disinfected with sodium hypochlorite which will be injected at the influent channel of the wet weather treatment facility. The chlorinated PE will pass through the wet weather disinfection tanks and the two 72-inch wet weather discharge pipes. Wet weather flow will discharge to New Haven Harbor through the existing 90-inch box culvert and overflow trough. **Table 4-37** summarizes the design criteria for wet weather disinfection.

**Table 4-37. Process Design for Wet Weather Disinfection**

| Flows and Chemical Requirements | | Criteria |
|---|---|---|
| Peak design flow (mgd) | | 127 |
| Maximum chlorine dose (mg/L) | | 15 |
| Minimum peak flow contact time (minutes)[a] | | 30 |
| Maximum effluent chlorine residual (mg/L) | | 1.5 |
| **Chlorine Disinfection** | **Wet Weather Disinfection Tanks** | **Wet Weather Discharge Pipes** |
| Tanks/pipes in service | 2 | 2 |
| Tank 1, pass 1 length (feet) | 234 | - |
| Tank 1, pass 2 length (feet) | 234 | - |
| Tank 1, pass 3 length (feet) | 234 | - |
| Tank 2, pass 1 length (feet) | 234 | - |
| Tank 2, pass 2 length (feet) | 245 | - |
| Tank 2, pass 3 length (feet) | 245 | - |
| Width per tank pass (feet) | 13 | - |
| SWD (feet) | 14 | - |
| Discharge pipe diameter (feet) | - | 6 |
| Discharge pipe length (feet) | - | 1,700 |
| Contact time (minutes) | 22 | 8 |

[a] Contact time was calculated for the wet weather disinfection tanks and wet weather discharge pipes and does not include the existing outfall culvert

### 4.9.2.1 Wet Weather Disinfection Tanks and Discharge Pipes

The wet weather disinfection tanks and piping are shown on Figures 30 through 32 of **Appendix D**. The tanks will be 250-feet long by 90-feet wide. The tanks have been sized to fit within the confines of the current plant site. The facility will have two tanks/cells, a common influent channel, and common effluent channel. Flow will complete three passes through the tanks before overflowing the weirs leading to the common effluent channel. Gates will also be constructed to drain the tanks by gravity after wet weather events.  A pump system will be provided to return the tank volume to secondary treatment when conditions to prevent wet weather discharge under short term peaks.

The two 72-inch wet weather discharge pipes will be approximately 1,700 feet long. During Phase 2 of the upgrade projects, the wet weather discharge pipes will intersect at the west side of the existing disinfection basin. This will allow the wet weather flows to be discharged into New Haven Harbor via the existing 90-inch box culvert and overflow trough. The hydraulic evaluation conducted for this LTCP update found that a new outfall is not needed for the Phase 3 project.

### 4.9.2.2 Sodium Hypochlorite Storage

Sodium hypochlorite storage sizing for the wet weather disinfection facility was calculated based on a bi-weekly chemical delivery for the worst case of the following assumptions:

- Biweekly chemical use for odor control is 2,000 gallons.
- Four days of MD flow (22 mgd to be chlorine disinfected).
- Three days with 4-hour rain events of 2-year hourly peak flow (127 mgd to be chlorine disinfected).

The sodium hypochlorite dosing pumps will be located with the existing sodium hypochlorite pumps north of the inlet works building. Currently, the plant has two fiberglass reinforced plastic 5,500-gallon tanks for the 15 percent concentration sodium hypochlorite used for odor control. Assuming weekly deliveries are available during wet weather season, additional storage capacity is not needed for chlorine disinfection use. However, operations and management of chemical storage might be facilitated by adding separate day tanks for both the odor control and wet weather disinfection facilities.

#### 4.9.2.3 Control Narrative

The wet weather flow will be calculated by the facility's computer centralized system by subtracting the wet weather flow splitting flow readings from the sum of the primary influent flowmeters readings minus the primary sludge withdrawal flow rate.

Chemical dosing will be flow paced from the wet weather flows and/or influent flow readings. Dosing will be initiated when manually selected or when the wet weather flow-splitting flow meters indicate that flows exceeding 60 mgd have occurred for a pre-determined time or are nearing the 60-mgd threshold to begin using the wet-weather disinfection facility.

### 4.9.3 Wet Weather Conveyance

A single 84-inch pipe currently conveys flows from primary treatment to secondary treatment. This is made possible by an easement that runs east to west through a wooded area owned by the City of New Haven Park Department and then under a traffic circle before reaching secondary influent on the other side. In order to construct the new wet weather treatment facility, a junction chamber will be built to intersect the existing 84-inch pipe. This will allow the flow to be redirected to a new 84-inch pipe that will carry it to the new wet weather flow splitter box. A new 84-inch pipe will be constructed to convey flows to secondary treatment. Once the new flow path is constructed, the existing unused section of 84-inch pipe can be closed off and abandoned in preparation for construction of the new wet weather disinfection tanks.

The common effluent channel of the wet weather disinfection tanks will flow into two 72-inch pipes to convey the chlorine-treated wet weather flows east to west and parallel to secondary treatment. A junction chamber will connect to the existing outfall structure.

## 4.10 Solids Treatment

The solids treatment process at ESWPAF consists of one gravity thickener, one sludge storage tank, and a third tank that can be used as either a gravity thickener or as sludge storage to meet process demands. In addition, there are two gravity belt thickeners (GBTs) and sludge dewatering systems. Primary sludge is currently conveyed to a 60-foot diameter gravity thickener. The other gravity thickener is often used as a sludge holding tank.

As a result of the primary treatment system improvements previously identified, including the inclusion of CEPT, additional gravity thickening volume, sludge storage volume and sludge dewatering capacity are required to meet the increased sludge volume produced by CEPT. To meet the additional primary sludge thickening volume requirements, the construction of a new 60-foot diameter gravity thickener, and a 30 percent increase in sludge dewatering capacity is recommended.

WAS from the secondary treatment process is conveyed for thickening to the two GBTs before dewatering. No additional improvements to this facility are recommended for this operation.

## 4.10.1 Primary Sludge Solids Treatment System

### 4.10.1.1 Existing Primary Sludge Thickening Flows and Loads

Historical operations and laboratory data for the ESWPAF were obtained from plant staff and spanned a period from 2017 to 2021. **Table 4-38** summarizes the primary sludge flows and loads for the analysis period. The primary sludge flow to the gravity thickeners ranges from 1.4 to 2.1 mgd with a sludge loading of 64,000 to 170,000 lbs of primary sludge per day. The solids loading rate ranged from 22.6 to 60.1 lbs/ft$^2$/day, which is higher of the recommended solids loading rate of approximately 20 to 30 lbs/ft$^2$/day for gravity thickeners.

