WPCF Evaluation and Fiscal Sustainability Plan with A Preliminary Engineering Report for Denitrification Filters and Odor Control

Town of Wareham, MA

Draft for Review

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Executive Summary

This report serves as a plant evaluation, fiscal sustainability plan and preliminary engineering report for the Town of Wareham to help evaluate the facility as a whole, prioritize upgrades for the denitrification filters and odor control, as well as discuss expansion options for the Water Pollution Control Facility (WPCF). This report analyzes the Town's wastewater needs and evaluates the current influent flows as well as provides recommendations for future WPCF expansion.

The report includes an evaluation of the existing processes at the WPCF. The existing process evaluation includes a description of the major systems at the facility; an evaluation of the system standards and guidelines to determine the system's capacity to handle future flows; an explanation of the operational issues of the system which were determined through site visits as well as comments from the Town; and recommendations for upgrades to properly handle current flows or accommodate future flows. From the existing process evaluation and recommendations, a criticality matrix is developed to rank the needs of the existing processes at the facility. The criticality matrix assesses the performance and condition of the processes and assigns a risk rating from Low to Very High to each piece of equipment. This risk ranking is a function of the probability the equipment will fail and the consequence of it failing.

Using the criticality matrix, planning level capital improvement costs are estimated for existing process improvements, creating a fiscal sustainability plan that the Town can use for annual facility budget planning. Additionally, a preliminary engineering report is provided for immediate upgrades that are in design, including the denitrification filter expansion, equalization basin odor control, and plant water well installation projects. In addition to these planned WPCF Improvement Projects, the report provides recommendations for the sludge processing and settling issues that the Town has been experiencing. These recommendations include performing a polymer jar test to analyze the efficacy of the polymer used at the facility to determine if the polymer selection is causing the facility's issues with solids settling and disposal.

To plan for future flows and facility expansion, it is recommended that the Town's Comprehensive Wastewater Management Plan (CWMP) be revised. This report notes that the Town has committed flow above the current design flow capacity of the facility and the facility currently encounters issues with flow and storage during peak wet weather events. Over-committed current flows without future flows defined by a CWMP makes it difficult to properly recommend upgrades or provide an estimate of future expansion costs. None the less, conceptual level expansions to the facility were investigated for a range of possible future flows. A conceptual level engineer's opinion of probable costs is provided to upgrade the facility to be a flow-through facility with a larger Modified Ludzack-Ettinger (MLE) system or a Membrane Bioreactor (MBR) system. Such a facility upgrade project is conceptually estimated to range up to $130,000,000 in 2025 dollars (mid-point of construction) depending on multiple factors and the date of construction. Additionally, there are multiple processes that are in need of upgrade due to either condition or capacity issues. The upgrades of these processes should be considered in conjunction with expansions to the treatment capacity. The top two priorities recommended for upgrades and capacity expansions are the preliminary treatment (headworks) and the secondary clarification processes.

In conclusion, the following are action items recommended to be taken.

- Action Item No. 1: Due to overcommitted flows, do not add additional influent flows until the following improvements have been made to the facility:

- Increase effluent discharge capacity to allow facility treatment design flows (up to 2 mgd) to be discharged.
- Complete construction of additional Equalization Basins 3 and 4.
- Complete construction of additional denitrification filters for additional redundancy.
- Complete additional influent nutrient testing to allow loading capacity to be determined. The current facility load capacity appears to be 60 to 70 percent of the design load capacity.

- Action Item No. 2: Complete the update to the Comprehensive Wastewater Management Plan (CWMP) to include future anticipated flows.
  - Determine the location of additional discharge capacity. This is currently being investigated as part of a separate project by GHD.
- Action Item No. 3: Continue with current design projects to upgrade the denitrification filters and add an equalization basin with odor control.
- Action Item No. 4: Perform a polymer jar test to analyze the efficacy of the polymer used at the facility to determine if the polymer selection is causing the facility's issues with poor thickening performance which seems to then cause an inability to dispose of enough liquid sludge which then cause issues with the facility being able to remove sludge from the secondary process which in turns leads to settling issues in the clarifiers and potentially with excessive backwashing and clogging of filters.
- Action Item No. 5: Due to condition and capacity issues, proceed with the most critical process upgrades, the top two priorities are indicated in the following table.

Table ES.1 Top Two Priority Process Upgrades

Priority	Need	Reason	Modifications	Budgetary Total Project Costs (2021 $)
1	Preliminary Treatment Upgrade (including septage receiving).	Most heavily used portion of the facility; significant rehabilitation is needed.	Rehabilitation of all processes and equipment.	$6,500,000
2	Secondary Clarification.	Shallow tanks have shown to be prone to failure; equipment is at or beyond its useful life; upgrade needed for future capacity.	Construction of two new 85-foot diameter secondary clarifiers will increase capacity from 1.56 mgd to 3 mgd.	$11,000,000

- Action Item No. 6: Use the fiscal sustainability plan for annual facility budget planning and existing process equipment improvements and rehabilitation. The Fiscal Sustainability Plan is intended to be a guide, used for planning and proactive improvements to the facility. It is not intended to imply a required level of spending from the Wareham WPCF enterprise fund. The budget for annual improvements needs to be considered in the context of what is affordable for the fund.

- The Town could use this information as part of an asset management program which would also incorporate revenue and major capital projects and would allow for proactive and continuous short- and long-term planning. The Massachusetts Department of Environmental Protection (MassDEP) has a history of providing grant funding for such projects and it is highly recommended that the Town pursue this state offered grant funding.

Table of Contents

Figure Index

Table Index

Appendix Index

1. Introduction

1.1 Purpose

The purpose of this report is to provide the Town of Wareham with an evaluation of their current Water Pollution Control Facility (WPCF) and recommend improvements to the facility. The report can be used as a Fiscal Sustainability Plan to help the Town plan for the future, and can also provide background and support for future projects.

1.2 Scope

The scope of this report is summarized below:

- Review of background information and data for the facility.
- Evaluation of liquid treatment processes.
- Evaluation of solid treatment processes.
- Recommended improvements to the facility.
- An engineer's opinion of probable costs for the recommended improvements.

In addition, the report was summarized in a fiscal sustainability format to assist the Town in pursuing SRF funding through the MassDEP.

Finally, a preliminary engineering report was developed as one of the final chapters for two processes that are currently under design—Equalization Basin 5 and additional denitrification filters.

1.3 Limitations

This project is intended to be an evaluation of the existing and future needs of the WPCF. Drawings and engineer's opinion of probable costs are intended to be for evaluation purposes. Products from this evaluation should be considered conceptual and do not represent either a preliminary or final design.

1.4 References and Guidelines

The following design guidelines and standards have been adopted for this project:

- TR-16 Guides for the Design of Wastewater Treatment Works, prepared by the New England Interstate Water Pollution Control Commission, 2011 Edition Revised in 2016
- Water Environment Federation; "Design of Municipal Wastewater Treatment Plants"; WEF Manual of Practice No 8; Fifth Edition; 2010
- Tchobanoglous, George; Burton, L. Franklin; Stensel, H. David; "Wastewater Engineering: Treatment and Reuse"; Metcalf and Eddy, Inc.; Fifth Edition; 2014

1.5 Summary of Other Projects or Reports

- *Equalization Basin Peak Storage Capacity Methodology Memorandum;* prepared by GHD; February 2019.
- *Modifications to Increase Influent Equalization Capacity;* prepared by GHD; April 2019.
- *Final Memorandum and Equalization Basin Recommendations;* prepared by GHD; June 2019.
- *Final Memorandum and Equalization Basin Recommendations: Supplemental Memo.* prepared by GHD; August 2019.
- *Memorandum of Capacity at the Wareham Water Pollution Control Facility;* prepared by GHD; September 2020.
- *Wareham WPCF Expansion Memorandum***.** Southeast New England Program (SNEP); prepared by GHD; September 2020.

2. Existing Facility and Data

2.1 Existing Wareham Facility

The existing Wareham WPCF is a 1.56 mgd average daily flow (ADF) facility that provides treatment for the Town of Wareham and a portion of the Town of Bourne. The facility was first put in service in 1972. It was then upgraded in 1979, 1989, and 2005. The WPCF is located at 6 Tony's Lane, Wareham, MA. An aerial photograph of the facility is shown in Figure 2.1.

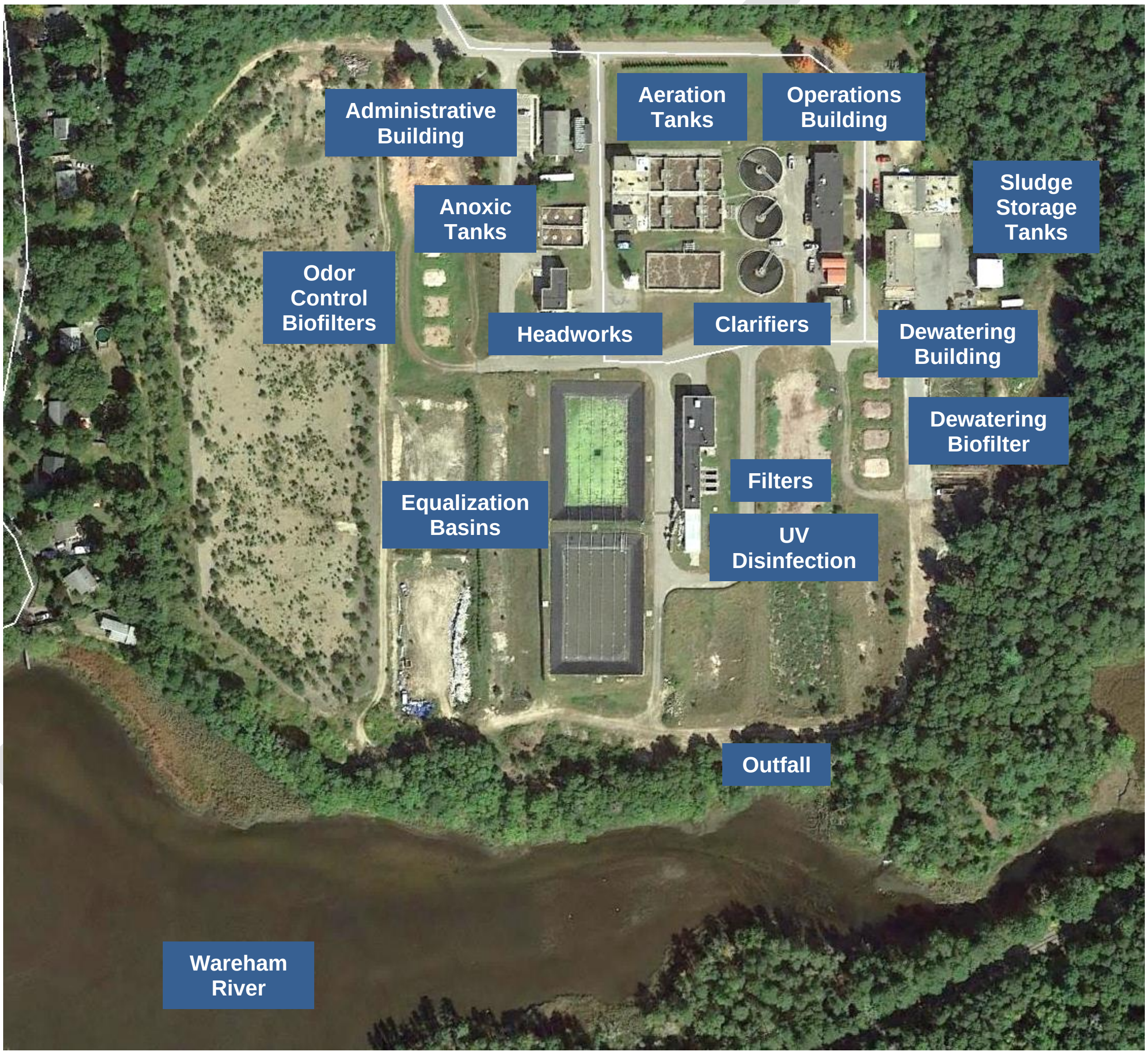


Figure 2.1 Existing Wareham WPCF Aerial

The existing secondary process is a Modified Ludzack-Ettinger (MLE) system consisting of two anoxic tanks and three aerobic tanks. The existing major processes and equipment are listed in Table 2.1.

Table 2.1 Major Processes and Equipment

Process	Number of Units
Influent Screen	1
Grit Tank	1
Anoxic Tanks	2
Aerobic Tanks	3
Secondary Clarifiers	3
Downflow Denitrification Filters[1]	3
Ultraviolet Radiation Modules	12
Equalization Basins[2]	2
Notes: 1. Three additional denitrification filters are under design. 2. Two additional equalization basins are under construction.	

2.1.1 Design Influent Flows

The existing WPCF is designed to treat the influent flows outlined in Table 2.2.

Table 2.2 Current Influent Design Flows from the Operations and Maintenance Manual

Parameter	Influent Flow (mgd)
Average Day – Permit Effluent Limit	1.56
Average Day – Design Treatment Capacity	2.00
Maximum Day	3.48
Peak Hour	5.39

The National Pollutant Discharge Elimination System (NPDES) permit effluent discharge limit for the WPCF is 1.56 mgd average day. The system is designed so that the secondary process can treat a peak of 2.00 mgd, with all additional influent being diverted to equalization basins until there is capacity to treat it.

2.1.2 Existing Effluent Limits

The Wareham WPCF discharges its effluent into the Agawam River. The effluent limits outlined in the NPDES permit are described in Table 2.3.

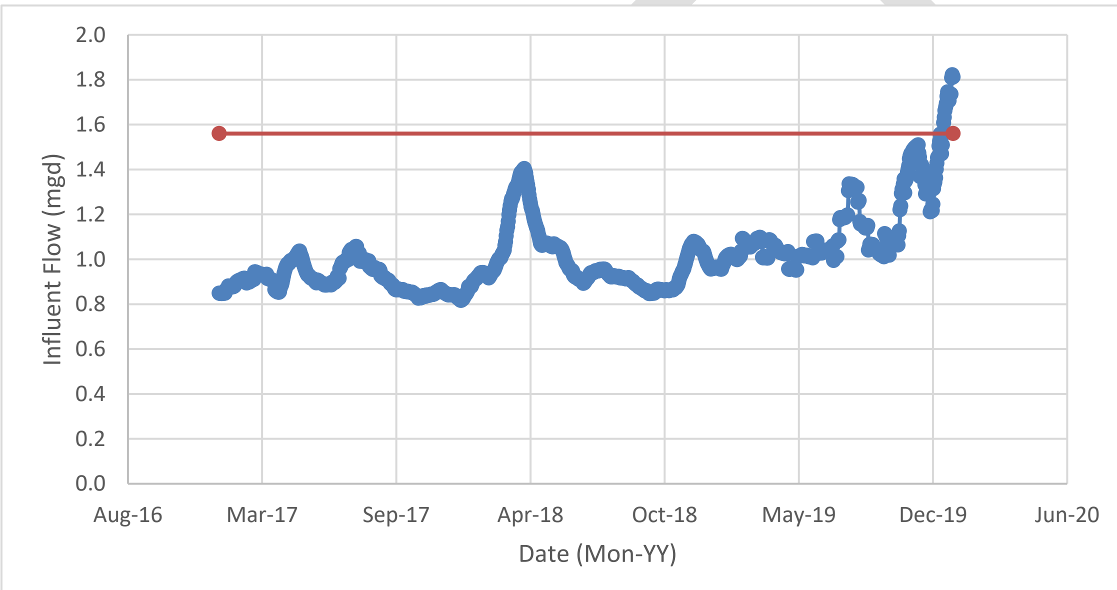
Table 2.3 Existing Effluent Limits

Effluent Characteristic	Effluent Limit	Limit Time Period
Flow	1.56 mgd	Average Monthly
BOD_5	10 mg/l	Average Monthly
TSS	10 mg/l	Average Monthly
pH Range	6.5 to 8.5	Daily
Fecal Coliform	14 cfu per 100 ml	Average Monthly
Enterococci	35 cfu per 100 ml	Average Monthly
Total Nitrogen (April 1 – October 31)	4 mg/l	Average Monthly

Effluent Characteristic	Effluent Limit	Limit Time Period
Total Nitrogen (November 1 – March 31)	Report	Average Monthly
Total Phosphorus (April 1 – October 31)	0.2 mg/l	Average Monthly
Total Phosphorus (November 1 – March 31)	Report	Average Monthly
Total Copper	Report	Average Monthly
Ammonia-Nitrogen	Report	Average Monthly

2.2 Analysis of flow data

The existing and projected flows at the facility were analyzed in the Memorandum of Capacity at the Wareham Water Pollution Control Facility, which is included in Appendix A. The WPCF has been experiencing a trend of increasing influent flows in the past years. The following figure shows the rolling 30-day average influent flow rate for 2017 through 2019.



Notes:

1. *The red horizontal line shows the 1.56 mgd permitted effluent discharge rate.*
2. *While the chart seems to show that the plant did not exceed its capacity and require discharges during spring 2018, the plant did need to discharge flow because there where consecutive days where the capacity was exceeded but the average 30-day flow was still under the permitted flow.*

Figure 2.2 Average 30-Day Rolling Flow for January 2017 through December 2019

In the last three years (2017 through 2019) the influent flows have not exceeded the permit on a rolling annual basis. In the month of December 2019, the average influent flow rate was 1.82 mgd. This flow exceeded the monthly reporting value of 1.56 mgd and a letter was sent from the WPCF to the State notifying them.

The maximum average rolling 365-day influent rate from the past three years was 1.18 mgd in 2019. This flow represents 76 percent of the permitted discharge rate. When the 365-day rolling average of the flow exceeds 80% of the permitted discharge rate, the NPDES permit requires an action plan to be submitted to the MassDEP. This action plan must be submitted to the MassDEP by March 31 of the calendar year following the 80% exceedance. This action plan must detail further flow increases and how the WPCF will maintain compliance with the permit's effluent load and flow limits.

While the flow to the facility has remained below the discharge permit limit, the flow memo determined two main problems with the flows to the facility—the peak influent flow rates and the committed future flows.

In the last few years, the facility has had multiple instances where the peak influent flows have exceeded both the secondary treatment capacity and storage volume of the existing equalization basins, leading to a non-permitted diversion of flow. The facility is currently constructing two additional equalization basins to help contain flow during peak influent flows. The additional equalization basins (Basin 3 and 4) are expected to be completed in the summer of 2021.

The Town has committed to allowing for future increases in flow from a number of parcels. Although these flows are not depicted in current (2020) flow data they need to be accounted for when planning for future flows and in analyzing the permit. The committed flows were presented at the Board of Selectmen presentation on February 11, 2020. When the committed future flows are added to the average annual influent flow rate for 2019, the influent flow rate would increase to 1.45 mgd. The flow rate of 1.45 MGD represents 93% of the permitted effluent discharge rate. This flow exceeds the 80% threshold, triggering a plan of action to be submitted to the MassDEP.

In addition to the facility's committed flows, the Town of Wareham has also allowed a new development, A.D. Makepeace, to connect to the sewer collection system. A.D. Makepeace has been allowed to contribute their full buildout flow through the Town's existing sewer connections. A.D. Makepeace has indicated that their future flows will likely be an additional 500,000 gpd. The timeline for when the A.D. Makepeace flow would be added is not definite. However, the addition of this flow to the Town's existing committed flows would be a total flow of 1.95 mgd. Influent flows of 1.95 mgd would exceed the WPCF's discharge limit and put the facility at 98% of its design maximum flow capacity.

2.2.1 Future Flows

The Town has an existing Comprehensive Wastewater Management Plan (CWMP) that is nearly 20 years old. A CWMP update was commissioned but has not yet been completed. A CWMP report is the manner in which future flows are estimated. This report needs to be completed so that planning can commence to accommodate future flows which have been committed beyond the permitted capacity of the facility.

2.3 Analysis of Influent Loads

As part of this project, influent concentrations and loads were analyzed to determine whether the current loads the facility is experiencing are within the design criteria. The facility typically measures influent BOD concentrations two to three times a week but does not regularly measure for influent nitrogen or influent phosphorus. It is recommended that the WPCF measure influent nitrogen and phosphorus concentrations weekly or multiple times per week in the future to help fine tune the operation of the facility.

For this project, the facility processed 10 influent nitrogen and phosphorus samples over the course of October and November 2020, four of these days aligned with BOD influent sampling. The TN:BOD ratio and the TP:BOD ratio from the sampling dates were used in conjugation with the average influent BOD sample data from 2017-2020 to estimate average nitrogen and phosphorus influent concentrations over the same time period. The estimated average concentrations were compared to typical values seen within industry standards. The estimated average concentrations are listed in the following table with the nutrient strength of the influent.

Table 2.4 Estimated Influent Values From Data

	Average Influent Concentration (mg/L)	Average Influent Loads (lbs/day)	Strength of Influent
BOD	259	2279	Medium
Nitrogen	38	332	Medium
Phosphorus	15	134	Strong

Regular testing of influent nutrient concentrations would be useful to measure the performance and capacity of the WPCF.

2.3.1 Future Influent Loads

The future influent loads for this facility will depend on the completion of the CWMP noted above in Section 2.2.1.

2.4 BioWin Model of Secondary Process

The secondary treatment process was modeled using a wastewater process modeling software called BioWin produced by EnviroSim Associates LTD. The model was created and run to validate the design criteria of the facility. The model was then run with actual data from the facility to confirm that the flows and loads the facility is experiencing can be treated within the design parameters of the facility and meet the discharge limits. The model, as seen in the figure below, showed that average and maximum monthly influent flows and loads could be treated within the existing secondary treatment process.

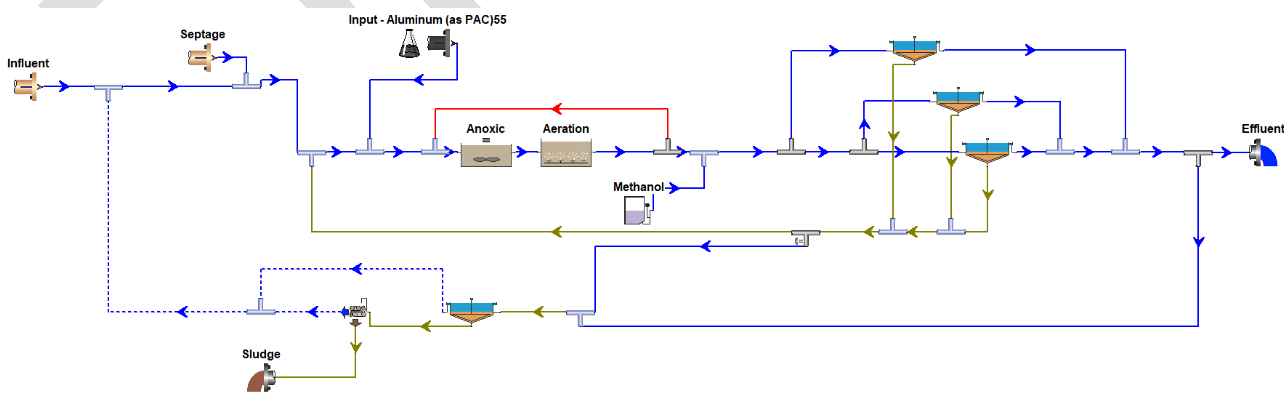


Figure 2.3 BioWin Model

Based on the estimates made from influent nutrient data and the flow data, the model run at maximum month conditions estimated that the loading capacity of the facility is between 60-70% of the design capacity. Additional data would help provide a more accurate estimate of the facility's current loading capacity.

2.5 Current Capacity

Based on flow and load data that was analyzed for this project, the current flow is approximately 70 to 80 percent of the facility's flow design capacity and the current load is approximately 60 to 70 percent of the facility's load design capacity. These values are based on limited data, estimates and models. Additional data would help improve the precision and accuracy of these estimates and could help fine tune the operation of the facility. The total remaining capacity of a wastewater treatment facility is determined by evaluating both the flows and loads. Based on an analysis of the Wareham WPCF influent flows and loads, flow rates are the limiting factor in determining the additional capacity of the facility. Therefore, the WPCF is estimated to be at 70 to 80 percent of the facility's design capacity. While this analysis shows that overall there is additional capacity in the facility, it should be noted that the facility has been experiencing capacity issues during peak flow events and the Town has committed flows above the facility's design capacity.

2.6 Permit Violations

Since 2017, the EPA has recorded the following permit violations at the Wareham WPCF, as shown in the following table.

Table 2.5 Permit Violations

Violation	Date
Flow	December 2019
Coliform, fecal general	July 31, 2019
Coliform, fecal general	August 31, 2019
Phosphorus	August 2019
Nitrogen	April 2017

3. Criticality Analysis Methodology

A criticality analysis is a decision-making tool that can be used to prioritize projects. It outlines capital projects recommended to maintain the existing level of service for the Town's infrastructure. No cost considerations are included for potential improvements required to meet the following:

- a more stringent effluent permit in the future,
- an expansion of capacity, or
- for improvements to existing infrastructure such as flood-proofing infrastructure to adapt to recently redefined Federal Emergency Management Agency (FEMA) floodplain definitions or upgrading a room/building to meet updated Building Code requirements.

This report also does not take into consideration any future expansions or upgrade plans for the facility. It is recommended that the Town review any future expansion plans with the recommendations of this report in order to determine the most cost-effective approach to meet both objectives for major equipment. For example, if a planning effort shows that a pumping station's flow is expected to increase substantially in the future, the future flow should be considered in the sizing of the replacement equipment (instead of replacing the equipment in-kind).

The design life of mechanical equipment is typically 20 years. The design life of concrete structures (buildings and tanks) is assumed to be a minimum of 50 years.

A criticality analysis is conducted by establishing a rating for three variables:

- Likelihood of Failure (LoF)
- Consequence of Failure (CoF)
- Risk Assessment Rating

The methodology used to determine each variable is described in this section.

3.1 Likelihood of Failure (LoF)

LoF is determined by considering both the condition and performance of existing equipment.

3.1.1 Condition Assessment

Knowledge of the remaining life of an asset allows a facility to make a sound decision related to rehabilitation options and the timing of replacements. The challenge for most facilities is to spend less time on reactive maintenance and more time on preventative maintenance. When work can be planned, the cost of maintenance has been shown to be less.

Condition issues exist if the asset currently operates sufficiently but either the critical equipment or structure is aged or in a deteriorated state. For this study, the design life of mechanical equipment is considered to be 20 years and the design life of concrete structures and underground pipes is a minimum of 50 years.

The criteria used in the condition assessment is outlined in Table 3.1.

Table 3.1 Condition Assessment

Rating Guidelines		
Condition Score	Condition Description of Asset	Range of Remaining Life
1 – Excellent	Asset is like new, fully operable, and well maintained.	80 to 100% remaining life left
2 – Good	Asset is sound and well maintained but may be showing some signs of wear.	55 to 80% remaining life left
3 – Moderate	Asset is functionally sound, showing normal signs of wear relative to use and age.	25 to 55% remaining life left
4 – Poor	Asset functions but requires a sustained high level of maintenance to remain operational.	10 to 25% remaining life left
5 - Failing	Effective life exceeded and/or excessive maintenance cost incurred.	10% or less remaining life left

3.1.2 Performance Assessment

Performance issues exist if the asset is either unable to sufficiently meet a level of service or if extraordinary means are necessary to keep it working properly to meet a level of service. Performance issues were noted during site walk-throughs and/or during discussions with WPCF staff. The criteria used for the performance assessment is outlined in Table 3.2.

Table 3.2 Performance Assessment

Rating Guidelines	
Performance Score	Performance Description of Asset
1 – Excellent	Asset consistently performs at or above required design standard and performs at full efficiency.
2 – Good	Asset is performing at required design standard. Efficiency of equipment may be slightly diminished.
3 – Moderate	Asset meets basic design standards but may require regular maintenance or other measures to perform at a high level. Asset has minor failures or diminished efficiency and some performance deterioration. Likely showing modest, increased maintenance and/operations costs.
4 – Poor	Asset cannot meet all required design standards (e.g. cannot meet peak conditions). Significant operational maintenance or other measures are required to sustain performance. Near-term scheduled rehabilitation or replacement needed.
5 - Failing	Asset cannot meet the required design standard. Immediate replacement or rehabilitation is needed.

3.1.3 LoF Ranking

After both a condition and performance score have been assessed, the higher of the two rankings is used as the LoF. For example, if a piece of equipment was installed a year ago (condition assessment rating of 1) but requires significant maintenance (performance assessment rating of 4), the LoF is rated as 4. For the WPCF, since the majority of the equipment is past its useful life and has the same condition rating (5), the performance assessment was used to sub-rank equipment with the same condition rating.

3.2 Consequence of Failure (CoF)

The criticality of a piece of equipment is determined by the CoF. Criticality can be significant in several areas including health and safety of personnel, meeting the facility's discharge permit limits, treatment process viability, damage to other assets that rely on the equipment, and the cost for rehabilitation or replacement. The guidelines used to establish a CoF are outlined in Table 3.3.

Table 3.3 CoF Guidelines

Rating	Guidelines	WWTF Examples
1 – Negligible	Failure of asset will not result in significant consequential damages. Alternative systems or processes are in place to allow the asset to be out of service for an extended period until repair/replacement, with negligible impact on performance or safety.	Failure of a plant water system if the facility can use potable water backup for all processes; or failure of an automatic control system for a process normally operated in manual mode; or failure of an HVAC system in a non-occupied building without cold or heat-sensitive equipment.
2 – Marginal	Failure of asset may result in minor to moderate consequential damages, minor violations, inconvenience to personnel, inability to meet required design standard, or some adverse publicity or complaints. Often used for assets which can be repaired or replaced prior to critical consequences occurring.	Failure of gate/valves infrequently used; or failure of an HVAC system in a normally occupied building such as a Control Building; or failure of instrumentation used for monitoring only where manual samples could be used instead; or failure of an odor control system which could lead to some complaints but not major negative publicity.
3 – Critical	Failure of asset likely to result in injury, significant permit violation, significant consequential damages, or significant negative publicity.	Failure of an influent pumping system, resulting in sewage overflow until a bypass system can be put in place; or failure of treatment processes which could result in effluent permit violation.
4 - Catastrophic	Failure of asset likely to cause serious injury or loss of life, long-term environmental damage, or sudden failure of other significant assets.	Failure of the main power distribution system, resulting in loss of entire treatment facility operation; or failure of gaseous chlorination system which could cause serious injury or loss of life.

3.3 Prioritization of Needs Using the Risk Assessment Matrix

The concept of risk can be used to prioritize scarce capital and operating budgets. The risk of not meeting the established level of service for a portion of the infrastructure is a function of the probability the equipment will fail (LoF) and the consequence of it failing (CoF). The two variables are used to assign a risk rating from the risk assessment matrix, shown in the following table. The risk assessment matrix allows the Town to develop a plan to prioritize projects by the risk they pose.

Table 3.4 Risk Assessment Matrix

CoF Rating → ↓ LoF Rating	Negligible (1)	Marginal (2)	Critical (3)	Catastrophic (4)
Failing (5)	Medium	High	Very High	Very High
Poor (4)	Medium	High	Very High	Very High
Moderate (3)	Low	Medium	High	Very High
Good (2)	Low	Low	Medium	High
Excellent (1)	Low	Low	Medium	High

4. Unit Process Evaluation

Wastewater and sludge treatment unit processes at the WPCF have been evaluated based upon current industry design practice and guidelines. The guidelines and design standards used are outlined in Section 1.4 References and Guidelines. The evaluation is presented in four sections as described below.

- Description—This section describes the process under evaluation. This includes a description of location of the process in the treatment process as well as the use of the process.
- Evaluation—This is a comparison of the engineering standards and guidelines with the existing process.
- Operational Issues (Performance and Condition)—This section describes the operational issues with the processes. The information in the section was obtained through site visits as well as comments from the Town. This section includes descriptions of both the performance and condition of the process.
- Recommendations—This section describes upgrades and improvements that are recommended. The recommendations are made using the information from the evaluation of the guidelines and regulations and the operational issues.

4.1 Preliminary Treatment

4.1.1 Description

Influent raw wastewater flows from two 18-inch influent lines to the influent box where it is combined with biofilter leachate and on-site sanitary flow. The influent box is a concrete structure 11.3-feet long by 10-feet wide by 11.9-feet deep, with a total volume of 10,000 gallons. From this influent box, flow continues through the influent channel, passing through a 9-inch Parshall flume and entering the north screenings channel inside the Headworks Building, passing through a rotary fine screen. The rotary fine screen is a 47-inch diameter rotary screen with ¼-inch openings. The screen is a Lakeside rotary drum screen, Model 47FS-0-250-93. Flow passes through the screenings basket, holding back screenings and allowing flow to continue through the channel. As the screenings being held back builds up and restricts the free flow area of the screen, the upstream liquid level rises and once it reaches a preset level, the concentric rotating rake is started. The rake is attached to the transport screw and rotates with the screw. The teeth of the rake pass through the bars of the screen and collect the screenings. The screenings are dropped into the screw conveyor trough as the rake reaches the topmost position. The screenings are conveyed up, washed, and dewatered by gravity due to the incline of the conveyor. The dewatered screenings get compacted and are dropped into dumpster for off-site removal. The south channel is a bypass channel equipped with a 30-inch-wide manual bar rack with 1½-inch spacing, allowing flow to bypass the rotary fine screen. Bypassing the north channel can be accomplished by closing slide gate SG-101 and opening gates SG-102 and SG-103 to allow equipment isolation.

After the Parshall flume, potassium permanganate is injected into the flow for odor control.

From the screenings channel flow continues below-grade, exiting the Headworks Building and into the influent channel of the vortex grit chamber. The vortex grit system is a 9-foot diameter and

10.75-foot deep WesTech unit, designed for a peak flow of 5.6 mgd. The grit influent channel has slide gates to allow for bypass of the tank. The vortex grit chamber has adjustable, rotating paddles that augment the spiraling flow to create a mechanically induced vortex which settles the grit and transports it to the center opening of the fixed floor plate for collection in the lower chamber and returns the lighter organic particles to the main flow. From the center opening, the grit is lifted by an airlift pump that discharges the grit to a classifier in the headworks building where the grit is washed and dewatered.

A refrigerated influent sampler draws from the influent channel after grit removal.

Flow continues through a 14-inch sluice gate in the equalization basin flow splitter box through a 14-inch line to the influent mixing box. Flow is measured by a magnetic flowmeter on this 14-inch line.

Flow in excess of the secondary treatment capacity is diverted through the equalization basin flow splitter box over a motorized weir slide gate to the two equalization basins. The design intent is to allow 2.00 mgd to the secondary treatment process and all overflow to the equalization basins. However, the weir slide gate is adjustable which allows the facility to adjust the amount of flow going to the secondary treatment process and equalization basins. The contents of the equalization basins are kept mixed and aerobic by a coarse bubble diffused air system that is supplied air by three equalization basin blowers.

During low flow periods, equalized wastewater is conveyed by the two equalization basin pumps through a 16-inch line to the influent mixing box where it is combined with pretreated septage from the septage receiving plant, filter backwash from the denitrification filters, process tank drainage and supernatant, return activated sludge, internal recycle mixed liquor, filtrate from the gravity belt thickener, and decant from the sludge storage tanks. The resulting mixed liquor proceeds through a 24-inch line to the anoxic tanks.