**Table 4-38. Summary of Historical Primary Sludge Flows and Loads**

| Parameter | AA | MM | MW | MD |
|---|---|---|---|---|
| Primary Sludge, mgd | 1.5 | 1.6 | 1.7 | 2.1 |
| Primary Sludge, PPD | 64,000 | 100,000 | 120,000 | 170,000 |
| Gravity Thickeners in Service | 1 | 1 | 1 | 12 |
| Solids Loading Rate, lbs/ft$^2$/day | 22.6 | 35.4 | 42.4 | 60.1 |
| TSS Removal, % | 65 | 65 | 68 | 75 |
| Thickened Primary Sludge, mgd | 0.15 | 0.28 | 0.34 | 0.46 |
| Thickened Primary Sludge, PPD | 41,500 | 64,900 | 81,600 | 127,600 |
| Gravity Thickeners Overflow, mgd | 1.4 | 1.3 | 1.4 | 1.6 |
| Gravity Thickeners Overflow, PPD | 22,500 | 35,100 | 38,400 | 42,400 |

### 4.10.1.2 Existing Primary Sludge Gravity Thickener Performance

The TSS removal rates were analyzed during the 2017 to 2021 monitoring period and are presented on **Figure 4-27**. As presented on **Figure 4-27**, the TSS removal percentage is plotted against the primary sludge TSS. The primary sludge TSS versus the TSS removal percent curve achieved by the primary sludge gravity thickeners is typical of WWTFs and indicates the primary sludge gravity thickeners at ESWPAF are performing within typical industry expectations.

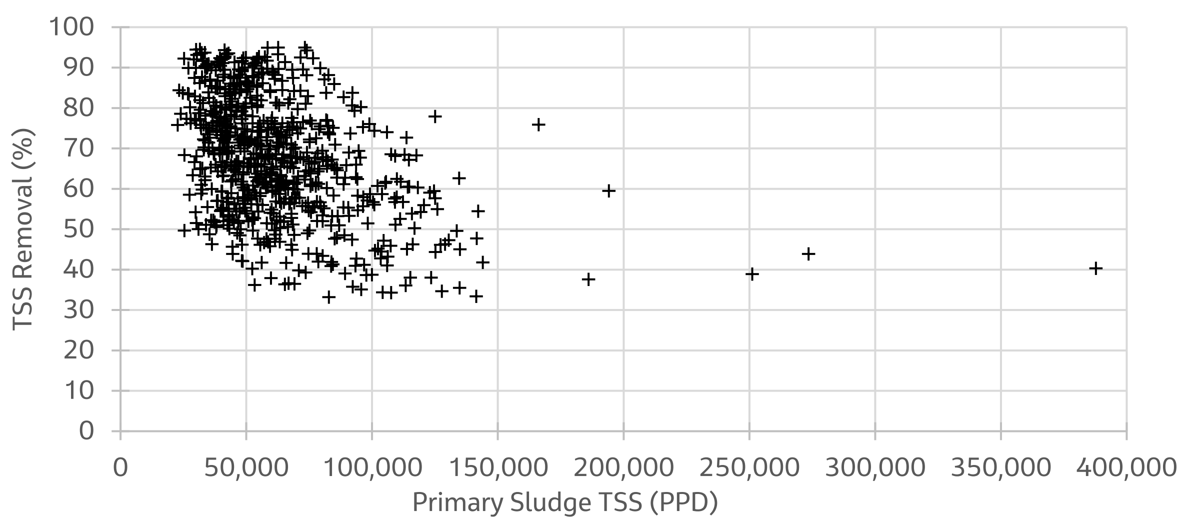



**Figure 4-27. Primary Sludge TSS vs TSS Removal Percent**

As presented on **Figure 4-28**, the primary sludge TSS removal percentage is plotted against the surface overflow rate of the gravity thickeners. The surface overflow rate versus the TSS removal percent slope is similarly typical of WWTFs and indicates the primary sludge gravity thickeners at ESWPAF are performing within typical industry expectations. As shown in the graph, as the surface overflow rate increases, the TSS removal performance typically decreases due to the higher hydraulic and TSS loadings.

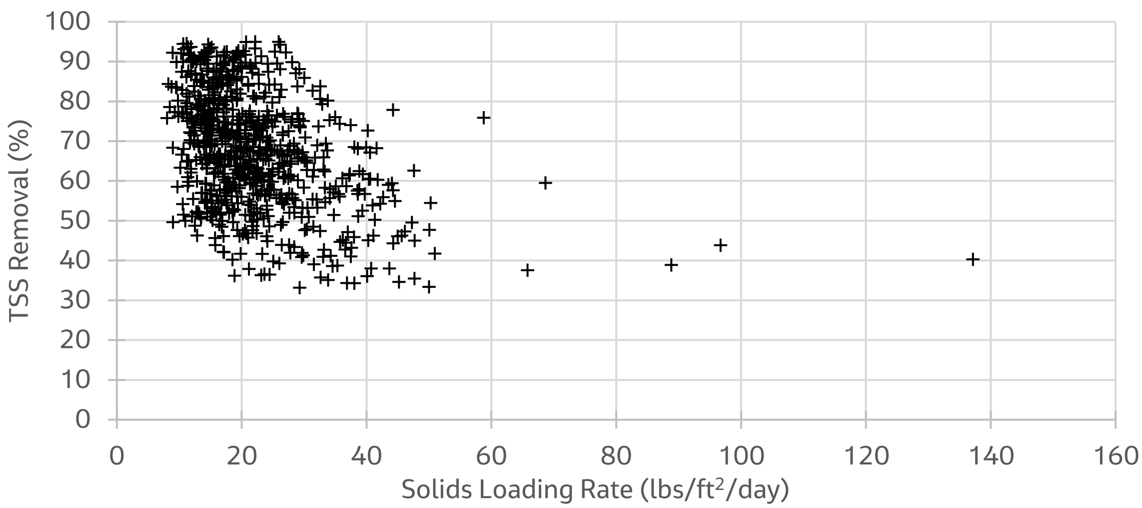



**Figure 4-28. Primary Sludge Gravity Thickener Surface Overflow Rate vs TSS Removal Percent**

### 4.10.1.3 Projected 2045 Primary Sludge Flows and Loads

The primary sludge flows and loads were projected to 2045 and are summarized in **Table 4-39** below. As previously identified, the incorporation of CEPT will increase the wet weather primary sludge flows and loading to the primary sludge gravity thickeners. To account for the additional projected wet weather flows and loads, two gravity thickeners are required to thicken the maximum month, week and day flow flows and loads.