Table 4.1 Preliminary Treatment Design Criteria

Equipment	Maximum Capacity (mgd)
Rotary Fine Screen	6.85
Vortex Grit Chamber	5.6

4.1.2 Evaluation

The screen and grit vortex chamber were both placed into service in 2001.

Based on design standards the upstream approach of the Parshall flume should be 10 times the throat width. For peak hour flows of 5.6 mgd the 9-inch Parshall flume is adequately sized. The rotary fine screen is sized for flows of 6.85 mgd, which is adequately sized for the 2.0 mgd peak hour flows that the secondary treatment process is sized for.

4.1.3 Operational Issues

The following operational issues were noted by the Town:

- Backup in the screenings channel causes the Parshall flume to provide incorrect readings.
- Grit vortex system does not operate properly, and grit deposits are noted further downstream of the unit.

4.1.4 Recommendations

The following table displays recommended improvements to the preliminary treatment system.

Table 4.2 Preliminary Treatment Recommendations

Issues	Recommended Improvements
Backup in the screenings channel causes the Parshall flume to provide incorrect readings.	Consider replacing the Parshall flume and evaluating the restrictions in the screenings channel.
Grit vortex system does not operate properly, and grit deposits are noted further downstream of the unit.	Evaluate the grit vortex system and consider replacing the system or any parts that need repair.

4.2 Equalization Basins

4.2.1 Description

After flowing through the preliminary treatment processes, flow in excess of the secondary treatment capacity (design intent of 2 mgd) is diverted to the equalization basin flow splitter box over a motorized weir slide gate and into one of the two equalization basins through an 18-inch DIP. The contents of the equalization basins were designed to be kept mixed and aerobic by a coarse bubble diffused air system that is supplied air by three equalization basin blowers. This coarse bubble diffuser system is no longer in use.

The facility has been designed for a peak hour flow of 2 mgd. The splitter box is intended to limit the flow to advanced treatment to not more than its design capacity of 2.0 mgd. When the flow is above 2.0 mgd it is diverted to the Equalization Basins for temporary storage to be bled back into the diversion chamber downstream of the grit vortex system for treatment once inflows diminish to below 2.0 mgd. This is accomplished by the flow meter on the 14-inch line out of the splitter box to the Anoxic Selectors, controlling the weir gate WSG-300 lowering it as necessary to maintain 2.0 mgd through the meter, with the remainder of the flow going over the weir to the equalization basins.

The equalization pumps are set to run automatically paced off the influent flow meter in order to maintain the 2.0 mgd flow.

Table 4.3 Equalization Basins

Number	2
Volume Each, mg	1.1

4.2.2 Evaluation

The equalization basins were constructed in the 2005 facility upgrade. They are each sized to hold 1.1 million gallons and temporarily store any flow from the collection system in excess of the design intent of 2.0 mgd. These basins are sufficiently sized to handle the daily peak flow events until the flow in excess of 2.0 mgd can be bled back into the secondary treatment system through the preliminary treatment effluent box. However, it has been noted that during wet weather events, due to inflow and infiltration both basins can fill quickly and reach their capacity. Thus, during wet weather events it is difficult to bleed excess flow back into the secondary treatment process readily enough to lower the basin water levels.

It is noted that the coarse air diffuser system is no longer operational and was taken offline as it was thought to add more odorous conditions to the head of the facility.

4.2.3 Operational Issues

The following operational issues were noted by the Town:

- Nearing capacity after peak wet weather events and not having the ability to bleed the flow back into the secondary treatment process as readily as would be required.
- The Town continues to receive odor complaints from adjacent residential neighbors.

4.2.4 Recommendations

The following table displays recommended improvements to the aeration tanks that are recommended.

- Odor control via chemical addition injection at the pumping station.
- Addition of fifth covered equalization basin to handle daily peaking events and minimize odor impacts from the uncovered equalization basins.
- Upgrade the facility and increase the capacity of the secondary and tertiary treatment to handle the peak influent flows.

Table 4.4 Preliminary Treatment Recommendations

Issues	Recommended Improvements
Backup in the screenings channel causes the Parshall flume to provide incorrect readings	Consider replacing the Parshall flume and evaluating the restrictions in the screenings channel.
Grit vortex system does not operate properly, and grit deposits are noted further downstream of the unit	Evaluate the grit vortex system and consider replacing the system or any parts that need repair.

4.3 Anoxic Tanks

4.3.1 Description

After preliminary treatment forward flow (flow not sent to equalization basins) passes through the preliminary treatment system and continues to one of two anoxic tanks via a 24-inch ductile iron gravity pipe. Raw wastewater is combined in the mixing box with pretreated and equalized septage from the septage equalization facility, filter backwash from the denitrification filters, process tank drainage and supernatant, return activated sludge, equalization basin flow and internal recycle mixed liquor, filtrate from the gravity belt thickener, and decant from the sludge storage tanks. The resulting mixed liquor proceeds to the anoxic tanks. Each of the two anoxic tanks is divided into three zones, each with a floating mixer to keep the solids mixed into suspension but not enough mixing to provide additional oxygen. The anoxic zones are unaerated to keep the dissolved oxygen levels below 1 mg/L. This allows for low enough dissolved oxygen where nitrate nitrogen can be removed by denitrification. The concentration of mixed liquid and suspended solids must be kept in balance and the pH of the anoxic zone should be close to neutral (7.0).

Table 4.5 Anoxic Tanks

Number	2
Tank No.	1
Dimensions (L x W x D), ft	52 x 24.75 x 12.1
Volume, gallons	116,640
Zones per Tank	3
Tank No.	2
Dimensions (LxWxD), ft	52x24.75x12.1
Volume, gallons	116,640
Zones per Tank	3

Table 4.6 Anoxic Tank Mixers

Number	6
Manufacturer	U.S. Filter
Type	Aqua-Lator DDM Mixer
Model	312-SF
Motor Manufacturer	U.S. Motors
Horsepower	3
RPM	1,200
Electrical Service Volts, Ph, Hz	480,3,60

4.3.2 Evaluation

The anoxic tanks were added in the 2005 upgrade. The structure is within its design life, but it is recommended that the tanks be taken down for cleaning and inspection.

4.3.3 Operational Issues

The following operational issues were noted by the Town:

- The concrete of Aeration Tank No. 2 shows signs of pitting and corrosion. The spray wash walls have shown signs of leakage and infiltration.

4.3.4 Recommendations

The following table displays recommended improvements to the aeration tanks.

Table 4.7 Aeration Tank Recommendations

Issues	Recommended Improvements
Mechanical equipment is nearing the end of its life and the condition of the structure is unknown.	Consider taking down the anoxic tanks for cleaning and inspection to assess the structure's condition.

4.4 Aeration Tanks

4.4.1 Description

Flow from the anoxic tanks flows to one of three aeration basins. The aeration basins utilize a Modified Ludzack-Ettinger (MLE) process to biologically remove nitrogen. Aeration is part of the secondary treatment process using activated sludge. The process is based on pumping air into a tank, which promotes the microbial growth in the wastewater. Bacteria uses the supplied oxygen to break down the organic matter in the wastewater. The MLE process reduces the carbonaceous biological oxygen demand (CBOD) and suspended solids (SS), as well as providing ammonia oxidation and nitrate reduction through nitrification and denitrification. Ammonia is converted to nitrate in the activated sludge process and subsequently removed as nitrogen gas through denitrification in the anoxic tanks with a high recycle rate. Diffused aeration provides the air-water interface that allows the transfer of oxygen to the mixed liquor. The diffused aeration also provides the mixing and turbulence to keep the activated sludge floc from settling and ensures that the raw wastewater comes into close contact with the activated sludge. Three aeration blowers supply air to the aeration system.

The facility has three aeration basins; two of the basins were constructed in the 1970s and retrofitted during the 2005 upgrade project and the third basin was constructed during the 2005 upgrade.

Table 4.8 Aeration Tanks

Number	3
Dimensions (L x W x D), ft	96 x 48 x 13.33
Total Volume, mg	1.38
MLSS mg/L	4,400

Number	3
Aerated SRT, (summer) days	9.8
Aerated SRT, (winter) days	7

4.4.2 Evaluation

Aeration Tank Nos. 1 and 2 are original to the facility and Aeration Tank No. 3 was added in the 2005 upgrade. The equipment in Aeration Tank Nos. 1 and 2 were replaced in the 2005 upgrade. Aeration Tank No. 1 was structurally rehabbed in the last five years. However, the condition of the original Aeration Tank No. 2 is unknown.

As indicated in Section 2, the secondary treatment process was modeled using the wastewater process modeling software BioWin. The model used influent data from the WPCF and showed that the process was close to 80 to 90% capacity with regards to nutrient loading. Based on this modeling, the aeration tanks are adequately sized for current loads. Once future flows are known, the future loads can be assessed to determine if the aeration tanks are sized to be able handle future loads.

4.4.3 Operational Issues

The following operational issues were noted by the Town:

- The concrete of Aeration Tank No. 2 shows signs of pitting and corrosion. The spray wash walls have shown signs of leakage and infiltration.

4.4.4 Recommendations

The following table displays recommended improvements to the aeration tanks.

Table 4.9 Aeration Tank Recommendations

Issues	Recommended Improvements
Possible degradation and corrosion of Aeration Tank No. 2.	Take Aeration Tank No. 2 down for inspection and address any pitting, corrosion, and exposed rebar.

4.5 Secondary Clarifiers

4.5.1 Description

The mixture of microorganisms and wastewater (mixed liquor suspended solids (MLSS)) is discharged from the aeration tanks to the secondary clarifiers to allow for separation of the liquid and solids. The secondary clarifiers separate biological floc from the treated liquid waste stream. The solids that settle to the bottom are returned to the influent mixing box and ultimately to the aeration tanks as return activated sludge (RAS) providing adequate levels of microorganisms for continuous treatment of the suspended and soluble organic solids and nitrification. Sludge in excess of what is required to be returned to the aeration basins is removed from the stream as waste activated sludge (WAS). The remaining wastewater, from which the solids have settled out, flows over weirs out the top of the clarifier and proceeds to filtration and disinfection.

The facility has three circular secondary clarifiers. The first and second secondary clarifiers were constructed in the 1970s and the third clarifier with associated equipment was constructed during the 2005 upgrade. The original clarifiers' equipment dates to the original construction and is past its useful design life.

Table 4.10 Secondary Clarifiers

Number	1 (3)
Manufacturer	WesTech
Type	Circular, center column feed
Diameter, ft	55
Sidewater Depth, ft	10.17
Unit Surface Area, sq.ft.	2,375.7
Total Surface Area, sq.ft.	7,127
Overflow Rate, Maximum Daily Flow, gpd/sq.ft.	281
Solid Loading Rate, Maximum Daily Flow, ppd/sq.ft.	18
Motor Manufacturer	Sterling Electric
Horsepower	1/4
RPM	1,720
Electrical Service Volts, Ph, Hz	480, 3, 60
Speed Reducer Manufacturer	SM-Cyclo
Model	CVVJS-4190DAY-7569:1
Main Gear Drive Manufacturer	WesTech

4.5.2 Evaluation

Two of the three Secondary Clarifiers are original to the facility and the third clarifier was added in the 2005 upgrade. Based on TR-16 design guidance documents, the secondary clarifiers have a sidewater depth that is lower than the recommended 12-feet of sidewater depth for up to 40-foot diameter secondary clarifiers.

The state point analysis was conducted to assess the solids-flux limitations for the sludge in the secondary clarifiers. At the design conditions of 4,400 mg/L of MLSS and 2.0 million gallons of flow as a peak flow for the clarifiers, the state point and the underflow rates are both below the settling curve. This analysis confirms that under the design conditions the clarifier surface area is sufficient to allow settling.

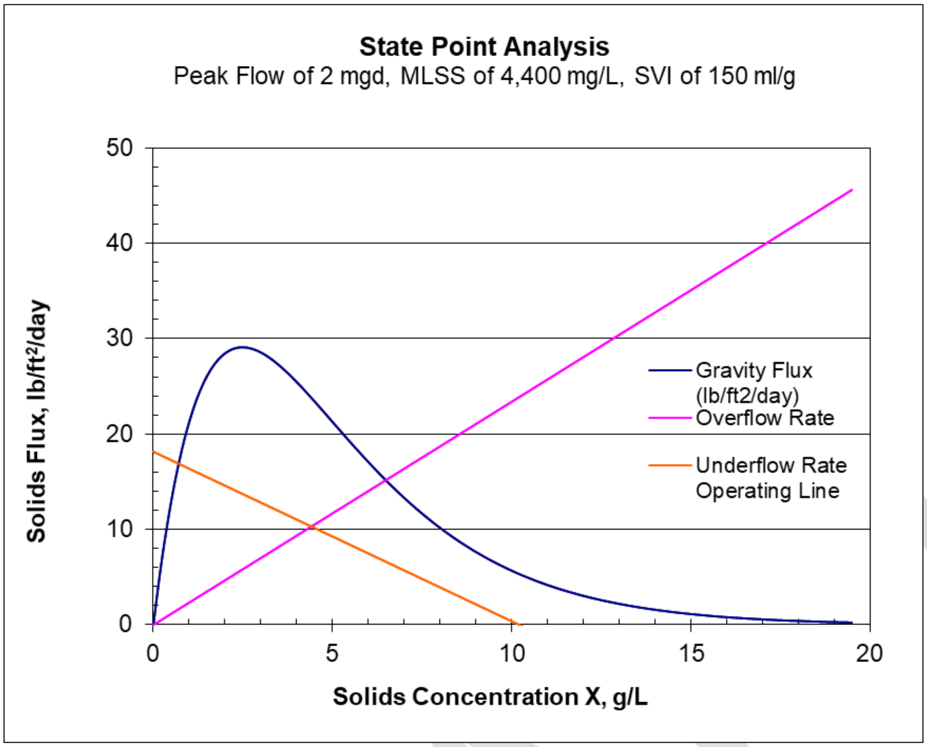


Figure 4.1 State Point Analysis at Design Conditions

In order to ensure proper settling, the entirety of the underflow operating line to the right of the state point should be completely under the settling curve. As can be seen from the figure below, once the MLSS concentration rises to 5,900 mg/L, the underflow rate operating line to the right of the state point is above the settling curve. This is indicative of the clarifiers being at risk of settling failure.

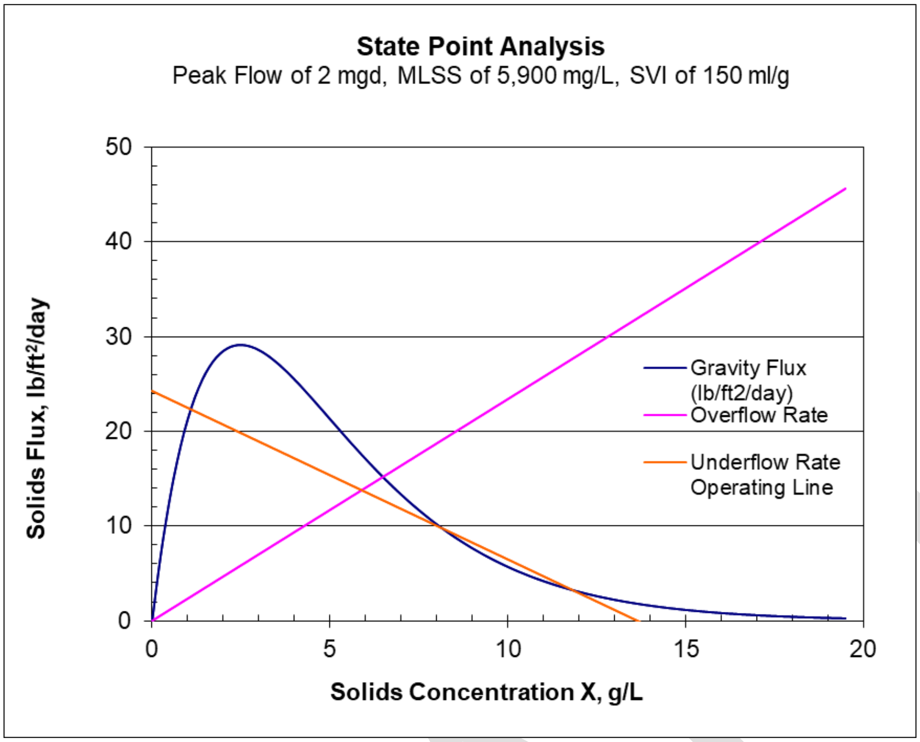


Figure 4.2 State Point Analysis at Risk of Settling Failure

Based on data over the past couple years, in approximately December of 2019 the MLSS concentration rose to a level of about 7,000 mg/L. Shortly thereafter, the solids percentage dropped off and the Town noted issues with getting the solids to settle and thicken. As can be seen from the figure below, at an MLSS of 7,000 mg/L both the state point and the underflow rate are above the settling curve. Having the state point above the settling curve, the clarifier is failing in clarification and the solids don't have enough time to settle before flowing over the weirs. In addition, the underflow line being above the settling curve is indicative of the clarifier failing at settling and the sludge not being removed fast enough.