**Table 4-39. Summary of Projected 2045 Primary Sludge Flows and Loads**

| Parameter | AA | MM | | MW | | MD | |
|---|---|---|---|---|---|---|---|
| Primary Sludge, mgd | 1.5 | 1.7 | | 1.8 | | 2.5 | |
| Primary Sludge (without Chemical Addition), PPD | 67,000 | 107,000 | | 124,000 | | 182,000 | |
| Chemical Addition | N/A | N/A | N/A | N/A | N/A | N/A | CEPT |
| Primary Sludge, PPD | 67,000 | 107,000 | 107,000 | 124,000 | 124,000 | 182,000 | 250,000[c] |
| Gravity Thickeners in Service | 1 | 1 | 2 | 1 | 2 | 2 | 2 |
| Solids Loading Rate, lbs/ft2/day | 23.7 | 37.8 | 18.9 | 43.9 | 21.9 | 32.2 | 44.2 |
| TSS Removal, % | 65[a] | 65[a] | 85[b] | 68[a] | 85[b] | 80[b] | 90[b] |
| Thickened Primary Sludge, mgd | 0.15 | 0.30 | 0.40 | 0.35 | 0.43 | 0.53 | 0.82 |
| Thickened Primary Sludge, PPD | 43,000 | 69,000 | 91,000 | 84,000 | 105,000 | 146,000 | 225,000 |
| Gravity Thickener Overflow, mgd | 1.3 | 1.4 | 1.4 | 1.5 | 1.4 | 2.0 | 1.7 |
| Gravity Thickener Overflow, PPD | 24,000 | 38,000 | 16,000 | 40,000 | 19,000 | 36,000 | 25,000 |

[a] Removal is based on the historical performance as shown in Table 4-39.
[b] Expected TSS removal with two gravity thickeners in operation.
[c] MD receives CEPT which increases primary sludge loading.

### 4.10.2 Primary Sludge Thickening Upgrades

A new 60-foot-diameter covered gravity thickener can be installed in the area adjacent to the existing gravity thickeners. Two gravity thickeners are needed to meet the increased sludge volume produced by CEPT in 2045. The existing primary sludge piping to the gravity thickener currently used will be replaced and extended to the new gravity thickener. New gravity thickened primary sludge pumps can be provided in the basement of the new primary sludge gravity thickeners.

#### 4.10.2.1 Primary Sludge Thickening Process Description

Solids can be separated from wastewater sludge by gravity. Solids that are heavier than water settle to the bottom of the thickener and are then compacted by the weight of the overlying solids. The new 60-foot-diameter circular gravity thickener can be used to thicken primary sludge solids before it is dewatered. Thickened sludge is moved to the center of the thickener by the rotating sludge rake. Steel pickets mounted to the rake mechanism provide a stirring motion, keeping the contents mixed, releasing any developing gas, and allowing large solids on the surface to be pulled under. The overflow from the gravity thickeners will flow by gravity back to the head of the plant.

The new gravity thickener will be covered with aluminum covers for odor control. These covers, along with the odor collection ductwork, will route the odorous air to the solids odor treatment train. **Table 4-40** summarizes the design criteria for the primary sludge gravity thickeners.

**Table 4-40. Primary Sludge Thickening Design Criteria**

| Primary Sludge Gravity Thickeners | | | |
|---|---|---|---|
| Tanks | 3 (2 + 1 spare) | | |
| Diameter, feet | 60 | | |
| SWD, From Top of Cone Up, feet | 10 | | |
| Tank Surface Area, each, $ft^2$ | 2,827 | | |
| Tank Volume, each, gal | 253,790 | | |
| Target Thickened Sludge Concentrations, % TS | 3-4 | | |
| | Dry Weather (60 mgd) | Wet Weather (187 mgd) | |
| Parameter | 75% TSS Removal Rate at Primary Clarifiers | 75% TSS Removal Rate at Primary Clarifiers | 90% TSS Removal Rate at Primary Clarifiers |
| Primary Sludge Flow Rate, mgd | 2.0 | 5.0 | 5.6 |
| Primary Sludge (without chemical addition), PPD | 150,000 | 296,000 | 296,000 |
| Chemical Addition | N/A | CEPT | CEPT |
| Primary Sludge Loading, PPD | 150,000 | 505,000[a] | 565,000[a] |
| Overflow Rate, gal/$ft^2$/day | 350 | 880 | 990 |
| Solids Loading Rate, lbs/$ft^2$/day | 26.5 | 89.3 | 100 |
| TSS Removal, % | 85 | 90 | 90 |
| Underflow Thickened Sludge, PPD | 128,000 | 455,000 | 509,00 |
| Thickened Sludge Flow, mgd | 0.5 | 2.0 | 2.1 |
| ***Thickened Primary Sludge Pumps*** | | | |
| Flow, gpm | 450 | | |
| Drive | Constant Speed | | |
| Units in Service | 2 duty, 1 standby | 4 duty | 4 duty |
| Hours of Operation per Day | 6 | 12 | 12 |

[a] Chemically enhanced primary sludge with 30 mg/l of Ferric Chloride and 20 mg/L of Sodium Hypochlorite.

### 4.10.3 Process Control Narratives

Gravity-thickened primary sludge will be pumped to the sludge holding tank. Two pumps (one per gravity thickener) will be in service during dry weather flows; two pumps will be standby (one per gravity thickener). All four pumps will be in service during peak wet weather flows.

The four gravity-thickened sludge pumps will operate in a rotating cycle based on a preselected time cycle. Pump(s) will be allowed to run if their respective gravity thickener rake arm is in service and the level in the sludge holding tank is below the high setting.

### 4.10.4 Sludge Storage

One 60-foot-diameter tank with a volume capacity of 0.25 mg, located north of the main building is currently used as sludge storage. The inclusion of CEPT at the primary clarifiers to treat the projected 2045 primary sludge flows and loads will increase daily thickened primary sludge to approximately 0.82 mgd as shown in **Table 4-39**. The current sludge storage tank will be able to store 30 percent of the total daily thickened primary sludge produced. As a result, the dewatering units will need to process the 70 percent remaining of the thickened primary sludge from CEPT

### 4.10.5 Gravity Belt Thickening

A GBT is a belt filter press (BFP) with a modified upper gravity drainage zone that allows water to drain through the moving, fabric-mesh belt while coagulating and flocculating solids. The secondary treatment WAS is currently thickened through the two GBTs before entering the sludge holding tank, where it is combined with the gravity-thickened primary sludge before dewatering. Separating primary and WAS thickening is a recommended practice for best performance of the two processes. Therefore, WAS will continue to be thickened at the GBTs.

The WAS dewatering unit processes will not be upgraded. However, a change in the hours of the GBTs operation might be necessary to accommodate the new flows. This will be further evaluated during design. A summary of the current and projected GBT performance is presented in **Table 4-41**.

**Table 4-41. Gravity Belt Thickening Performance**

| Parameter | AA | MM | MW |
|---|---|---|---|
| ***Current Performance (2017 – 2021)*** | | | |
| WAS, mgd | 0.31 | 0.40 | 0.46 |
| WAS, PPD | 27,000 | 38,000 | 42,000 |
| TSS Removal, % | 0.85 | 0.89 | 0.90 |
| GBT Filtrate, mgd | 0.23 | 0.30 | 0.33 |
| GBT Filtrate, PPD | 4,000 | 4,000 | 4,400 |
| Thickened WAS, mgd | 0.07 | 0.10 | 0.13 |
| Thickened WAS, PPD | 23,000 | 34,000 | 37,600 |
| ***Projected Performance (2045)*** | | | |
| WAS, mgd | 0.35 | 0.43 | 0.48 |
| WAS, PPD | 29,000 | 39,000 | 44,000 |
| TSS Removal, % | 0.85 | 0.89 | 0.90 |
| GBT Filtrate, mgd | 0.27 | 0.33 | 0.35 |
| GBT Filtrate, PPD | 4,000 | 4,000 | 5,000 |
| Thickened WAS, mgd | 0.08 | 0.10 | 0.13 |
| Thickened WAS, PPD | 25,000 | 35,000 | 39,000 |

[a] Removal based on historical performance.

### 4.10.6 Sludge Dewatering and Incineration

The BFPs, and centrifuges will require replacement within the next 10 years. It is recommended that the current dewatering equipment be replaced with three high solids centrifuges (two duty and one standby). The Multiple Hearth Furnace is in good condition and should be capable of operation for the next 10 to 15 years without any significant repairs. However, some of the ancillary equipment is showing signs of age and will require replacement or rebuilding within the next 20 years. Sludge produced in the facility is collected with the sludge in the sludge storage tanks.

The historical annual average sludge mass collected in the sludge storage tanks is approximately 108,800 PPD, of which approximately 64,800 PPD are sludge produced in the facility and 44,100 PPD are contract operations sludge. The reported annual average solids cake sent to the incinerator is approximately 88,000 PPD, of which approximately 43,900 PPD are solids produced in the facility and 44,100 PPD are solids from contract operations. A summary of the current dewatering unit performances and incinerator loadings are presented in **Table 4-42**.

**Table 4-42. Current Dewatering Units Performance**

| Parameter | AA | MM | MW |
|---|---|---|---|
| ***Current Performance (2017 – 2021)*** | | | |
| ESWPAF Sludge (TPS + TWAS), PPD | 64,800 | 96,200 [a] | 111,700 [a] |
| Contract Operations Sludge, PPD | 44,100 | 66,800 | 71,500 |
| Total Sludge (ESWPAF + External), PPD | 108,800 | 140,500[a] | 161,600[a] |
| Reported Cake to Incinerator, PPD | 88,000 | 109,000 | 116,000 |
| ESWPAF Cake, PPD | 43,900 | 68,800 [a] | 84,400 [a] |
| Contract Operations Cake, PPD | 44,100 | 66,800 [a] | 71,500 [a] |

[a] Sludge quantity for MM and MW are not concurrent. The ESWPAF and External sludge for MM and MW conditions are not concurrent.

Historically, the facility has been able to handle annual average sludge quantities of approximately 108,800 PPD. As it is projected that the facility will produce an annual average of approximately 68,000 PPD of sludge by 2045, contract operations may be limited to an annual average of approximately 40,800 PPD to ensure that the total amount of processed solids does not exceed the capacity of the existing incinerator.

It is anticipated a significant increase in the production of thickened primary sludge after the installation of the second gravity thickener. As a result, the contract operation solids will need to be reduced to ensure that the total amount of processed solids does not exceed the capacity of the existing incinerator at MM and MW conditions. A summary of the projected dewatering unit performances and incinerator loading are presented in **Table 4-43.**

**Table 4-43. Projected Dewatering Units Performance**

| Parameter | AA | MM | MW | MW |
|---|---|---|---|---|
| ***Projected Performance (2045)*** | | | | |
| Gravity Thickeners in Service | 1 | 1 | 1 | 2 |
| ESWPAF Sludge (TPS + TWAS), PPD | 68,000 | 101,000[a] | 115,000[a] | 135,000[a] |
| Reported Total Sludge Capacity (ESWPAF + External), PPD | 108,800[b] | 140,500[b] | 161,600[b] | 161,600[b] |
| Reported Cake to Incinerator Capacity, PPD | 88,000[b] | 109,000[b] | 116,000[b] | 116,000[b] |
| Projected ESWPAF Cake, PPD | 47,200 | 72,200[a] | 86,900[a] | 102,300[a] |
| Projected Contract Operations Cake, PPD | 40,800 | 62,000[a] | 68,200[a] | - |

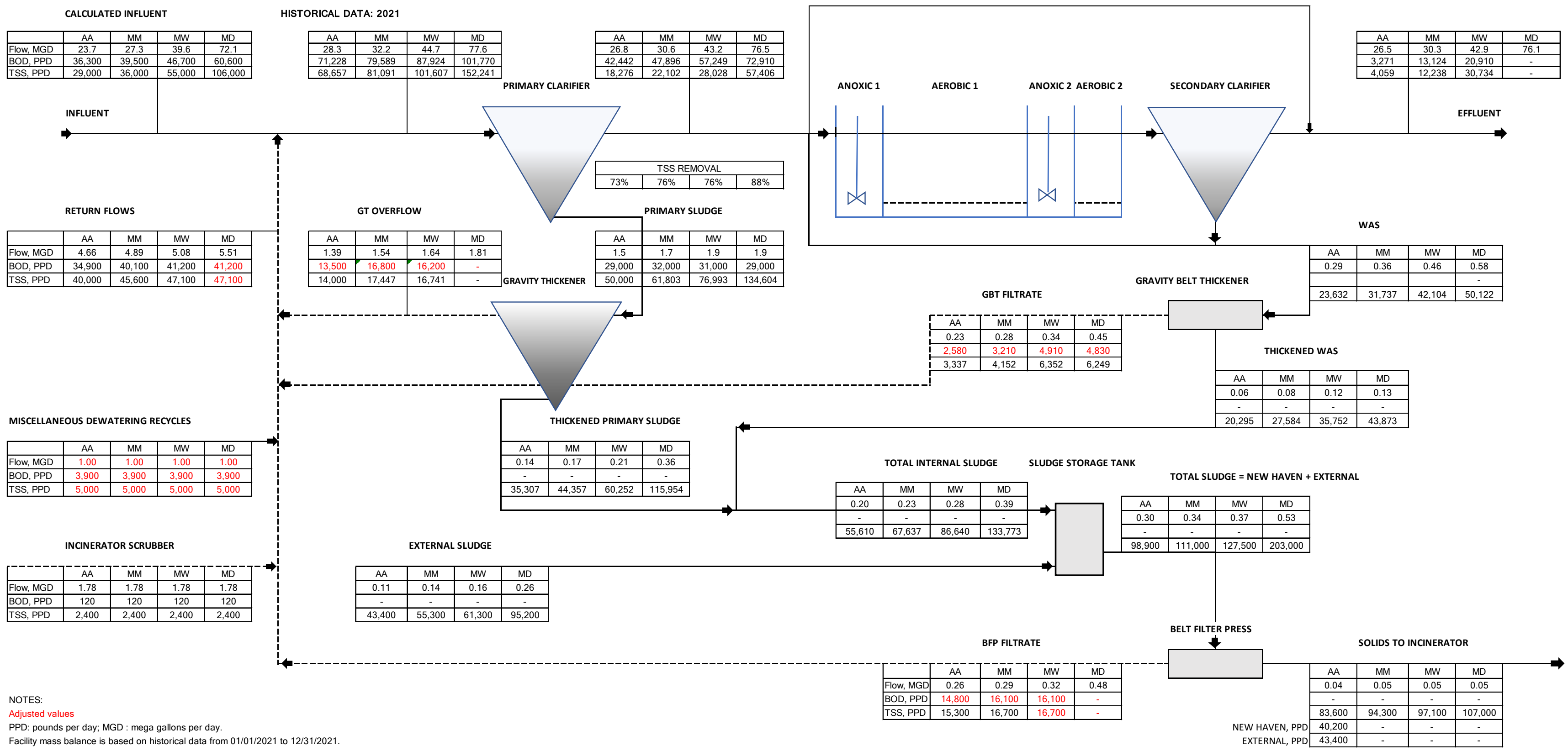
[a] Sludge quantity for MM and MW are not concurrent. The ESWPAF and External sludge for MM and MW conditions are not concurrent.