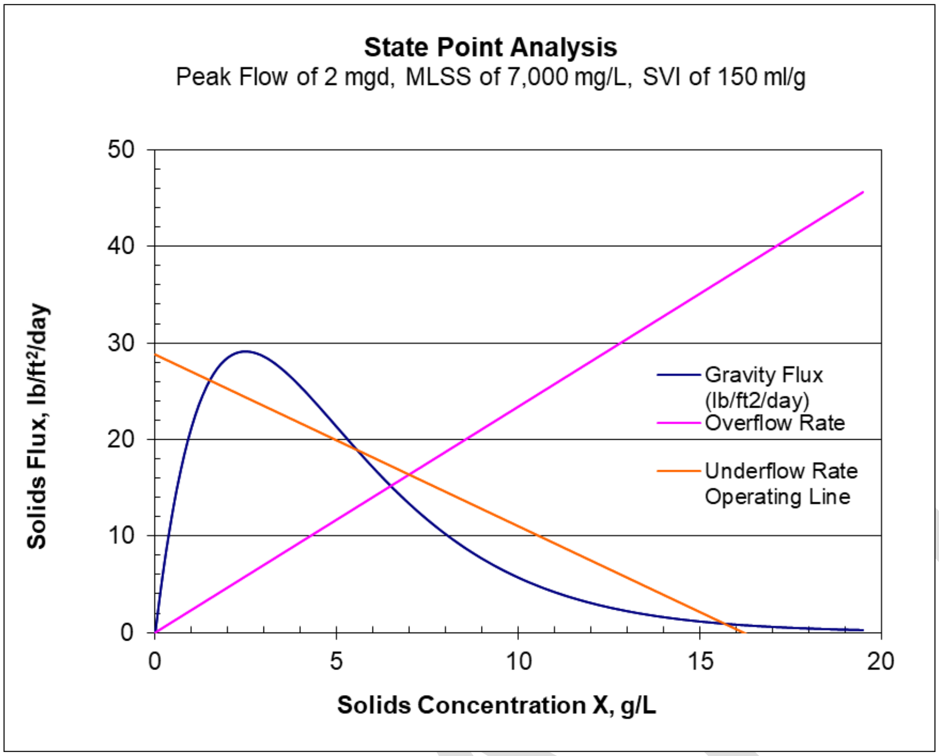


Figure 4.3 State Point Analysis at December 2019 Conditions

The above figures show that the existing clarifiers will fail to settle properly under high MLSS concentrations (at least over 6000 mg/L). This high concentration is typically seen when the facility is lacking storage for sludge possibly due to issues with thickening sludge; this is discussed in more detail in a later section.

4.5.3 Operational Issues

The following operational issues were noted by the Town:

- The Town has noted difficulty in getting the solids to settle in the secondary clarifiers. There have been occurrences where the sludge blanket is 9-feet of the 10-foot sidewater depth. This causes solids to overflow the clarifiers and enter the denitrification filters which causes those to clog more quickly. As the filters clog, they enter the backwash cycle and reduce the capacity of the filters which causes a negative cycle limiting the ability of the facility to pass the full flow through to disinfection and out to the river.

4.5.4 Recommendations

The following table displays recommended improvements to the secondary clarifiers that are recommended.

Table 4.11 Denitrification Filter Recommendations

Issues	Recommended Improvements
Solids overflow clarifier weirs, clogging filters.	Add equalization capacity to temporarily store additional secondary effluent when filters capacity is constrained due to backwash.
Inadequate secondary clarifier sidewater depth.	De-rate the existing clarifiers and consider additional clarifiers with increased sidewater depth meeting current design guidance.

The addition of equalization capacity to store secondary effluent has been pursued and the facility is currently undergoing construction of Equalization Basins No. 4 and 5 to help with these emergency situations in the short-term.

4.6 Denitrification filters

4.6.1 Description

The Town has three Leopold denitrification filters. Flow that passes over the secondary clarifier effluent weir continues by gravity through a 20-inch ductile iron pipe to these filters where suspended solids and remaining nitrate nitrogen that was not removed in the anoxic selectors is removed from the clarified effluent stream.

Periodically the filters are backwashed to release solids that have collected in the filter, prevent short-circuiting and purging of retained gases. Once the accumulation of material reaches the filter loading rate, the filters backwash independently of each other. The backwash frequency is approximately every 20 hours. The filter design parameters are shown in the table below.

Table 4.12 Deep Bed Filters

Quantity	3
Manufacturer	Leopold
Type	Gravity Deep Bed
Length	9.5 feet
Width	16 feet
Media Depth	6 feet
Surface Area	152 Sq. ft.
Total Surface Area (3 filters operating)	456 sq. ft.
Hydraulic Loading Rate Average (3 filters operating) Maximum (3 filters operating)	 2.3 gpm/sq.ft. 3.0 gpm/sq.ft.

4.6.2 Evaluation

The filters, including structure and equipment, was installed in the 2005 upgrade of the facility. The filters are designed to handle an average hydraulic loading of 1.5 mgd and a maximum hydraulic loading of 2 mgd with all filters operating. Updated guidelines from TR-16 (2016) state that filtration systems should be designed to handle peak hour flows with one unit in backwashing mode. Using the TR-16 guidelines the capacity of the existing filters is 1 mgd at average hydraulic loading and 1.3 mgd at maximum hydraulic loading conditions. These rates are below the design flowrates of the secondary treatment, suggesting that bottlenecks occur at the filters. The following table shows the design capacity for the processes under various conditions.

Table 4.13 Design Capacity for Filtration and Secondary Treatment

	Existing Secondary Treatment Design Capacity	Existing Filter Design Capacity with All Filters Operating[3]	Existing Filter Capacity with one Filter in Backwash[4]
Average / Annual Average Day[1]	1.56	1.5	1
Maximum / Peak Hour[2]	2	2	1.3

Notes:
1. Filtration design uses "Average" hydraulic loading; Secondary Treatment uses "Annual Average Day" flowrate.
2. Filtration design uses "Maximum" hydraulic loading; Secondary Treatment uses "Peak Hour" flowrate.
3. Existing Filter Design Capacity with All Filters Operating does not meet TR-16 guidelines.
4. Existing Filter Capacity with one Filter in Backwash is according to TR-16 guidelines.

GHD proposes adding additional filtration capacity through additional downflow filters to meet the updated TR-16 guidelines.

4.6.3 Operational Issues

There were no specific operational issues noted by the Town. Since the filter equipment was installed in 2005, the equipment is three-quarters through its design life (of 20 years for equipment).

4.6.4 Recommendations

The following table displays recommended improvements to the denitrification filters that are intended for the next facility upgrade.

Table 4.14 Denitrification Filter Recommendations

Issues	Recommended Improvements
Existing filter capacity issues when one filter is backwashing mode.	Add additional filtration capacity to meet guidance document and have sufficient capacity with one filter in backwash mode.

4.7 UV Disinfection

4.7.1 Description

After passing through the denitrification filters, flow passes through one of three horizontal UV modules. Each of these banks contains four modules, with each module accommodating six lamps. UV disinfection provides broad-spectrum UV wavelengths to the wastewater, inactivating harmful microorganisms in prior to effluent discharge to the Agawam River. The UV disinfection system at the WPCF is a Trojan UV3000 Plus horizontal system.

Table 4.15 Ultraviolet (UV) Disinfection System

Manufacturer	Trojan
Model	UV3000 Plus™
Number of Channels	1
Number of Banks	3
Number of Modules	4
Number of Lamps per Module	6
Total Number of Lamps	72

4.7.2 Evaluation

The existing system, including structure and equipment, was installed in the 2005 upgrade of the WPCF. The UV disinfection flow channel is rated for a peak flowrate of 2.0 mgd with a design UV transmittance of 65%, minimum at 253.7 nm. Under peak design two out of the three banks can be in operation and continue to provide proper disinfection. Per TR-16, it is recommended that a UV system be capable of delivering the design dose and disinfecting effluent at peak instantaneous flows with one bank of modules out of service. As such, the existing UV disinfection system is adequately sized for existing flows.

4.7.3 Operational Issues

The following operational issues were noted by the Town:

- During high flow events the effluent weir leaks and does not consistently restrict flow to the effluent permit limits.

4.7.4 Recommendations

The following table provides recommendations for improvements based on the Town's operational issues. Since the UV disinfection system works and is adequately sized there are no recommendations for replacement. Once future flows to the facility are known, the UV system will likely be undersized to meet peak flows with one module out of service. At that time the UV system should be upgraded.

Table 4.16 UV Disinfection Recommendations

Issues	Recommended Improvements
Inaccuracies with effluent weir operation during high flow events causing permit violations.	Fix leakage and hydraulics issues as required until future flow requirement is known.

4.8 Septage Receiving

4.8.1 Description

The Town of Wareham processes septage at their Septage Receiving station located at the Headworks Building. A large volume of septage as well as septage with strong characteristics up to 8,000 mg/L of BOD_5 and 30,000 mg/L of SS can be accommodated at the facility. The system consists of a keypad access system and truck hookup station to which septage haulers connect into a 4-inch inlet quick connect to a 4-inch discharge line. Activation of the septage receiving system requires the use of a key and personal identification number (PIN), with a maximum of 75 authorized discharges per PIN. The septage passes through a rock trap where large rocks and debris fall out of the liquids stream. The liquid continues to the septage receiving equipment, which consists of rotary fine screen to capture, wash, and dewater screenings (rags, plastics, and other debris). The screenings are transported into a container for disposal. The tank has an aerated grit chamber. The grit chamber air blower adds air through removable diffusers to the bottom of the tank to mix and aerate the septage and capture grit. A grit dewatering screw conveys, washes, and dewaters the captured grit. Grit from the screw is deposited in a grit container for transport off site. Two septage pumps convey treated septage to the septage equalization tank facility.

The following tables provide design parameters for the various equipment that makes up the Septage Receiving system.

Table 4.17 Septage Receiving System Design Data

Septage Flow, mgd Annual Average Day Maximum Day Peak Hour	1.56 2.00 2.00
Rotary Fine Screen Diameter, in	31
Fine Screen Maximum capacity, gpm	2060
Fine Screen Clear opening spacing, in	1/4
Fine Screen Motor, HP / RPM	2 / 1750
Grit Screw Diameter, in	8
Grit Screw Motor / HP / RPM	Reliance / 1 / 175
Grit Classifier Diameter, in	8
Grit Classifier Motor / HP / RPM	Reliance / 2 / 175
Grit Blower Manufacturer – Model	Dresser Roots – 22 URAI
Grit Blower Airflow, scfm (at 4.5 psig)	8
Grit Blower Motor / HP / RPM	Baldor / 3 / 3,560
Electrical Service, Volts / Ph / Hz	480 / 3 / 60

The septage that has passed through the septage receiving system is pumped to the septage equalization tanks. Septage withdrawal from these tanks is controlled automatically with a timer at regular intervals throughout the day. The equalization tanks allow the septage to equalize such that when flow is withdrawn and fed into the aeration basins, the impact of the septage BOD_5 and soilds loads on the secondary treatment process is minimized. The septage equalization facility consists of two sides, each having two covered equalization tanks, two submerged turbine tank aerators, two tank aeration rings, two tank aeration blowers and two septage transfer pumps. Air is normally supplied to the equalization tank to mix the contents and provide partial treatment. Each of the septage equalization tanks has a maximum capacity of 64,600 gallons. The bottom of each tank is sloped to the dividing wall between the pump room and the tanks. A 10-inch reinforced concrete roof slab is provided over each tank to prevent odors from being released to the atmosphere. Air from the headspace of each tank is conveyed through the Headworks Odor Control Biofilter.

The following table provides design data for septage equalization.

Table 4.18 Septage Equalization System Design Data

Number of Tanks	4
Tank Capacity (each), gallons	64,600
Blowers, number	4
Blower Manufacturer	Sutorbilt
Blower No. 1 Motor / HP / RPM	Lincoln / 20 / 1,750
Blowers No. 2-4 Motor / HP / RPM	Lincoln / 15 / 1,750
Submersible Aerators, number	4
Aerator Manufacturer, Model	Philadelphia, 3809 Q PTSS
Aerator Input RPM	1,200/900
Aerator Motor / HP / RPM	Reliance / 15/11 / 1,165/870
Transfer Pumps, number	4
Transfer Pump Manufacturer, Type	Komline Sanderson, Dual Plunger
Transfer Pump Motor / HP / RPM	Reliance / 15 / 1,765
Electrical Service, Voltz / Ph / Hz	480 / 3 / 60

4.8.2 Evaluation

The Septage Receiving facility is designed for a peak flow of 2.0 mgd. Currently, future incoming septage receiving is unknown. The facility is adequately sized for the current flows.

4.8.3 Operational Issues

The following operational issues were noted by the Town:

- Issues with the screen, including broken parts that need to be replaced.
- Septage equalization tank blowers are not mixing the septage properly.
- Sludge settling issues and process upset due to high concentrations and loads in the secondary treatment process.

4.8.4 Recommendations

The following table displays recommended improvements to the septage receiving facility that are intended for the next facility upgrade.

Table 4.19 Septage Receiving Recommendations

Issues	Recommended Improvements
Septage receiving screen is broken.	Replace screen on screenings unit.
Septage equalization tank blowers are not mixing the septage properly.	Inspect tank and diffuser system, evaluate condition, and consider replacing blowers.
Sludge setting issues and process upset due to loads in the secondary treatment process.	Pump screened and de-gritted septage directly into the solids handling process to reduce the additional nutrient loading in the secondary treatment process.

4.9 Sludge Processing

4.9.1 Description

The WPCF has four sludge storage tanks located at the northeast end of the Dewatering Building. The sludge storage tanks serve to store waste activated sludge (WAS) and scum prior to dewatering on the gravity belt thickener (GBT) or pumping to a sludge hauling tanker truck. The sludge storage tanks also hold thickened sludge from the GBT (conveyed by the thickened sludge transfer pumps) prior to pumping to a sludge hauling tanker truck. The sludge holding tanks act to increase digestion by increasing the contact time, thickening up the sludge by separating the liquid for the solid components using the means of settling, and acting as a storage container of the sludge before being sent for pressing disposal. Each sludge storage tank is equipped with a mechanical aerator, aeration ring, and an ultrasonic tank level transmitter. The sludge is kept mixed and aerated to maintain aerobic conditions in the tanks. The air space in the tanks is drawn through the dewatering biofilters for odor reduction.

The sludge storage tanks store thickened sludge from the GBT mixed with secondary scum prior to pumping to a sludge hauling tanker truck for offsite disposal. Under normal operation of the GBT, sludge is wasted directly to the equipment for thickening. If the GBT is not available, WAS can be sent to the sludge holding tanks for storage and later transferred to the GBT for thickening or pumped to a tanker truck.

Table 4.20 Gravity Belt Thickener Design Data

Manufacturer	Tiger Flow
Number of Pumps	3

4.9.2 Evaluation

The tanks and equipment were initially constructed in the 1970s and are past their useful design life. There is 10% remaining life on the concrete structures until they have reached 50 years of minimum design life, which results in a high LoF rating. Once the concrete structure reaches 50 years, it is recommended to conduct an evaluation and determine the expected remaining life.

Additionally, the Town has noted problems with sludge thickening and disposal. Data at the WPCF was analyzed between 2017 and 2020, including the mass of sludge disposal (in pounds), the mixed liquor suspended solids (MLSS) concentration, and the percent solids concentration of the thickened sludge coming off the gravity belt press. This data is shown below in the following figure. The figure also contains boxes that represent noted changes in the plant data as described below.

Box 1: From April 2017 through April 2018 the MLSS concentration was stable, and the percent solids were normal.

Box 2: From September 2018 to April 2019 the solids concentration decreased even though the MLSS concentration stayed stable, and the facility increased sludge disposal to compensate.

Box 3: From October 2019 to September 2020 the solids concentration at times was very low and the MLSS concentration increased as the secondary process became overloaded with sludge because sludge could not be removed from the facility fast enough to maintain the MLSS concentration. The sludge is hauled in trucks and if it is too watery, more and more trucks will be needed for disposal of the same mass of solids; however, not enough trucks could be sent to keep up with the need to haul the more liquid sludge. This leads to more sludge being stored in the aeration tanks and leads to higher MLSS concentrations. This in turn leads to poorly settling sludge which in turn will lead to more solids escaping the secondary clarifiers and into the feed of the filters. This can then lead to excessive backwashing and clogging of the filters.

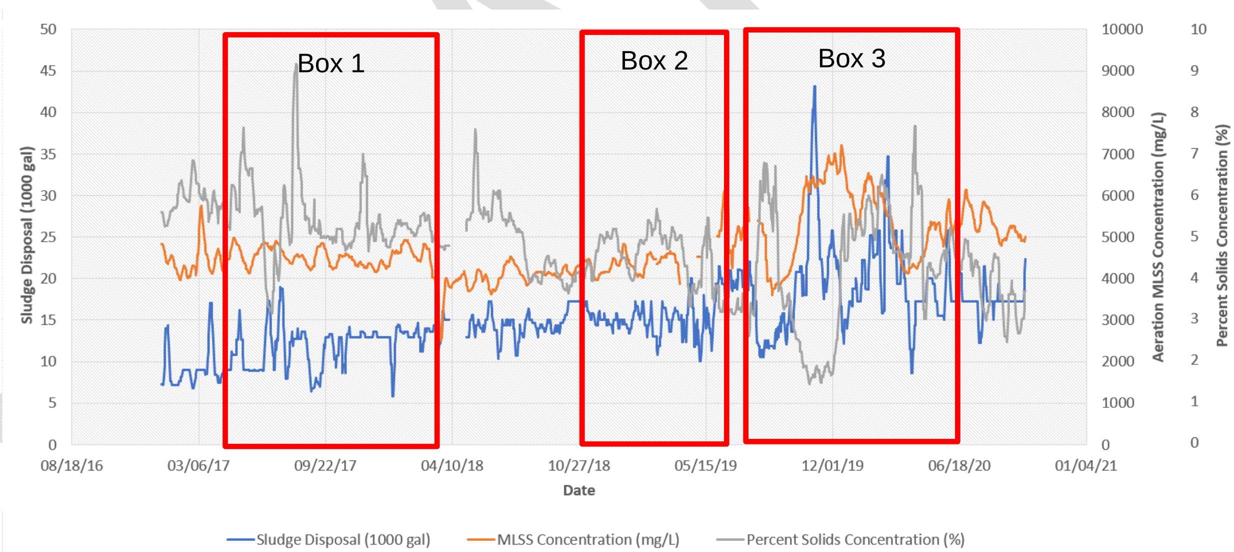


Figure 4.4 Impact of Thickened Sludge Solids Concentration and Sludge Hauling on MLSS Concentration

The cause of the lower percent solids concentration is not known. However, possible causes could include problems with the polymer used to thicken the sludge or operation of the gravity belt thickener. The facility has been adding a larger dose of poly-aluminum chloride (PAC) to help treat the increased influent load of phosphorus. There is a possibility that the additional PAC may be interfering with the cationic polymer and reducing its effectiveness. Additional testing would be required to determine how well the polymer is settling solids.