[b] Total sludge quantity and dewatering units' performance are based on historical data as shown in Table 4-42.

### 4.10.7 Solids Mass Balances

Facility solid mass balances were prepared for 2021 and projected 2045 flows and loads conditions. The solid mass balances for 2021 flows and loads conditions were based on the historical data and performance of the process units at the facility. The primary influent flows and loads for 2021 are lower than the values presented in **Table 4-13**. This year was selected as a better representation of the solids production in the facility as in previous years the facility experienced an increase in American Green fuel discharges. Results of the supplemental sampling were used to estimate the BOD loading of the return flows. BOD and TSS loadings for the raw influent were calculated from the difference between the primary influent and return flow loads. A miscellaneous dewatering recycles loading were included within the return flows to account for the unmeasured sludges from the FOG decant, solids handling wash water, sump discharges and other operations in the facility. The solid mass balances for the 2021 flows and loads conditions are shown on **Figure 4-29**.

The solid mass balances for the projected 2045 conditions were based on the primary influent flows and loads for 2021 projected to the future population in 2045. The projected primary influent flows and loads for 2045 are lower than the values presented in **Table 4-14**. This is because the projected values are based on much lower flows and loads observed in 2021. Historical performance of the process units was used to estimate the TSS removal in the primary clarifiers, gravity thickener, gravity belt thickeners and dewatering units. A calibrated BioWin model was used to estimate the projected waste activated sludge production. BOD and TSS loadings for the raw influent were calculated from the difference between the primary influent and return flow loads. A miscellaneous dewatering recycles loading were included within the return flows to account for the unmeasured sludges from the FOG decant, solids handling wash water, sump discharges and other operations in the facility. The solid mass balances for the projected 2045 flows and loads conditions are shown on **Figure 4-30**.




CALCULATED INFLUENT

| | AA | MM | MW | MD |
|---|---|---|---|---|
| Flow, MGD | 23.7 | 27.3 | 39.6 | 72.1 |
| BOD, PPD | 36,300 | 39,500 | 46,700 | 60,600 |
| TSS, PPD | 29,000 | 36,000 | 55,000 | 106,000 |

HISTORICAL DATA: 2021

| AA | MM | MW | MD |
|---|---|---|---|
| 28.3 | 32.2 | 44.7 | 77.6 |
| 71,228 | 79,589 | 87,924 | 101,770 |
| 68,657 | 81,091 | 101,607 | 152,241 |

| AA | MM | MW | MD |
|---|---|---|---|
| 26.8 | 30.6 | 43.2 | 76.5 |
| 42,442 | 47,896 | 57,249 | 72,910 |
| 18,276 | 22,102 | 28,028 | 57,406 |

TSS REMOVAL

| | | | |
|---|---|---|---|
| 73% | 76% | 76% | 88% |

Effluent

| AA | MM | MW | MD |
|---|---|---|---|
| 26.5 | 30.3 | 42.9 | 76.1 |
| 3,271 | 13,124 | 20,910 | - |
| 4,059 | 12,238 | 30,734 | - |

RETURN FLOWS

| | AA | MM | MW | MD |
|---|---|---|---|---|
| Flow, MGD | 4.66 | 4.89 | 5.08 | 5.51 |
| BOD, PPD | 34,900 | 40,100 | 41,200 | 41,200 |
| TSS, PPD | 40,000 | 45,600 | 47,100 | 47,100 |

GT OVERFLOW

| AA | MM | MW | MD |
|---|---|---|---|
| 1.39 | 1.54 | 1.64 | 1.81 |
| 13,500 | 16,800 | 16,200 | - |
| 14,000 | 17,447 | 16,741 | - |

PRIMARY SLUDGE

| AA | MM | MW | MD |
|---|---|---|---|
| 1.5 | 1.7 | 1.9 | 1.9 |
| 29,000 | 32,000 | 31,000 | 29,000 |
| 50,000 | 61,803 | 76,993 | 134,604 |

WAS

| AA | MM | MW | MD |
|---|---|---|---|
| 0.29 | 0.36 | 0.46 | 0.58 |
| | | | - |
| 23,632 | 31,737 | 42,104 | 50,122 |

GBT FILTRATE

| AA | MM | MW | MD |
|---|---|---|---|
| 0.23 | 0.28 | 0.34 | 0.45 |
| 2,580 | 3,210 | 4,910 | 4,830 |
| 3,337 | 4,152 | 6,352 | 6,249 |

THICKENED WAS

| AA | MM | MW | MD |
|---|---|---|---|
| 0.06 | 0.08 | 0.12 | 0.13 |
| - | - | - | - |
| 20,295 | 27,584 | 35,752 | 43,873 |

MISCELLANEOUS DEWATERING RECYCLES

| | AA | MM | MW | MD |
|---|---|---|---|---|
| Flow, MGD | 1.00 | 1.00 | 1.00 | 1.00 |
| BOD, PPD | 3,900 | 3,900 | 3,900 | 3,900 |
| TSS, PPD | 5,000 | 5,000 | 5,000 | 5,000 |

THICKENED PRIMARY SLUDGE

| AA | MM | MW | MD |
|---|---|---|---|
| 0.14 | 0.17 | 0.21 | 0.36 |
| - | - | - | - |
| 35,307 | 44,357 | 60,252 | 115,954 |

TOTAL INTERNAL SLUDGE

| AA | MM | MW | MD |
|---|---|---|---|
| 0.20 | 0.23 | 0.28 | 0.39 |
| - | - | - | - |
| 55,610 | 67,637 | 86,640 | 133,773 |

TOTAL SLUDGE = NEW HAVEN + EXTERNAL

| AA | MM | MW | MD |
|---|---|---|---|
| 0.30 | 0.34 | 0.37 | 0.53 |
| - | - | - | - |
| 98,900 | 111,000 | 127,500 | 203,000 |

INCINERATOR SCRUBBER

| | AA | MM | MW | MD |
|---|---|---|---|---|
| Flow, MGD | 1.78 | 1.78 | 1.78 | 1.78 |
| BOD, PPD | 120 | 120 | 120 | 120 |
| TSS, PPD | 2,400 | 2,400 | 2,400 | 2,400 |

EXTERNAL SLUDGE

| AA | MM | MW | MD |
|---|---|---|---|
| 0.11 | 0.14 | 0.16 | 0.26 |
| - | - | - | - |
| 43,400 | 55,300 | 61,300 | 95,200 |

BFP FILTRATE

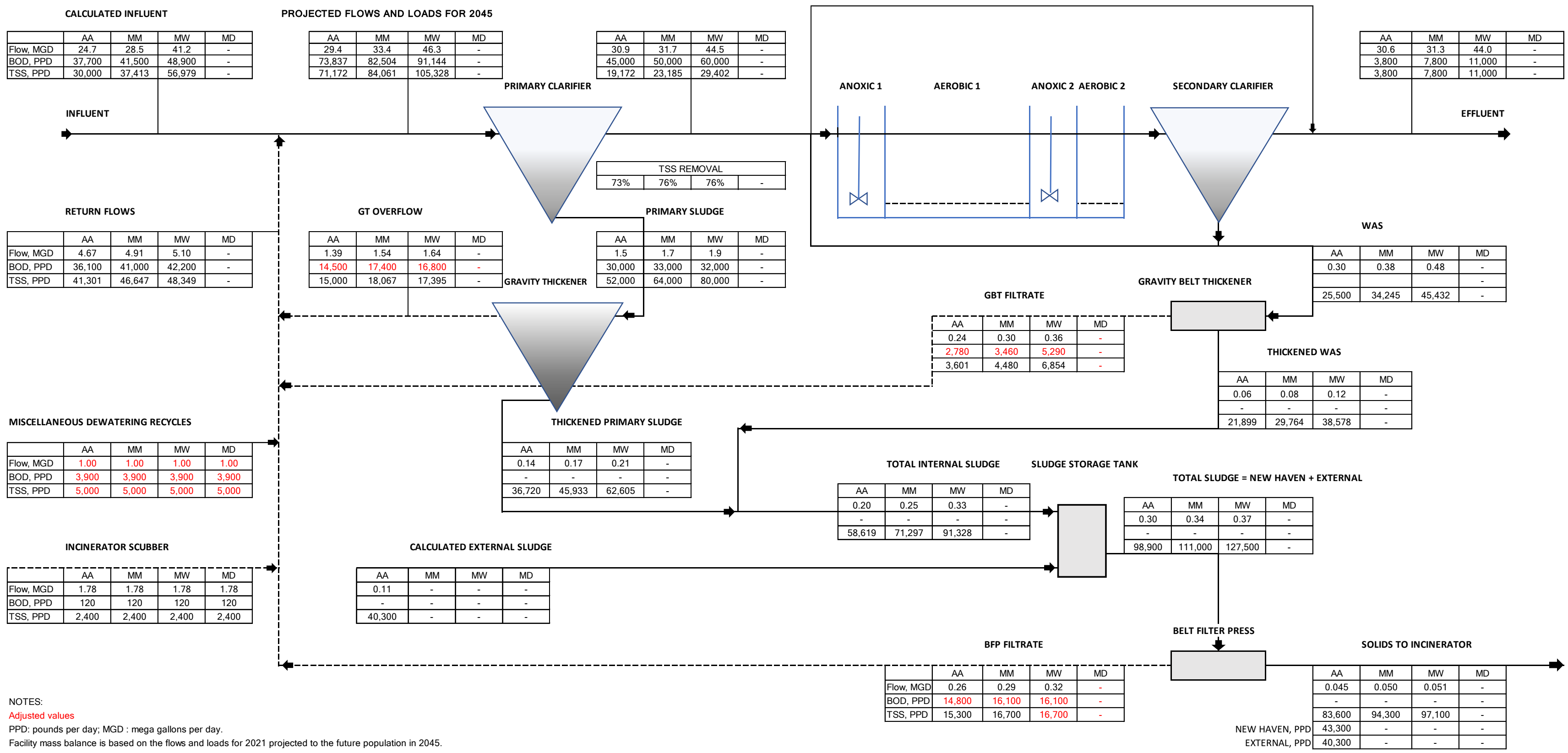
| | AA | MM | MW | MD |
|---|---|---|---|---|
| Flow, MGD | 0.26 | 0.29 | 0.32 | 0.48 |
| BOD, PPD | 14,800 | 16,100 | 16,100 | - |
| TSS, PPD | 15,300 | 16,700 | 16,700 | - |

SOLIDS TO INCINERATOR

| | AA | MM | MW | MD |
|---|---|---|---|---|
| | 0.04 | 0.05 | 0.05 | 0.05 |
| | - | - | - | - |
| | 83,600 | 94,300 | 97,100 | 107,000 |
| NEW HAVEN, PPD | 40,200 | - | - | - |
| EXTERNAL, PPD | 43,400 | - | - | - |

NOTES:
Adjusted values
PPD: pounds per day; MGD : mega gallons per day.
Facility mass balance is based on historical data from 01/01/2021 to 12/31/2021.
BOD masses for the return flows were adjusted based on the supplemental sampling results.
Thickened primary and WAS sludge masses for MM, MW and MD are not concurrent. Total internal sludge for MM, MW and MD are based on historical data.
External and total internal sludge masses for MM, MW and MD are not concurrent. Total sludge MM, MW and MD are based on historical data.
Belt Filter Press filtrate MM masses were assumed the same as MM masses. MM filtrate masses are calculated as the difference between total sludge and solids to incinerator.
Influent masses are calculated as the masses difference between primary influent and return flows.
Miscellaneous dewatering recycles is added to account for the unmeasured sludge from FOG decant, solids handling wash water, sump discharges, hose wash downs, and other operations in the facility.

## Figure 4-29. Solid Mass Balances for 2021




NOTES:

Adjusted values

PPD: pounds per day; MGD : mega gallons per day.

Facility mass balance is based on the flows and loads for 2021 projected to the future population in 2045.

The removal performance of the solids handling unts is the same as observed in 2021.

BOD masses for the return flows were adjusted based on the supplemental sampling results.

Thickened primary and WAS sludge masses for MM, MW and MD are not concurrent.

External sludge AA mass was calculated as the mass difference between the total sludge and total internal sludge

Belt Filter Press filtrate MW masses were assumed the same as MM masses. The MM filtrate masses are calculated as the difference between total sludge and solids to incinerator.

Influent masses are calculated as the masses difference between primary influent and return flows.

Miscellaneous dewatering recycles is added to account for the unmeasured sludge from FOG decant, solids handling wash water, sump discharges, hose wash downs, and other operations in the facility.