4.9.3 Operational Issues

The following operational issues were noted by the Town:

- Age of the storage tanks are from the 1970s and past their useful life.
- Issues with thickening and disposing of sludge.

4.9.4 Recommendations

The following table displays recommended improvements to the sludge processing that are recommended for the next facility upgrade.

Further upgrades were evaluated in the criticality matrix, process evaluation, and fiscal sustainability plan but should be further considered with future expansions to the facility.

Table 4.21 Sludge Processing Recommendations

Issues	Recommended Improvements
Problematic thickening issues due to polymer used.	Conduct polymer jar test to determine the effectiveness of the polymer being used to settle solids. If this is not the reason for the low solids concentration, continue to troubleshoot cause including considering additional training for the gravity belt press.
Storage tanks past their useful life.	Inspect the tanks to determine the condition of the tanks and replace the tanks if they show signs of significant wear. Upon replacing the tanks, increasing the storage capacity should be considered.

4.10 Plant Water System

4.10.1 Description

The facility's plant water system is located in the pump room of the Filter Building. The system consists of three pumps, a holding tank, and a control panel. The pumps draw suction from the UV channel, downstream of the UV reactor prior to the composite sampler. Discharge from the pump system is metered, chlorinated with sodium hypochlorite solution, and split into two pipes; one pipe is dedicated to the Filter Building for seal water, chemical feed, and flushing needs and the other pipe branches off to supply the Headworks Building, Operations Building, Sludge Dewatering Building, and other plant water needs including yard hydrants, chemical solution makeup, biofilter irrigation, and the foam spray system.

Table 4.22 Plant Water Skid

Manufacturer / Model	Tiger Flow / SP6C
Number of Pumps	3
Capacity, gpm @ 219 TDH	225
Modor / HP / RPM	General Electric / 20 / 3,545
Water Tank	1

Manufacturer / Model	Tiger Flow / SP6C
Manufacturer / Model	Simmons Pump Corporation / TRG-646B
Capacity, gallons	185

4.10.2 Evaluation

The plant water system is unable to produce adequate volume of plant water and denitrification filter backwash water. Previously, the facility has had to use Town potable water to replace the plant water needs when the system is not providing an adequate volume.

The use of Town potable water is a significant financial cost to the facility.

When the denitrification filters clog and back-up, cleaned wastewater flow cannot proceed forward to fill the denitrification filter backwash clearwell or plant water system. Plant water is important for the operation of the facility; it is used for pump seal water, preliminary treatment screenings, and for cleaning equipment.

4.10.3 Operational Issues

The Town has noted that they have had to use Town potable water to replace the plant water needs when the system is not providing an adequate volume.

4.10.4 Recommendations

The following table displays recommended improvements to the plant water system that are recommended for the next facility upgrade.

Table 4.23 Septage Receiving Recommendations

Issues	Recommended Improvements
Inadequate plant water supply.	Construct a water well on the WPCF to provide connections to the plant water system in the denitrification filter backwash clear well.

4.11 Chemical Feed Systems

4.11.1 Description

The facility's chemical feed system comprises the following chemicals:

- Soda Ash
- Sodium Hypochlorite
- Potassium Permanganate
- Methanol

The soda ash system is located inside a 32-ton capacity storage silo adjacent to Aeration Tank No. 3. Soda ash is provided to supply a 4% soda ash solution to the application points at the EQIR line to the mixing chamber. The soda ash solution provides alkalinity adjustments as part of the process control of the activated sludge process. The storage silo has two centrifugal feed pumps, one tank, and one mixer, and provides soda sash to the application points through one of two 1-inch PVC lines.

The sodium hypochlorite system is located in the Chemical Room of the Filter/Blower Building and provides sodium hypochlorite for plant water disinfection. A 55-gallon storage barrel stores the chemical while two metering pumps provide sodium hypochlorite to the following application points: effluent flushing water and return activated sludge discharge.

The potassium permanganate system, located in the Chemical Room of the Filter/Blower Building, was provided for oxidation of odorous compounds in the wastewater throughout various stages at the WPCF. The potassium permanganate system consists of one eduction vacuum blower, one potassium permanganate vacuum eductor, one potassium permanganate vacuum receiver with filter shaker, one potassium permanganate dry feeder with dust collector, and one potassium permanganate dissolving tank with internal hydraulic mixer and dust and mist control.

The methanol system supplies methanol to the secondary effluent line prior to entering the denitrification filters. The methanol is supplied as a carbon source to enhance denitrification in the filters. A 2,000 capacity ConVault above-ground tank storing the methanol is located adjacent to the UV disinfection building.

4.11.2 Evaluation

Some of the chemical systems were installed in the 2005 upgrade and the equipment is close to the end of its design life.

The potassium permanganate system is no longer being used for odor control. This system can be abandoned as the Town is currently pursuing alternate odor control solutions at the facility.

The soda ash solution has been clogging the lines making delivery of this chemical difficult. An alternative alkalinity additive should be considered.

The methanol facility and feed system are nearing the end of the equipment’s life. Water piping to the feed pumps is not heated and has had issues with freezing. The system should be further evaluated.

4.11.3 Operational Issues

The following operational issues were noted by the Town:

- The soda ash mixer and feed lines have become inoperable over the past few years as the piping has become clogged.

4.11.4 Recommendations

The following table displays recommended improvements to the chemical systems that are recommended for further evaluation.

Table 4.24 Septage Receiving Recommendations

Issues	Recommended Improvements
Clogged soda ash lines.	Rehabilitate or replace the existing soda ash storage and feed system. If replacement is the preferrable alternative, consider another chemical for alkalinity adjustments or a different method of chemical feed.
Potassium Permanganate	Abandon the system as the Town is currently perusing alternative odor control.

4.12 Criticality Matrix – A Summary of Evaluations

The following table provides a summary of the assets and projects identified, and has assigned a risk rating from Low to Very High as a function of the probability the equipment will fail (LoF) and the consequence of it failing (CoF). This risk assessment matrix allows the Town to develop a plan to prioritize projects by the risk they pose. The prioritization of the projects identified are ranked from Very High priority to Low priority. Within each priority category, they are ranked by condition (CA Rating) from worst condition (5) to best condition (1). Also included in the table are projects that the Town has indicated they will pursue and address.

Table 4.25 Criticality Matrix

Process	CA Rating	PA Rating	LoF Rating	CoF Rating	Rating Matrix
Denitrification Filters	3	5	5	3	(4) Very High
Plant Water System	4	3	4	3	(4) Very High
Effluent Flow Meter	2	4	4	3	(4) Very High
Gravity Belt Thickener	2	4	4	3	(4) Very High
UV Disinfection	2	4	4	3	(4) Very High
Septage Receiving - Rotary Fine Screen	4	4	4	3	(4) Very High
Septage Equalization Blowers	4	4	4	3	(4) Very High
Soda Ash System	4	4	4	3	(4) Very High
Methanol System	3	3	3	4	(4) Very High
Secondary Clarifiers	4	5	5	3	(4) Very High
Influent Box	4	3	4	3	(4) Very High
Anoxic Tanks	3	4	4	3	(4) Very High
Aeration Tanks	4	3	4	3	(4) Very High
Sludge Storage Tanks	4	4	4	3	(4) Very High
Aeration Blowers	4	3	4	3	(4) Very High
Septage Equalization Tanks	4	3	4	3	(4) Very High
Septage Equalization Transfer Pumps	4	3	4	3	(4) Very High
Plant Water Pumps	4	3	4	3	(4) Very High

Process	CA Rating	PA Rating	LoF Rating	CoF Rating	Rating Matrix
Denitrification Filter Equipment	3	4	4	3	(4) Very High
Sludge Treatment Equipment	2	4	4	3	(4) Very High
Parshall Flume	2	4	4	2	(3) High
Vortex Grit Unit	3	4	4	2	(3) High
Anoxic Tank Mixers	4	4	4	2	(3) High
Potassium Permanganate System	3	4	4	2	(3) High
Sodium Hypochlorite System	4	4	4	2	(3) High
Influent Channel	3	2	3	3	(3) High
Emergency Generators	3	2	3	3	(3) High
Equalization Basins	3	3	3	3	(3) High
Return Activated Sludge Pumps	3	3	3	3	(3) High
Waste Activated Sludge Pumps	3	3	3	3	(3) High
Septage Grit System	3	3	3	3	(3) High
Sludge Storage Transfer Pumps	3	3	3	3	(3) High
Thickened Sludge Transfer Pumps	3	3	3	3	(3) High
Equalization Basin Pumps	3	3	3	3	(3) High
Equalization Basin Blowers	NA	4	4	1	(2) Medium
Scum Pumps	3	3	3	2	(2) Medium
Sludge Storage Tank Equipment	3	3	3	2	(2) Medium
Headworks Building Biofilter System	3	2	3	2	(2) Medium
Dewatering Building Biofilter Systems	3	2	3	2	(2) Medium
Headworks Rotary Fine Screen	2	2	2	2	(1) Low
Headworks Bypass Bar Rack	2	2	2	2	(1) Low
Polymer System	2	2	2	2	(1) Low
Poly-Aluminum Chloride (PAC) System	2	2	2	2	(1) Low

5. Sustainable Design

There are many opportunities to incorporate sustainability considerations into the wastewater treatment process, thereby reducing the carbon footprint of the facility and realizing operational savings through the minimization of wasted power.

The sustainability alternatives discussed in this section are considered either good practice or better than standard practice. The alternatives have been evaluated and categorized as one of the following options:

- Measure to be considered in preliminary and final design—more analysis is required on these items to determine whether these are recommended items.
- Not recommended measure—these items are not recommended for implementation.

5.1 Water Conservation

Installation of Reduced Flow Plumbing

Water usage may be minimized through the installation of reduced flow plumbing such as water-saving toilers, reduced flush devices, and restricted shower heads. **This is an item to be considered in the next facility upgrade.**

Reduced Infiltration and Inflow

Locating and repairing sources of inflow and infiltration in the collection system helps minimize the amount of water that needs to be pumped to and treated by the facility. It is recommended that Wareham continue to work to reduce I/I within the existing collection system. **This is an item to be considered prior to the next facility upgrade.**

Reclaimed Wastewater Reuse

Potable water usage can be minimized through the reuse of effluent water (plant water) for non-potable purposes. The facility currently uses plant water for spray wash in the screenings and influent wet well, for the polymer blend unit, for the plant spray hydrants, and pump seal system. An assessment should be conducted to determine where there are any other economic effluent reuse opportunities at the facility. **This is an item that will be considered in the next facility upgrade.**

Landscaping

Landscaping water conservation measures can be accomplished through the planting of native species to eliminate supplementary watering needs and use of landscaping features, such as open-grid pavers. **This is an item to be considered in the next facility upgrade.**

5.2 Energy Efficiency

Energy Audit

An energy audit is used to determine if the equipment at a facility is properly sized for a process. There is some existing mechanical equipment at the facility that is past its useful life and in need of replacement. **Because of the volume of equipment replacement and process modifications, it is not recommended that an energy audit of existing equipment be conducted at this time.**

Optimizing Existing Infrastructure

It is possible that existing infrastructure can be reused in future construction. **This is an item to be considered in the next facility upgrade.**

Sub Metering

Energy usage can be minimized through system monitoring. Sub-metering will allow the facility to track the energy usage of individual processes and equipment. **This is an item to be considered in the next facility upgrade.**

Energy Management System

Energy management systems are used to lock out specified process operations during periods of peak energy demand to minimize demand charges from the local utility. **It is recommended that this be considered in the next facility upgrade.**

Upgrade Existing Motors to Variable Frequency Drives

Variable frequency drives (VFDs) should be considered for all major equipment and process modifications at the facility. **This is an item to be considered in the next facility upgrade.**

Process Optimization

Most wastewater treatment facilities are designed with oversized equipment to account for uncertainty in influent variations, to provide additional capacity for future growth, and to meet State and local regulatory criteria. Probes and the use of up-to-date electronic equipment for use in Supervisory Control and Data Acquisition (SCADA)controls can help with process optimization. The WPCF has various influent probes and flow metering devices. However, they are past their useful life and replacement should be considered along. It is recommended that an optimization analysis and replacement plan be conducted. **This is an item to be considered prior to or during the next facility upgrade.**

Reduce Ventilation and Heating Requirements

Codes should be examined for provisions that allow for lower heating requirements and fewer air changes when an area is unoccupied in order to reduce energy consumption for ventilation and heating. The Operations and Sludge Processing Buildings HVAC systems will be redone with consideration for high-efficiency HVAC equipment. **This is an item to be considered in the next facility upgrade.**

Implementation of Instrumentation and Control Systems

Instrumentation and control systems, such as SCADA, are used to help match supply with demand. SCADA can be used to monitor energy usage trends and to remotely optimize process control through the measurement of variables such as liquid and gas flow rates, chemical residual, and dissolved oxygen concentrations. The Wareham WPCF has a SCADA control system that can be analyzed for process control optimization. As issues arise with functionality of the system it should be addressed. **This is an item that should be addressed as individual issues arise.**

Optimize Lighting

Energy efficiency measures to be considered for the lighting system include adding motion sensors on lights in non-process buildings, using high-efficiency fixtures, and maximizing the use of natural light through the use of windows, translucent panels, skylights, etc., to reduce reliance on artificial lighting. In order to limit light pollution, light sensors or light timers should be considered and exterior lighting should be limited to what is required by local codes or for safety. **For any processes and buildings that will be upgraded, this is an item to be considered in the next facility upgrade.**

Optimize Building Envelope

Upgrading building envelope requirements, using upgraded insulation and window requirements, should be considered at the facility. **This is an item to be considered in the next facility upgrade for any new or modified buildings.**

5.3 Energy Recovery

Hydroelectric Potential

If adequate head is present in an effluent pipe, a hydro-turbine could be utilized to recover a portion of the potential energy in the flow with a low head generation device. **This is an item that is unlikely to be feasible at this facility.**

Anaerobic Sludge Digestion

Anaerobic sludge digestion is a process in which microorganisms break down organic materials in the absence of oxygen. A by-product of the process is the production of methane gas, which can be harvested and used as a biogas. The biogas can be used to power boilers, generators, pumps, or blowers. **Due to the high infrastructure costs of anaerobic digestion, it is not recommended that anaerobic digestion be retained for further evaluation.**

Effluent Heat Recovery

Typical wastewater effluent contains enough heat, extractable through a heat exchanger, to be considered as a building heating source. Effluent heat pumps have a relatively low impact on energy consumption at a facility. **This is an item to be considered in the next facility upgrade.**

5.4 Alternative Energy

Solar

The Town could consider solar photovoltaic (PV) systems to produce renewable energy onsite. There is available land adjacent to the biofilters serving preliminary treatment that should be investigated for solar feasibility. **This is an item to be considered in the next stage of design at the facility.**

Wind

It is not recommended that wind energy is pursued further at this time.

Geothermal

Geothermal systems use the nearly constant temperature of the earth to act as a heat source and heat sink to heat and cool building through a heat pump and a heat exchanger. A heat exchanger is a system of pipes buried in the shallow ground near the building. **This is an item to be considered in the next facility upgrade.**

5.5 Site Considerations

Low Pollution Generator

The Town should consider the installation of a low-polluting emergency generator at the facility. **This is an item to be considered in the next facility upgrade.**

5.6 Summary

Table 5.1 summarizes the sustainability considerations that are recommended to be considered during preliminary and final design.

Table 5.1 Summary of Sustainable Design Considerations

Water Conservation	Energy Efficiency	Energy Recovery	Alternative Energy	Site Considerations
Installation of reduced flow plumbing	Optimizing existing infrastructure	Hydroelectric potential	Solar	Low-polluting generator
Reduced I/I	Sub-metering	Effluent heat recovery	Geothermal	
Reclaimed WW reuse	Energy management systems			
Landscaping	Upgrade existing motors to VFDs			
	Process optimization			
	Reduce ventilation and heating requirements			
	Implement instrumentation and control systems			
	Optimize lighting			
	Building envelope upgrade			

6. Fiscal Sustainability Plan

The fiscal sustainability plan (FSP) is a tool that can help the WPCF plan and prioritize future projects. The hope is that the plan can help the facility spend less time on reactive maintenance and more time on preventative maintenance. The fiscal sustainability plan uses the ranking system determined by the criticality matrix and assigns allowances to upgrade the systems. The plan recommends updates for fiscal years 2021 through 2025, further upgrades are recommended to be reevaluated in the future and considered in conjunction with a full facility upgrade. The Fiscal Sustainability Plan is intended to be a guide, used for planning and proactive improvements to the facility. It is not intended to imply a required level of spending from the Wareham WPCF enterprise fund. The budget for annual improvements needs to be considered in the context of what is affordable for the fund.