**Figure 4-30. Solid Mass Balances for the Projected 2045 Flows and Loads Conditions**

# 5. Cost Estimate and Implementation Schedule

## 5.1 Implementation Plan

As noted elsewhere in this report, the City of New Haven CSO LTCP extends over several years and consists of multiple strategies to reduce, eliminate, and minimize the impacts of CSO events, in accordance with EPA's NMC guidelines. Accordingly, several different types of projects are envisioned, and those projects are to be implemented through multiple phases of work. In general, the LTCP has been organized into the following two remaining phases of work:

- Intermediate-term Improvements
- Long-term Improvements

The focus of intermediate-term improvements has been the collection system. These improvements include closing CSOs, improving regulators (for example, raising weirs), and making major improvements to piping and pump stations within the system. The primary focus of long-term control improvements is the ESWPAF. The purpose of the proposed improvements at the WPAF is to provide additional capacity at the ESWPAF to accommodate increased flows due to the cumulative impacts of CSO closures and modifications of CSO regulators in the system.

The previous update to the CSO LTCP (CH2M 2018) and other evaluations, such as the *Wet Weather Capacity Improvements at the East Shore Water Pollution Abatement Facility* (CH2M 2011a) discussed several proposed improvements to the East Shore WPAF. The make-up and scope of the proposed improvement projects are periodically updated as more information becomes available, as the status and condition of existing facilities changes, and as new and potentially beneficial technologies become more developed.

Some of these improvement efforts at the WPAF (collectively known as Phase 1) have already been completed. These improvements include the following:

1. Electrical Upgrades
2. Odor Control Upgrades
3. Nitrogen Removal Improvements (Carbon Addition, 2nd Anoxic Zone)
4. Gravity Thickener and Sludge Storage
5. Process Air Compressor Upgrade

This previous section of this CSO LTCP Update discusses the remaining improvement projects, further defines the implementation plans for those projects, discusses new technologies (as appropriate). Updated estimated construction costs for those projects are presented here.

Phase 2 of the proposed improvements projects will provide a new treatment train (that is, a Wet Weather Treatment System) to allow wet weather flows of up to 147 mgd to be processed and an additional odor control scrubber system. Flow through the biological treatment system will be limited to 60 mgd. Peak flows greater than 60 mgd will be separated after primary treatment and be disinfected before being discharged through the plant outfall. **Phase 2** improvements are anticipated to include the following:

1. An additional scrubber system for odor control.
2. Piping modifications to convey PE to the facility.
3. A Flow-Splitting Facility for splitting primary effluent flows between the biological treatment system and the wet weather disinfection system.
4. A Wet Weather Disinfection System, including a dedicated chlorine contact tank to disinfect wet weather flows. Note: The system could be covered and/or connected to the existing odor control system, if feasible during this phase of work. Alternatively, the system could be designed to be covered and connected to odor control in the future.
5. Wet Weather Discharge Piping to convey disinfected flow to the plant outfall.

**Phase 3** improvements will allow the ESWPAF to process wet weather flows of up to 187 mgd. Potential projects include the following:

1. A new preliminary treatment building
   a. Influent flow metering
   b. Screening
   c. Grit removal
2. Primary treatment upgrades
   a. A new pipe gallery, or an extension to the existing pipe gallery
   b. Primary clarifier influent flow splitter
   c. Primary sludge pumps
   d. A fourth primary clarifier
   e. Rehabilitation of the three existing primary clarifiers
   f. Chemically-enhanced primary treatment (ferric chloride and polymer)
      i. Consider pilot testing CEPT between Phase 2 and 3
3. Secondary treatment improvements to incrementally increase treatment capacity
   a. Settleability improvements
      i. Hydrocylones
   b. Improvements to increase capacity
      i. MOB, or
      ii. MABR
4. Disinfection and outfall improvements
   a. Raise the height of the baffle walls of the existing chlorine contact basin to address hydraulic restrictions at the 100-year flood. This improvement is not required for the 25-year design flood.
   b. The existing outfall has been determined to be in relatively good condition. However, the Authority should plan for a future joint rehabilitation project.
5. Solids treatment and processing
   a. A third gravity thickener

## 5.2 Construction Cost Estimates – Background

"Order-of-magnitude" (that is, Class 5) cost estimates, as defined by the Association for the Advancement of Cost Engineering International were prepared for the various plant upgrade projects discussed in this CSO LTCP Update. Actual costs can be expected to range from -20 percent to -50 percent on the low side and +30 percent to +100 percent on the high side of the estimated cost.

This level of accuracy is consistent with costs prepared to compare the relative merits of several alternatives using sketches, general assumptions, and historical costs from similar projects before an exact project definition and specific preliminary design drawings are available. Because of the accuracy of this type of estimate and the variable nature of several factors, including the final scope of the project, this level of estimate is not a prediction of final construction costs.

The construction cost estimates for several projects were based on previous work prepared for the City and Authority (*Wet Weather Capacity Improvements and Nitrogen Reduction at the East Shore Water Pollution Abatement Facility,* [CH2M, 2011a], CSO LTCP Update [CH2M 2018]). Previous cost estimates were updated for inflation using several methodologies, as appropriate, including Engineering News Records indexes and revisions to unit costs for certain commodities and certain allowances to reflect current industry trends, cost data, and bidding experience.

Several allowances were incorporated into the estimates including a 15 percent allowance for General Conditions, a 5 percent allowance for Mobilization/Bonds/Insurance, a 5 percent allowance for contractor overhead, and an 10 percent allowance for contractor profit. A contingency value of 20 percent was also included in the construction cost estimates to account for in-scope items that are not yet defined at this level of estimate.

The estimated construction costs were not escalated to the mid-point of construction in this LTCP Update because the schedules for the projects are not firm. Costs for engineering, legal, administrative, and other costs were estimated by the Authority and added to the construction cost estimates.

## 5.3 Construction Cost Estimates – Summary

**Table 5-1** summarizes the construction cost estimates and approximate schedules for the Phase 2 and Phase 3 long-term improvement projects discussed in this report. The estimated construction costs are itemized according to the major unit process groupings described above. The Phase 2 project (Wet Weather Treatment System) is of high priority, is the most completely defined at this time, and will most likely be the next major project to be implemented.

The implementation schedule for the Phase 3 projects is unknown at this time. The currently-envisioned project groupings may be further prioritized and subdivided into smaller projects as the Authority's future needs evolve and become more solidified.

**Table 5-2** is a summarizes the estimated cost and schedules for all of the major components of the LTCP, as estimated in this 2022 CSO LTCP Update.