The fiscal sustainability plan is presented in the following table:

Table 6.1 Fiscal Sustainability

Process	Notes/ Recommendations	Year 1 (FY 2021)	Year 2 (FY 2022)	Year 3 (FY 2023)	Year 4 (FY 2024)	Year 5 (FY 2025)
Denitrification filters	Add additional filters for redundancy	Costs included as part of the Wareham WPCF Improve-ments Phase 1 Project				
Plant Water System	Construct and connect a well to the plant water system					
Effluent Flow Meter	Analyze meters electronics and SCADA data records					
Gravity Belt Thickener	Perform Polymer Jar Test					
UV Disinfection	Fix leakage and hydraulics issues as required until future flow requirement is known		$10,000			
Septage Receiving - Rotary Fine Screen	Replacement of screen. Does not include full system or equipment overhaul		$110,000			
Septage Equalization Blowers	Inspect tank and diffuser system, evaluate condition, and consider replacing blowers. Consider sending septage to solids treatment process		Inspect septage tanks, re-evaluate costs after inspection			
Soda Ash System	New Sodium hydroxide building and system.		$1,040,000			
Methanol Storage Tank	Add grounding rod		$30,000			

Process	Notes/ Recommendations	Year 1 (FY 2021)	Year 2 (FY 2022)	Year 3 (FY 2023)	Year 4 (FY 2024)	Year 5 (FY 2025)
Secondary Clarifiers	Secondary clarifiers have limited long-term use if additional flow capacity is required; address short comings only as required until a decision has been made for flow requirement.[1]			$330,000		
Influent Box	Allowance for concrete repair and bypass pumping			$220,000		
Anoxic Tanks	Replacement of water spray lines, baffle tie lines			$50,000		
Aeration Tanks	Allowance to repair concrete and inspect diffusers in Tank 2			$50,000		
Sludge Storage Tanks	Allowance for concrete repair; Additional storage is dependent on future flow requirements			$280,000		
Aeration Blowers	New valves, replace controls and verification of existing programming				$110,000	
Septage Equalization Tanks	take tank out of service rehab the concrete; check Septage Equalization Submersible Aerators				$60,000	
Septage Equalization Transfer Pumps	Replacement of pumps				$380,000	
Plant Water Pumps	Replace plant water system pumps				$230,000	
Denitrification Filter Equipment	Town personnel replace pumps in-kind				$230,000	
Sludge Treatment Equipment (Booster Pumps, Air compressor, Air Dryer)	Town personnel replace equipment in-kind					$20,000
Parshall Flume	Evaluation and replacement of					$10,000

Process	Notes/ Recommendations	Year 1 (FY 2021)	Year 2 (FY 2022)	Year 3 (FY 2023)	Year 4 (FY 2024)	Year 5 (FY 2025)
	electronics as needed					
Vortex Grit Unit	Diagnose the poor removal efficiency and replace pumps if needed, replace equipment as needed					$460,000
Anoxic Tank Mixers	Replace existing mixers with submersible mixers					$80,000
Potassium Permanganate System (Blower, Hooper and feeder, dissolver tank, Mixer, pumps)	Replace potassium permanganate system					$180,000
Sodium Hypochlorite Metering Pumps	Replace sodium hypochlorite metering pumps					$40,000
Total[1]		**Not Applicable**	**$1,190,000[2]**	**$650,000**	**$1,010,000**	**$790,000**

Notes:

1. *Cost shown in dollar amount for the physical year that the projects are proposed for (see column header).*
2. *Total cost does not include a cost for septage tank blowers, an inspection of the existing septage tanks is recommended to assign a cost to their improvement.*

6.1 Short-Term Upgrades and Recommendations

The process evaluation and criticality matrix analysis highlighted several priority upgrades that are needed at the facility to continue to functionally operate the facility and meet permit limits.

The processes that were considered top priority are recommended to be upgraded first. The processes are described in more detail below:

Plant Water System: When the facility experiences an upset that reduces the effluent flow, the facility is unable to produce an adequate volume of plant water. Plant water is necessary for the operation of the facility and is used in processes such as pump seal water, cleaning of equipment, and rinsing of grit and screenings. In past years the facility has had to use Town potable water to replace the plant water needs when the plant water system is not providing adequate amounts. The use of Town potable water is a significant financial cost to the facility. The recommendation for this problem is to construct and incorporate a well on the facility site which will be tied into the plant water system. The decision to construct a well is explained in more detail in Section 7 and the basis of design for the well will be described in the draft Basis of Design memorandum currently in development in the design project for this improvement.

Denitrification Filters: As described previously in the process evaluation, the denitrification filters are currently undersized for peak flow due to the upgrades to the wastewater guidelines. This report recommends that additional denitrification filters are added to the filter system to allow for increased redundancy to the system and to meet the updated wastewater guidelines. The decision-making process for additional denitrification filters is explained in greater detail in Section 7and the basis of design for the denitrification filter expansion is included in Appendix D.

Odor Control and Equalization: For multiple years the WPCF has had occasional odor concerns on site and around the neighboring residential properties. Currently the WPCF holds daily peak flow in open raw wastewater equalization basin. There is a concern that this system is creating additional odors for the site and neighborhood. The Town has requested the construction of an additional equalization basin that is covered and has an odor control system that treats the air from within the basin to help reduce daily odors from the site. The options considered as well as design criteria is explained in greater detail in Section 7 and the basis to design for the fifth equalization basin and odor control system is included in the basis of design memorandum in Appendix E.

Effluent Flow Meter: Currently, data from the effluent flow meter is not being recorded or incorporated into facility analysis. It is recommended that the effluent flow meter be fixed, and that the data be recorded and analyzed. This data would be very helpful for the operation of the facility and to confirm that permit limits are being met.

Sludge Thickening Process: Recently the facility has experienced problems with thickening and disposing of sludge. This problem has led to problematically high concentrations of solids in the secondary treatment system which has caused settling issues in the clarifiers and clogging in the denitrification filters.

The mass of sludge disposal (in pounds), the mixed liquor suspended solids (MLSS) concentration, and the percent solids concentration coming of the gravity belt press were analyzed from 2017 through 2020. From April 2017 through April 2018 the MLSS concentration was stable and the percent solids were a good level (Box 1). From September 2018 through April 2019 the MLSS concentration stayed stable but the solids concentration dropped (Box 2); the facility increased the volume of sludge disposal to compensate. The MLSS concentration was unstable and at times the solids concentration was much lower from October 2019 through September 2020 (Box 3). The graph showing the solids treatment trends is below.

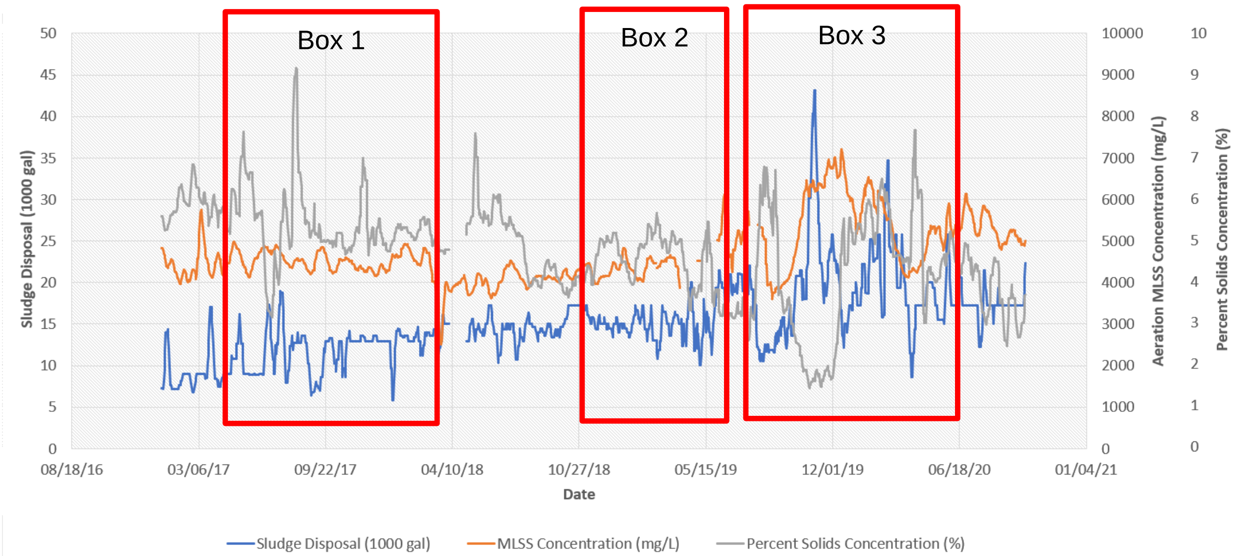


Figure 6.1 Impact of Thickened Sludge Solids Concentration and Sludge Disposal on MLSS Concentration

The cause for the lower percent solids concentration is not known. Possible causes could include problems with the polymer used to thicken or operation of the gravity belt thickener. The Town noted that the facility did not note any changes to the operation of the gravity belt thickener during the time when the concentrations were unstable. The facility has been adding a larger dose of poly-aluminum chloride (PAC) to help treat the increased influent load of phosphorus. There is a possibility that the additional PAC may be interfering with the cationic polymer and reducing its effectiveness. A more in-depth investigation into the root cause of the solids issues is recommended, along with a polymer jar test to determine how well the polymer is settling solids. Changes to the polymer, PAC dosing, or operation of the gravity belt thickener may be helpful to improve solids thickening.

In addition to the top priority facility upgrades, secondary priority level issues were also considered. The secondary priority level upgrades were analyzed from the process evaluation and criticality matrix and are listed below:

- Upgrade soda ash storage system and replace piping for the silo.
- Upgrade septage receiving rotary fine screen.
- Upgrade septage equalization blowers.
- Upgrade methanol storage tank.
- Upgrade UV disinfection system.

Further upgrades were evaluated in the criticality matrix, process evaluation, and fiscal sustainability plan but should be further considered with future expansions to the facility.

6.2 Facility Expansion

As noted in the flow memorandum included in Appendix A, the Town of Wareham has committed flow above the current design flow of the facility. To plan for future flows and expansions to the facility the development of an updated Comprehensive Wastewater Management Plan (CWMP) is highly encouraged.

Without defined future flows it is difficult to recommend upgrades or provide cost estimates. For this report conceptual costs were estimated to upgrade the existing facility into a flow-through facility (without the need for equalization) and to increase the capacity to handle an average daily flow up to 3 mgd. These design flow capacities were chosen using the current flows and committed future flows but would need to be fine turned with flows from the updated CWMP.

Conceptual designs for expansions to the facility were investigated as part of the *Wareham WPCF Expansion Memorandum* and corresponding project. Options to upgrade the facility into a larger Modified Ludzack-Ettinger (MLE) system or convert the system into a Membrane Bioreactor (MBR) were both considered. The facility expansions for this evaluation were based off the conceptual designs from the previous project. The conceptual level engineer's opinion of probable costs for upgrade the facility into a larger MLE system or an MBR are described in the next sections.

6.2.1 Description of Engineers' Opinion of Probable Costs

Engineers' Opinion of Probable Capital Costs for infrastructure, recommended as part of a multi-year planning project, are initially developed as part of the planning process. As a project progresses, it is critical that these costs are updated and refined at each stage of the planning and design process to accurately reflect items that may impact them. Items that could impact cost include, but are not limited to:

- Changes in bidding climate and tariffs.
- Design changes resulting from future law, regulation, or code changes.
- Design changes resulting from industry or manufacturer advances, updates, or changes.
- Owner-driven decisions and changes.

- Unknown conditions discovered through field investigations during design (borings, surveys, etc.).
- Design decisions regarding proprietary equipment/sole sourcing of equipment.

Allowances for Engineers' Opinion of Probable Costs

Allowances are carried for some of the processes that are not typically designed during a planning or conceptual level design. The allowances that are carried in the engineer's opinion of probable costs are:

- Electrical
- Instrumentation
- Heating, ventilation, and Air Condition (HVAC)
- Yard Piping
- Site Work
- Plumbing
- Painting

Contingency

Due to the conceptual nature of this design, a 30% construction contingency is carried to cover undeveloped parts of the project and bidding variability. During final design, a reduced contingency would be carried, as more design details will be addressed. The final design contingency is primarily for variability in the bidding climate, project changes before bidding, and change orders due to unforeseen conditions.

Legal, Fiscal, and Engineering Allowance

The Legal, Fiscal, and Engineering Allowance represents the project costs that cover legal and financial work, and engineering design and construction phase services. For the conceptual engineers' opinion of probable costs, the legal, fiscal, and engineering allowance value is based on previous wastewater projects. This is only intended to cover costs related to the design and construction of the expansion.

The Engineers' Opinion of Probable Capital Costs presented in this report would continue to be refined and updated at each major stage of the design process and prior to construction financing.

6.2.2 Engineers' Opinion of Probable Costs for the MLE Expansion

An engineers' opinion of conceptual costs was developed for the MLE expansion. In addition to the allowances described above the processes and buildings that were considered in the engineer's opinion of probable costs were:

- Preliminary Treatment
- MLE Reactors
- Clarifiers

- Denitrification Filters
- UV Disinfection
- Effluent Pump Station
- Solids Treatment
- Septage Receiving Building Rehab
- Odor Control
- Process & Filter Building
- Operations Building Allowance
- Administration Building

The cost of an upgrade to the facility to make the facility a flow through MLE treatment process that could handle 3 mgd could be between 60,000,000 and 110,000,000 dollars (in 2021 dollars). The varying costs could be depended on whether the facility was upgraded all at once or in stages to increase the treatment capacity in multiple phases. The range of costs are also dependent on what upgrades are chosen; some upgrades to the existing equipment and buildings could be done during the facility expansion or could be completed at a later date.

The total capital costs in the engineers' opinion of probable costs are in 2021 dollars. Inflation would adjust the final construction costs. Currently, GHD usually estimates a 3% annual inflation rate, however this is subject to change depending on multiple factors and the date of construction. For example, if the 3% annual inflation stayed constant and the midpoint of construction was in 2025, the $110,000,000 total construction cost would be $125,000,000 in 2025 dollars. Changes in bidding climate, supply costs, or inflation rate could change the total construction costs value.

A conceptual layout of the 3 mgd and a potential 7 mgd expansion of the plant are shown in the following figure. The 7 mgd layout is used to show potential site space needs in the future.

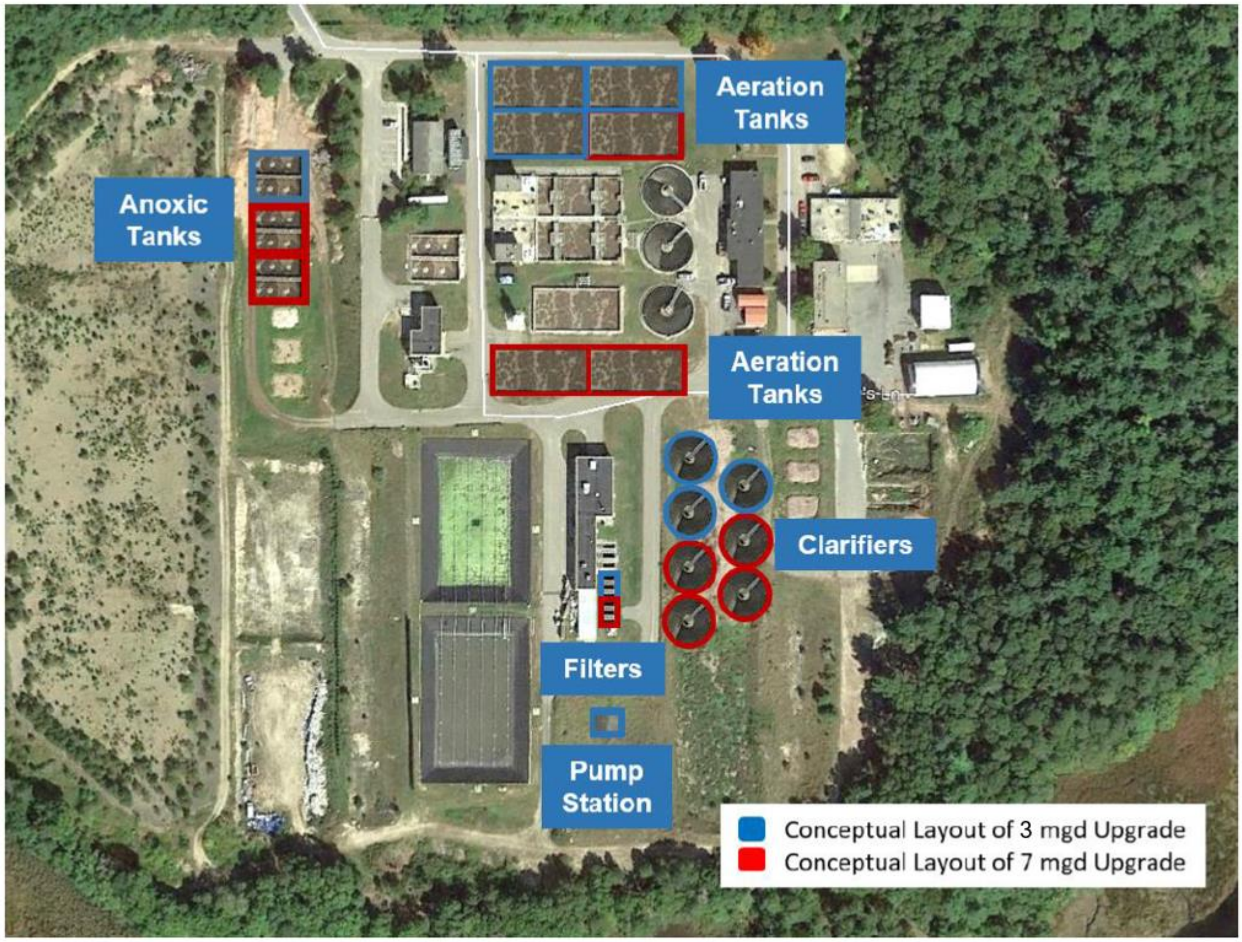


Figure 6.2 Conceptual Facility Site Layout

6.2.3 Engineers' Opinion of Probable Costs for the MBR Expansion

An engineers' opinion of conceptual costs was developed for the MBR expansion. In addition to the allowances described above, the processes and buildings that were considered in the engineer's opinion of probable costs were:

- Preliminary Treatment
- Membrane Bioreactor
- Decommission Clarifiers
- Decommission Denitrification Filters
- UV Disinfection
- Effluent Pump Station
- Solids Treatment
- Septage Receiving Building Rehab
- Odor Control

- Process & Filter Building
- Operations Building Allowance
- Administration Building

The cost of an upgrade to the facility to make the facility a flow through MBR treatment process that could handle 3 mgd could be between $80,000,000 and $115,000,000 (in 2021 dollars). The expansion of the facility to an MBR system would involve decommissioning the secondary clarifiers and denitrification filters, and converting the existing MLE secondary treatment system to a membrane bioreactor. Similar to the MLE, the range of costs are dependent on how the Town wishes to expand the facility and which upgrades they chose to include in the facility expansion.