**Table 5-1. Estimated Construction Costs for Wet Weather Improvements at the East Shore Water Pollution Abatement Facility**

| ESWPAF Improvement Project | Million (2022) | Grant % | CWF Grant | NH Loan Share | GNH Loan Share | 2022 | 2023 | 2024 | 2025 | 2026 | 2027 | 2028 | 2029 - 2040 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Phase II - Wet Weather Treatment System and Odor Control | $65.0 | 50 | 32.5 | 13.0 | 19.5 | | | | | | | | |
| Phase III - Preliminary Treatment Improvements | $69.2 | 40 | 27.7 | 16.6 | 24.9 | | | | | | | | |
| Phase III - Primary Treatment Improvements (w/CEPT) | $58.5 | 40 | 23.4 | 14.1 | 21.1 | | | | | | | | |
| Phase III - Biological Treatment Improvements (hydrocyclones) | $4.3 | 40 | 1.7 | 1.0 | 1.5 | | | | | | | | |
| Phase III - Biological Treatment Improvements (capacity) | $50.3 | 40 | 20.3 | 12.2 | 18.3 | | | | | | | | |
| Phase III - 4th Gravity Thickener | $6.0 | 40 | 2.4 | 1.4 | 2.1 | | | | | | | | |
| Phase III – Disinfection and Outfall Improvements (Allowance) | $3.2 | 40 | 1.3 | 0.8 | 1.1 | | | | | | | | |
| **Subtotal (ESWPAF Improvements)** | **$256.5** | | **109.3** | **59.1** | **88.6** | | | | | | | | |

## Table 5-2. CSO Long-Term Control Plan Implementation Schedule and Project Cost Estimate

| 2022 Long Term Control Plan Update | Million (2022 $) | Grant % | CWF Grant | NH Loan Share | GNH Loan Share | 2022 | 2023 | 2024 | 2025 | 2026 | 2027 | 2028 | 2029-2040 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ***Intermediate Term Improvements*** | ***$ 173.9*** | | ***$ 83.2*** | ***$ 31.8*** | ***$ 58.9*** | | | | | | | | |
| 2022 Long Term Control Plan Update | 0.5 | 55% | 0.3 | 0.0 | 0.2 | | | | | | | | |
| Yale Campus/Trumbull Street Phase 2A Separation (CWF 2012-04) | 20.0 | 50% | 10.0 | 4.0 | 6.0 | | | | | | | | |
| Orchard Street Sewer Separation (CWF 2019-05) | 17.0 | 50% | 8.5 | 3.4 | 5.1 | | | | | | | | |
| Capacity Upgrade of East Street Pump Station (CWF 2017-01 + VE) | 53.3 | 50% | 26.7 | 10.7 | 16.0 | | | | | | | | |
| Process Air Compressor Improvements for Low Level Nitrogen Control | 12.9 | 20% | 2.6 | 0.0 | 10.3 | | | | | | | | |
| Phase II - Wet Weather Treatment System & Odor Control (CWF 2024-01) | 65.0 | 50% | 32.5 | 13.0 | 19.5 | | | | | | | | |
| Fair Haven CSO Improvements (CWF 2023-02) | 3.5 | 50% | 1.8 | 0.7 | 1.1 | | | | | | | | |
| Wet Weather Flow Conveyance Study from West Side (CWF 2024-02) | 0.5 | 55% | 0.3 | 0.0 | 0.2 | | | | | | | | |
| 2027 Long Term Control Plan Update and Model Update (CWF-2025-01) | 1.2 | 55% | 0.7 | 0.0 | 0.5 | | | | | | | | |
| ***Long Term Improvements (ESWPAF)*** | ***$ 518.4*** | | ***$ 236.0*** | ***$ 112.5*** | ***$ 169.8*** | | | | | | | | |
| Fair Haven CSO Improvements - Phase 3 (CWF 2028-01) | 20.0 | 50% | 10.0 | 4.0 | 6.0 | | | | | | | | |
| Wet Weather Conveyanace Improvemens from West Side to Harbor | 25.5 | 50% | 12.8 | 5.1 | 7.7 | | | | | | | | |
| Capacity Upgrade of Boulevard Pump Station | 45.9 | 50% | 23.0 | 9.2 | 13.8 | | | | | | | | |
| Capacity Upgrade of Union Pump Station, Force Main, Bridge over RR | 25.2 | 50% | 12.6 | 5.0 | 7.6 | | | | | | | | |
| 2032 Long Term Control Plan Update | 0.8 | 55% | 0.4 | 0.0 | 0.4 | | | | | | | | |
| Phase III Wet Weather Improvements at the ESWPAF: (Preliminary Treatment, Primary Treatment, Gravity Thickening, Disinfection, Outfall Improvements, Biological Treatment Improvements) | 233.0 | 40% | 93.2 | 55.9 | 83.9 | | | | | | | | |
| 2037 Long Term Control Plan Update and Model Update | 1.5 | 55% | 0.8 | 0.0 | 0.7 | | | | | | | | |
| Fair Haven CSO Improvements - Phase 4 | 92.8 | 50% | 46.4 | 18.6 | 27.8 | | | | | | | | |
| CSO Storage Tanks/Separation/Green Infrasturcture | 73.7 | 50% | 36.9 | 14.7 | 22.1 | | | | | | | | |
| ***Estimated Total*** | ***$ 692.3*** | | ***$ 319.2*** | ***$ 144.3*** | ***$ 228.8*** | | | | | | | | |

# 6. References

CH2M HILL, Inc. (CH2M). 1997. *Long-Term Combined Sewer Overflow Control Plan*. Final.

CH2M HILL, Inc. (CH2M). 2011a. *Wet Weather Capacity Improvements at the East Shore Water Pollution Abatement Facility*. Final, amended. February.

CH2M HILL, Inc. (CH2M). 2011b. *Combined Sewer Overflow Long-Term Control Plan Update*. March.

CH2M HILL, Inc. (CH2M). 2011c. *Wet Weather Capacity Improvements and Nitrogen Reduction at the East Shore Water Pollution Abatement Facility.*

CH2M HILL, Inc. (CH2M). 2015. *Hydraulic Model Update*. Final Report. January.

CH2M HILL, Inc. (CH2M). 2018. *City of New Haven Long-Term Combined Sewer Overflow Control Plan*. Final, revised. March.

Federal Emergency Management Authority (FEMA). May 16, 2017. *FEMA Risk Insurance Study Number 09009CV002D, Volume 2 of 10, New Haven County, Connecticut, Version 2.3.3.2.*

FEMA. Revised, July 8, 2013. *Flood Insurance Rate Map for New Haven County, Connecticut, Panel 442 of 635, Map 09009C0442J.*

New England Interstate Water Pollution Control Commission (NEIWPCC). 2016. *Technical Release 16, Guides for the Design of Wastewater Treatment Works* (TR-16). Revised. May.

U.S. Army Corps of Engineers. 1988. *New England Coastline Tidal Flood Survey, Fairfield, Conn. To East Haven, Conn.*, Plate C-5.

U.S. Environmental Protection Agency (EPA). 1995. *Combined Sewer Overflows: Guidance for Nine Minimum Controls*. EPA 832-B-95-003. May.