The costs in the engineers' opinion of probable costs are in 2021 dollars. Inflation would adjust the final construction costs. Currently, GHD usually estimates a 3% annual inflation rate, however this is subject to change depending on multiple factors and the date of construction. For example, if the 3% annual inflation stayed constant and the midpoint of construction for the 3 mgd expansion was in 2025, the $115,000,000 total construction cost would be $130,000,000 in 2025 dollars. Changes in bidding climate, supply costs, or inflation rate could change the total construction costs value.

6.2.4 Options for Incremental Capacity Upgrades

It is understood that the existing facility is in need of several critical process upgrades. These are needed as a result of either condition or capacity issues. With flows being committed for up to 33% more than the current rated capacity of the facility, the Town needs to provide at least 0.5 mgd more capacity during average daily flow conditions.

In order to make these upgrades cost effective, the Town would need to choose the path of building upon the existing process, which would then preclude expansion by MBR. This path of upgrades would focus on the existing MLE process, upgrading and expanding existing processes to increase the capacity of the facility over time.

The three highest critical upgrades recommended are upgrading the preliminary treatment and septage receiving processes, addition of secondary clarification capacity, and increase solids storage capacity. The preliminary treatment equipment is heavily used and the overall condition of it is aging and corroding. Preliminary treatment protects downstream processes from grit and screenings buildup issues. As the preliminary treatment equipment conditions worsens, it risks the downstream tanks and equipment. Thus, upgrades to the preliminary treatment process (including septage receiving) would include the following:

- Equipment for two (2) new fine screening systems and one (1) new grit vortex system.
- Expansion of the Preliminary Treatment Building and additional screenings channel
- Allowance to improve the septage receiving station
- Upgrade of septage holding tanks (including targeted equipment replacement and concrete repair or replacement).

Two of the three secondary clarifiers are original to the plant and the equipment in each clarifier is beyond its useful life. In addition, the depth of all three clarifiers are much shorter than current design guidelines recommend. It is recommended that new clarifiers be added which have sufficient sidewater depth, rather than upgrade the equipment in the existing clarifiers. The new clarifiers would add capacity to the facility's treatment process in order to accommodate flow increases at the facility. The recommended improvements to the secondary clarification process include the following:

- Addition of two new 85-foot diameter clarifiers, including internal mechanism, density current baffles, weirs, and baffles.
- Addition of a RAS pump station and distribution box for the new clarifiers.
- Demolition of the existing clarifiers.

The facility has been experiencing issues with solids settling and low solids concentration in its thickened sludge. It is recommended that additional sludge storage tanks be added to increase the settling time allowing the polymer to thicken the sludge more prior to entering the gravity belt thickener. Additional settling tanks will allow for more solids to be removed from the WPCF. This upgrade includes the following:

- Addition of up to two solids holding tanks (including hatches and railing).
- Equipment for the additional solids tanks including blowers, coarse bubble aeration system, and an allowance for piping.

Finally, the denitrification filters and the UV Disinfection system are unable to handle an increase in flow. It is recommended that additional filters and UV Disinfection capacity be added to increase treatment capacity and higher flows. This upgrade includes the following:

- Upgrade the UV system and increase capacity from 1.56 mgd to 3.5 mgd, including a new concrete structure, two channels, and new UV modules and associated controls equipment.
- Addition of three denitrification filters and associated equipment to increase filtration capacity from 1.56 mgd to 2.2 mgd.

Critical upgrades, including the reason for the upgrade as well as recommended modification, are shown in the following table.

Table 6.2 Critical Upgrades

Priority	Need	Reason	Modifications	Budgetary Total Project Costs (2021 $)
1	Preliminary Treatment Upgrade (including septage receiving).	Most heavily used portion of the facility; significant rehabilitation is needed.	Rehabilitation of all processes and equipment.	$6,500,000
2	Secondary Clarification.	Shallow tanks have shown to be prone to failure; equipment is at or beyond its useful life; upgrade needed for future capacity.	Construction of two new 85-foot diameter secondary clarifiers will increase capacity from 1.56 mgd to 3 mgd, including treating peak flows	$11,000,000

Priority	Need	Reason	Modifications	Budgetary Total Project Costs (2021 $)
3	Targeted Sludge Processing Upgrade	Current issues with solid settling and sludge holding capacity.	Up to two additional tanks to increase storage capacity.	$1,250,000
4	UV	System is at or beyond its useful life, the system is unable to handle an increase in flow.	Upgrade to increase capacity from 1.56 mgd to 3.5 mgd and modernize the system, including treating peak flows	$4,500,000
5	Denitrification Filters	Denitrification filters have experienced clogging, the current WPCF Improvements project shall improve redundancy; additional equipment is needed to increase treatment capacity.	Upgrade to increase capacity; three more filters and additional equipment will raise facility capacity from 1.56 mgd to 2.2 mgd, including treating peak flows.	$8,000,000

As mentioned in Section 2, the secondary process seems to be at 60-70% capacity. This will require an upgrade soon if additional nutrient testing confirms this loading. This upgrade may be higher on the priority list if additional sampling shows process to be higher than 60-70% of its capacity.

7. Preliminary Engineering Report for Denitrification Filters, Fifth Equalization Basin Odor Control and Plant Water System Well

In the spring of 2019, the Town of Wareham and GHD discussed current issues at the facility and determined three processes which were in need of upgrades. In August 2019 GHD prepared and submitted a Project Evaluation Form to the Massachusetts Department of Environmental Protection (Mass DEP) as part of the state revolving fund (SRF) loan program. The PEF requested nine million dollars for upgrades to the facility. In January 2020 MassDEP listed the Town of Wareham on the Intended Use Plan, eligible for a nine-million-dollar loan for the facility upgrades. The Town of Wareham passed a funding warrant article in their Town Meeting in December 2020. The SRF application, project drawings and specifications, and following preliminary engineering report will be submitted to MassDEP in February 2021 as part of the SRF loan program.

7.1 Denitrification Filters

Problem

Over the last few years, the denitrification filters have had periods where solids have clogged the denitrification filters. During these events the filters are unable to backwash rapidly enough and the flow backs up and overflows the filter walls. The overflow leads to an unpermitted discharge.

Using the most up-to-date Massachusetts wastewater guidelines, the current denitrification filters are not sized correctly to handle the peak flows. The updated TR-16 guidelines require the denitrification system be able to treat peak flow rates while one filter is backwashing.

Alternatives Considered

One Additional Filter: GHD investigated adding one additional filter to the denitrification filter system. The additional filter would add redundancy to the system to allow one filter to backwash during peak flows.

Three Additional Filters: GHD investigated adding one additional bank of three filters to the denitrification filter. The additional bank of filters would add redundancy to the system by allowing one whole bank of filters to be taken offline at a time or to allow one filter to backwash during peak flows.

Chosen Project

GHD investigated and created conceptual designs of the filter options. Using the design criteria from Leopold filters, which is the manufacturer of the existing filters, GHD estimated the average design flow capacity and maximum design flow capacity for both filter options. The design capacities are presented in the following table.

Table 7.1 Design Capacity of Denitrification Filters

	Average Design Flow Capacity (mgd)	Maximum Design Flow Capacity (mgd)
Capacity of 4 Filters (3 active and 1 backwashing)	1.00.	2.0
Capacity of 6 Filters (5 active and 1 backwashing)	1.67	3.3
Note: The secondary treatment system has an average design flow capacity of 1.56 mgd and a maximum design capacity of 2.0 mgd.		

The facility is designed to provide an average daily flow of 1.56 mgd and maximum flow of 2 mgd to the denitrification filter system. By adding one additional filter (for total of four filters) the maximum design flow capacity meets the maximum design flow of 2 mgd; however, the average design flow is only 1 mgd. This means that on average the denitrification filter system would be seeing flow above its average capacity and would often be operating at a flow closer to its maximum design capacity.

By adding three additional filters (for a total of six) the average design capacity is 1.67 mgd, this is slightly above the average flow seen by the secondary treatment system. The maximum design capacity of the six filters is 3.3 mgd which is also above the maximum flow from secondary treatment system. The six filters would be able to operate with in their average design conditions. A total of six filters would also allow the filters to be built into banks of three. While operationally it is recommended that all filters are typically on to allow biological growth, one entire bank of filters could be brought offline for a short period for maintenance.

GHD estimated conceptual engineer’s opinions of probable costs for both filter options and presented them at a November 17, 2020 meeting with the town. A record of the meeting minutes from that meeting are included in Appendix B. The conceptual level engineer's opinion of probable costs for both options are presented in the following table.

Table 7.2 Conceptual Level Engineer's Opinion of Probable Costs

	1 Additional Filter	3 Additional Filters
Cost	$1,500,000	$3,500,000

GHD and the Town discussed the different options for additional denitrification filters during the November 17, 2020 meeting; at the meeting the Town requested three additional filters to allow for the rated capacity of the facility to be processed. A record of this request is included in the minutes attached in Appendix B. The basis of design for the three additional filters is included in the draft basis of design memorandum in Appendix D.

7.2 Fifth Equalization Basin and Odor Control

Problem

Odors have been a concern for multiple years at the WPCF. Neighbors to the west of the facility have on occasion complained of odors. The facility currently holds untreated daily peak flow in one of two open equalization basins at the facility. There is a concern that the raw wastewater in the open basins is creating an odor on the site.

An analysis of the influent data to the WPCF showed that most days the instantaneous peak flow to the facility was at a rate of 5.47 mgd. The peak instantaneous flow was anticipated to occurred when the largest influent pump station was pumping. The peak instantaneous flow is expected to occur for an hour or less each day. GHD calculated that the storage volume required during one hour of peak instantaneous flow was approximately 150,000 gallons, assuming 2 mgd could still continue forward into the treatment process. For additional redundancy GHD also calculated the storage volume required during one and a half hours of peak instantaneous flow and estimated it to be approximately 250,000 gallons.

Alternatives Considered

Small Aquastore Tank: A 150,000 gallon covered Aquastore tank was considered to be placed in the existing depression to the west of Equalization Basins 1 and 2. The tank would be circular with a sloped bottom and a cover.

Large Aquastore Tank: A 1.2 million gallon covered Aquastore tank was considered to replace Equalization Basin 1. The tank would be circular with a sloped bottom and a cover. The large Aquastore tank would require a pump station be constructed to pump flow into the tank.

Fifth Equalization Basin: A 250,000 covered equalization basin was considered to be placed in the existing depression to the west of Equalization Basins 1 and 2.

Chosen Project

Conceptual costs for all three options were estimated and are presented in the following table.

Table 7.3 Engineer's Opinion of Conceptual Level Costs for Equalization Basin

Lump Sum Work	Covered Fifth Basin (250,000 gal)	Aquastore (150,000 gal)	Aquastore (1,200,000 gal)
Subtotal of Project Cost Estimate	$1,972,000	$1,615,000	$5,194,000
Contingency (30%)	$592,000	$485,000	$1,558,000
Total Construction	$2,564,000	$2,100,000	$6,752,000
Legal, Fiscal & Construction Phase Engineering (25%)	$641,000	$525,000	$1,688,000
Total Project Costs (2020 Dollars)	$3,200,000	$2,600,000	$8,400,000
Midpoint Construction (Spring 2022)	$3,345,075	$2,717,873	$8,780,821
Cost per Gallon Storage ($/gal)	13.38	18.12	7.32

The three options and their conceptual costs were presented to the Town at a meeting on December 3, 2020. The minutes and a copy of the presentation slides are included in Appendix C. During the meeting the three options were considered and discussed. The large Aquastore tank was above the budget allocated by the Town. It would require an additional pump station to pump flow from the preliminary treatment system into the Aquastore tank due to the overall height of the tank. The use of the large Aquastore tank would also involve decommissioning the exiting Equalization Basin 1 which was unfavorable for the Town.

The covered fifth basin and the small Aquastore tank were similar in cost; however, the covered fifth basin could hold approximately 100,000 more gallons than the small Aquastore tank. Therefore, the cost per gallon of storage was more economical for the covered fifth basin. Both options were within the Town's planned budget. Upon discussion, the Town decided that it would prefer the 250,000 gallon covered fifth equalization basin because it was more cost-effective. The record of the decision is included in the minutes in Appendix B. A basis of design for the fifth equalization basin and odor control system is included in the draft basis of design memorandum in Appendix E.

7.3 Plant Water System Well

Problem

When there is a plant upset at the WPCF, the facility is unable to produce an adequate volume of plant water or denitrification filter backwash water. When the denitrification filters clog and back-up, as described previously, cleaned wastewater flow cannot proceed forward to fill the denitrification filter backwash clearwell or plant water system. Plant water is important for the operation of the facility; it is used for pump seal water, preliminary treatment screenings, and for cleaning equipment. Previously, the facility has had to use Town potable water to replace the plant water needs when the system is not providing an adequate volume. The use of Town potable water is a significant financial cost to the facility.

Alternatives Considered

Continue Use of Town Water: The cost for Town potable water use on the WPCF site in 2019 was approximately $20,000. A similar rate would be assumed for future years and the cost for that water use would be needed in order to add that amount to the WPCF annual budget.

Construct Well: The well would be constructed on the WPCF site and would provide connections to the plant water system in the denitrification filter backwash clear well.

Chosen Project

The construction and use of a well offers the WPCF more control and flexibility over the plant water use. The Town requested the construction of the well during a November 17, 2020 meeting. A copy of the minutes from that meeting are included in Appendix B. The well will be located on the WPCF site and will include connections to provide water to the plant water system and the denitrification filter clear well. The WPCF will continue to have the option to use Town potable water if ever needed.

8. Conclusion

This evaluation analyzed the current state of the WPCF with the goal to help the Town plan for future maintenance, upgrades, and expansions. The report recommended the following actions as top priority and immediate projects for the WPCF.

Action Item No. 1: Flows (facility capacity)

Recommendations are based on managing the risks associated with the flow conditions at the Wareham WPCF. Having documented known non-permitted diversions with MassDEP, certain actions must be initiated at the WPCF, and several have been, including the new equalizations basins and planning for a new denitrification filters. Once these improvements are online, the risk of non-permitted diversion will still exist, however at a reduced level. It is our goal to recommend a flow management policy that will allow the Town to grow while minimizing risks of non-permitted diversion. However, some immediate actions are strongly recommended:

- It is recommended to not increase flows through additional connections at least until the new Equalization Basins 3 and 4 are online; the expected date of operation is summer of 2021.
- It is also recommended to not add any new flow until the new denitrification filter is installed. This project is in its planning stages and the new filter is expected to be online in late 2022 (provided the Town acquires SRF funding in 2021). However, once Equalization Basins 3 and 4 are installed, the risk of non-permitted discharge will be reduced. The installation of the new denitrification filters will further reduce the risk of non-permitted diversions.
- Even with the new equalization basins and new denitrification filters, there may be weather events that still result in a non-permitted discharge. However, we expect these events will have to be much larger than the events that have caused previous non-permitted diversions.
- It is recommended to install a flow meter at the upstream manhole of the influent line of the Cohasset Narrows Pump Station to confirm the amount of flow from the Town of Bourne. This

meter should be in place for spring and summer of 2021 to get a comprehensive reading of seasonal and non-seasonal flows.

- It is recommended that additional flow not be added above 1.5 mgd until additional discharge capacity is determined and connected to the facility and until the loading capacity can be verified after additional nutrient testing
  - GHD is currently investigating additional discharge capacity options as part of a separate project to increase effluent discharge capacity to allow facility treatment design flows (up to 2 mgd) to be discharged as well as another option to discharge flows more than 2 mgd.
  - The facility may be load limited and may require additional aeration tank capacity soon. This needs to be verified after additional nutrient testing.
  - It is recommended that the facility measure influent loads of nitrogen and phosphorus on at least a weekly basis.

Action Item No. 2: Comprehensive Wastewater Management Plan (CWMP)

It is recommended that Town revise their CWMP. A CWMP will help the Town determine their future wastewater flows. With defined future wastewater flows the Town can more accurately plan for the future and for expansions to the WPCF. Expanded discharge capacity can also be outlined in the CWMP. An updated CWMP as well as the creation of flow neutral bylaws and regulations would be important steps to help make the Town eligible for zero percent financing for nutrient related wastewater treatment projects from the State revolving fund. For the WPCF to expand, additional discharge capacity will be needed; GHD is currently investigating additional discharge capacity options as part of a separate project.

Action Item No. 3: Current Design and Construction Work

The Wareham WPCF is currently in the middle of design for upgrades to the facility. The construction of the upgrades is being funded through the State Revolving Fund (SRF). Construction for the upgrades should occur in 2021 and 2022. The upgrades are as follows:

- Expand the denitrification filter system to include three additional filters to add necessary redundancy.
- Construct a fifth covered equalization basin and odor control system to contain and treat the odors from the daily peak flow.
- Construct a well on the site to supplement the plant water system.

Action Item No. 4: Operational Suggestions

It is recommended that the WPCF troubleshoot the low solids concentration yield on the gravity belt thickener. This appears to be leading to a backup of solids at the facility from time to time and leads to process problems in the liquid and sludge treatment portion of the facility (and potentially with excessive backwashing and clogging of filters). The primary action recommended for this is:

- Perform a jar test of the gravity belt thickener feed sludge to determine the efficacy of the polymer and if a better polymer is available.

Action Item No. 5: Incremental Capacity Upgrades

The Wareham WPCF has overcommitted future flows to the facility. However, without clear future flows planning for and designing a full facility expansion is difficult. There are major processes at the facility that are in need of major upgrades, which should be coordinated with expansions to the treatment system in order to increase capacity.

The top five processes in need of capacity expansions and upgrades are listed in section 6.3.4. The top two incremental upgrades are listed in the following table and include the description of the modifications as well as an engineer's opinion of probable costs for the upgrade in 2021 dollars.

Table 8.1 Incremental Capacity Upgrades

Priority	Need	Modifications	Budgetary Total Project Costs (2021 $)
1	Preliminary Treatment Upgrade (including septage receiving).	Rehabilitation of all process and other equipment.	$6,500,000
2	Secondary Clarification.	Construction of two new 85-foot diameter secondary clarifiers will increase capacity from 1.56 mgd to 3 mgd.	$11,000,000

Action Item No. 6: Annual Improvements

The fiscal sustainability plan (FSP) can help the Town prioritize and budget for other future maintenance project and upgrades unless or until an upgrade becomes necessary.

- The fiscal sustainability plan outlines recommended upgrades for the next five years. These upgrades are based on the existing Likelihood of Failure (LoF) and Consequence of Failure (CoF) of the equipment.
- In addition to the upgrades that are recommended in the FSP, routine maintenance is required at the facility along with replacement of any equipment that unexpectantly fails at the facility.
- Beyond the five years of upgrades that were outlined in the FSP, the facility should consider other upgrades in conjunction with a total facility expansion and upgrade.

The Fiscal Sustainability Plan is intended to be a guide, used for proactive improvements and planning. It is not intended to imply a required level of spending. It is understood that the Town of Wareham WPCF operates as an enterprise fund. The budget for annual improvements needs to be considered in the context of what is affordable for the fund. The Town could use this information as part of an asset management program which would also incorporate revenue and major capital projects and would allow for proactive and continuous short- and long-term planning. The Massachusetts Department of Environmental Protection (MassDEP) has a history of providing grant funding for such projects and it is highly recommended that the Town pursue this state offered grant funding.

Appendices

Appendix A
Draft Memorandum of Capacity at the Wareham Water Pollution Control Facility

Memorandum

September 24, 2020

To:	Town of Wareham	Ref. No.:	11217251
From:	Marc Drainville, P.E. BCEE; Russ Kleekamp; Lenna Quackenbush	Tel:	774-470-1647

Subject: Draft Memorandum of Capacity at the Wareham Water Pollution Control Facility

1. Introduction

The purpose of this memorandum is to provide information on the capacity at the Wareham Water Pollution Control Facility (WPCF). The memorandum discusses the high peak influent flow rates due to I/I and diversions that the plant has experienced as well as the committed flows and permit discharge levels for the of the facility.

1.1 Permit and Design Flow

The National Pollution Discharge Elimination System (NPDES) permit authorizes Wareham to discharge an average annual wastewater effluent flow of 1.56 million gallons per day (mgd) to the Agawam River. The permit is analyzed monthly and on a rolling 12-month period. When the average annual flow reaches 80% of the permitted flow rate in a calendar year, a plan of action is required to be submitted to the Department of Environmental Protection (DEP) by March 31st of the following year. The treatment plant is designed to be able to convey flow at an average daily flow of 2.00 mgd. The design and permitted flow rates are summarized in the table below and the NPDES permit is included as an attachment to this document.

Table 1.1 Design and Permitted Flow Rates

Parameter	Permitted Flow (mgd)
Permit	1.56
80% of Permit	1.25
WPCF Capacity	2.00

2. Peak Influent Flows and Non-Permitted Diversions at the WPCF

The WPCF has exceeded its capacity multiple times in the last three years. When the WPCF has sustained flows above the rate that the secondary treatment process can handle while its equalization basins are full, the plant diverts flow to an unlined depression on the site. These diversions are technically non-permitted and must be reported to Mass DEP. A number of these diversions have taken place at the WPCF in the last three years including prolonged diversions in spring 2018 and spring 2019. A photograph of the diversion is presented below.

Figure 2.1 Non-Permitted Diversion at WPCF

The spring 2018 diversion led to the evaluation of and decision to increase the volume of equalization at the plant. The additional equalization basins that are scheduled to be constructed by spring/summer of 2021 will help to limit the future number of diversions at the plant. However, the additional equalization basins will not do anything to increase the capacity that the secondary treatment process can treat or the effluent flow that is able to be discharged to the Agawam River under the permit. The evaluation of the diversions during the March 2018 during the three Nor'easters that struck the area in that month (March 2nd, 7th, and 13th), concluded that 1.3 million gallons of additional equalization would have been necessary to keep the WPCF from exceeding its capacity. The worst of the three Nor'easters in March delivered precipitation equivalent to a 24-hour 1.5 year magnitude storm. An additional capacity of 2.7 million gallons was designed into the new equalization basins to contain flow from a 1.5-year magnitude storm scaled to tolerate increases in precipitation that are projected to occur in 2050. A figure of the additional equalization basin layout is included as an attachment.

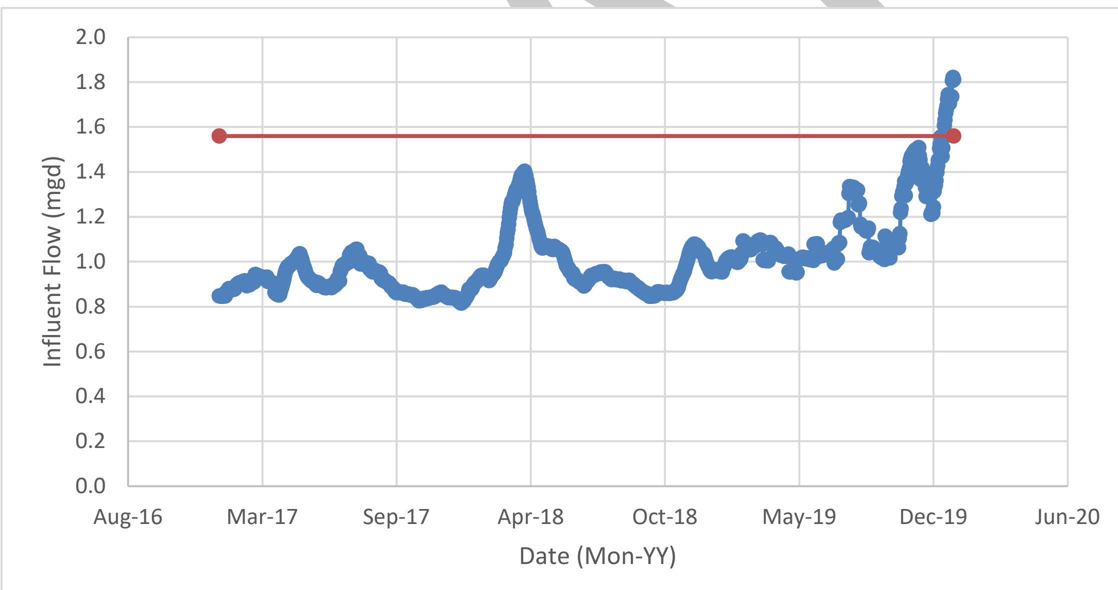
It should be stated that even with the new equalization basins there may be weather events that still result in a non-permitted discharge. However, we expect these events will have to be much larger than the events that have caused previous non-permitted diversions. If we were to design to the 50- or 100-year storm event, the required equalization would require many acres of new basins that would likely remain dry, except for once or twice every 100 years. However, in these larger events, regulatory agencies will likely understand that treatment plants cannot account for these storms.

3. Committed Future Flows and Effluent Discharge Permit

In addition to the diversions from peak influent flow events, the WPCF is also trending to exceed its discharge permit in the future.

3.1 Current Flows

The WPCF has been experiencing a trend of increasing influent flows in the past years. The figure below shows the rolling 30-day average influent flow rate for 2017 through 2019.



Note: The red horizontal line shows the 1.56 mgd permitted effluent discharge rate.

Figure 3.1 Average 30-Day Rolling Flow for January 2017 through December 2019

In the last three years (2017 through 2019) the influent flows have not exceeded the permit on a rolling annual basis. In the month of December 2019 the average influent flow rate was 1.82 mgd. This flow exceeded the monthly reporting value of 1.56 mgd and a letter was sent from the WPCF to the State notifying them. The maximum average rolling 365-day influent rate from the past three years occurred in 2019 and was 1.18 mgd. This flow represents 76 percent of the permitted discharge rate. When the 365- day rolling average of the flow exceeds 80% of the permitted discharge rate, a plan must be submitted to the State outlining a plan of action. While the chart seems to show that the plant did not exceed its capacity and require discharges during spring 2018, the plant did need to discharge flow because there where consecutive days where the capacity was exceeded but the average 30-day flow was still under the permitted flow.

3.2 Committed Future Flow Rates

The town has committed to allowing for an increase in flows to the WPCF (see table below). Although these flows are not depicted in the flow data for 2017 through 2019, they need to be accounted for when planning for future flows and in analyzing the permit. These flows were presented in the Board of Selectmen presentation on February 11, 2020.

Table 3.1 Committed Flows

Committed Future Flows	Flow (gal per day)
Bourne (approximate remaining capacity)	100,000
Robertson Plaza/Delta Dental	12,000
Bay Point	37,000
Great Hill Park	20,000
Woodland Cove	32,000
Assisted Living Facility, Sandwich Road	10,140
Chapel Lane	1,700
Minot Forest Condominium's	1,320
Single Family Home – 240 Oak Street	330
Single Family Home – 14 Tremont Street	330
A.D. Makepeace (Rosebrook Building)	60,000
Total	**275,000***
**Actual total is 274,820, rounded to nearest thousand*	

3.3 Future Flows

If the known committed future flows are added to the facility, the influent flow rate could increase by 275,000 gallons per day. These committed future flows could be added at any point. Because these flows have already been approved and the Town has no control over the timing, they should be considered during flow analysis. When the committed future flows are added to the average annual influent flow rate for 2019 (per Section 3.1 above), the influent flow rate would increase to 1.45 mgd. The flow rate of 1.45 MGD represents 93% of the permitted effluent discharge rate. A flow rate of 1.45 mgd would force the town to submit a plan of action to MassDEP, the entity which issued the permit. The NPDES permit requires that the WPCF submits the plan to MassDEP by March 31 of the calendar year following the 80% exceedance. The plan must describe further flow increases and how the WPCF will maintain compliance with all effluent and flow limits.

In addition to the committed flows the Town of Wareham has also allowed A.D. Makepeace to connect to the sewer collection system. A.D. Makepeace has been allowed to contribute their full buildout flow through existing sewer connections. A.D. Makepeace has indicated that their likely flow will be an additional 500,000 gpd in the future, for the Business Development Overlay District (BDOD). The timeline for when the A.D. Makepeace flow would be added is not definite. However, when this flow is added with the additional committed flow the total flow is estimated to be 1.95 mgd. This flow would exceed the discharge limit and put the plant at 98% of design maximum flow capacity.

4. Findings

The WPCF has experienced both an increasing number of peak flow events and an increasing trend in overall influent flow rates. In the last three years (2017-2019) the WPCF has had to discharge untreated wastewater multiple times due to these increasing flow rates combined with I/I flows. A previous study found that the WPCF lacked adequate equalization volume to handle peak influent flow rates during storms such as the 2018 March Nor'easters. Additional equalization basins are scheduled to be built and completed in calendar year 2021 to allow the WPCF to be able to handle high flows. The overall trend in influent flow rates has also been increasing and with 275,000 gallons per day of additional committed flows, the WPCF is at risk of nearing its permitted discharge rate and exceeding 80% of the discharge rate which requires submitting a plan of action to the State.

If, during this or a future calendar year, the Town exceeds 80% of the permitted discharge flow, the permit allows only three months between the exceedance of 80% of the discharge rate and date of plan submittal. For calendar year 2019 the Town was at 76% of their permitted discharge flow. If the 275,000 gpd of committed flows were online, the Town would have been at 93% of the permitted discharge flow for 2019, and the plan of action would have been due in March of 2020. Additionally, if all of the allowed A.D. Makepeace flow was also added the WPCF would exceed its discharge capacity and be at 98% of its design flow capacity.

While the commitments are not actual flows and do not trigger the required plan of action, the Town needs to start developing this plan of action, as the Town has committed well over 80% of their permitted discharge flow.

While the Wareham WPCF is designed to convey up to 2.0 MGD of wastewater, there are two processes that are undersized; the equalization basins, which were identified as being undersized in a previous study and may be further undersized as flows exceed the design average of 1.56 mgd, and denitrification filters, which are lacking a backup required by current standards. The diversions required in the spring of 2018 and 2019 were a result of these two undersized processes flooding the WPCF grounds, and potentially resulting in a raw sewage spill to the Agawam River.

5. Recommendations

Our recommendations are based on managing the risks associated with the flow conditions at the Wareham WPCF. Having documented known non-permitted diversions with MassDEP, certain actions must be initiated at the WPCF, and several have been including the new equalizations basins and planning for a new denitrification filters. Once these improvements are online, the risk of non-permitted diversion will still exist, however at a reduced level. It is our goal to recommend a flow management policy that will allow the Town to grow while minimizing risks of non-permitted diversion. However, some immediate actions are strongly recommended:

- Given that the current flows and commitments have well exceeded 80% of the total permitted discharge flows and having a known condition where the WPCF is performing non-permitted diversions to manage elevated I/I flows, we cannot recommend adding any additional flow (committed or not), as the potential for non-permitted diversions will be exacerbated. Further detail:

- It is recommended to not increase flows through additional connections at least until the new Equalization Basins 3 and 4 are online; the expected date of operation is spring/summer of 2021.
- It is also recommended to not add any new flow until the new denitrification filter is installed. This project is in its planning stages and the new filter is expected to be online in late 2022 (provided the Town acquires SRF funding in 2021). However, once Equalization Basins 3 and 4 are installed, the risk of non-permitted discharge will be reduced. The installation of the new denitrification filters will further reduce the risk of non-permitted diversions.
- Even with the new equalization basins and new denitrification filters, there may be weather events that still result in a non-permitted discharge. However, we expect these events will have to be much larger than the events that have caused previous non-permitted diversions.

- Although any future connections to the collection system will require Town approval, it appears as though the Town has overcommitted the discharge capacity of its wastewater treatment facility. Current flows and all committed flows (1.95 mgd) well exceed the permitted discharge capacity of the facility (1.56 mgd).
- It is recommended to install a flow meter at the upstream manhole of the influent line of the Cohasset Narrows pump station to confirm the amount of flow from the Town of Bourne. This meter should be in place for spring and summer of 2021 to get a comprehensive reading of seasonal and non-seasonal flows.
- The Town should continue efforts to expand discharge capacity either through permit modification or a new effluent discharge site.
- The town should complete the Comprehensive Wastewater Management Plan (CWMP) update which would include projections for future flows. These projections are critical for planning associated with future effluent discharge sites or permit flow expansions, cost-effective upgrades at the WPCF, and responses to EPA if the WPCF exceeds it permitted flow. Additionally, completion of the CWMP is one of five requirements to receive 0% financing through the State's Revolving Fund program (for example, a 2% loan for 20 years is roughly $180,000 for every $1M of loaned monies).

The Wareham WPCF is currently undergoing a full evaluation. A flow evaluation is only one means of evaluating a wastewater treatment facility. It is possible that further limitations on capacity may be discovered including treatment capacity. This may lead to further restrictions, such as further flow restrictions based on excessive loads, once the evaluation is complete. This memo will be updated if further restrictions are found.

Appendix B
Meeting Minutes November 17, 2020

Minutes

December 10, 2020

Subject/Client:	Wareham WPCF – Denitrification Filter and Odor Control	Ref. No.	11217251
From:	LQ Lenna Quackenbush	Tel:	774-470-1654
Venue/Date/Time:	Microsoft Teams; November 17, 2020 @ 9:00 a.m.		
Copies To:	All Attendees		
Attendees:	Guy Campinha (Town of Wareham) Russ Kleekamp (GHD) Marc Drainville (GHD) Sara Greenberg (GHD) Lenna Quackenbush (GHD)	Absent:	

This meeting included a powerpoint presentation which is included as an attachment. The presentation includes notes for this meeting.

A. DENITRIFCATION FILTER

1. Ms. Quackenbush presented the background on the existing denitrification filters and current problems.
2. Ms. Quackenbush explained the options to add addtitional filters, including one additional filter, two addditional filters, or three additional filters.
3. Mr. Campinha explained that the filters are not treating to the design conditions. Around four years ago Leopold representatives came to look at the problem but could not provide an answer as to why.
4. Mr. Campinha decided to proceed with the design of three additional filters.

B. ODOR CONTROL

1. Ms. Greenberg presented the background on the odor problems and complaints at the WPCF.

C. ADDITIONAL 5TH BASIN – COVERED

1. Ms. Quackenbush presented the option to build a fifth smaller basin, which is covered with odor control.

D. ADDITIONAL 5TH BASIN – AQUASTORE TANK

1. Ms. Quackenbush presented two options to add aquastore covered glass lined tanks.
   a. The first option would be a 150,000 gallon fifth basin.

b. The second option would be a 1.2 million gallon aquastore tank to replace one of the existing basins. A pump station would be needed for this option.

E. COVER EXISTING BASIN

1. Ms. Greenberg presented the options for removable floating covers. None of the options seem to work.
    a. Mr. Campinha explained that there had been grease built up under the hexagonal covers.
2. Ms. Quackenbush presented covering the existing basins with aluminum covers.
    a. Mr. Campinha mentioned that he had looked into buying the residential properties near the site.

F. CHEMICAL ADDITION

1. Ms. Greenberg presented the chemical addition options that were considered.
2. Bioxide
    a. Mr. Campinha mentioned that he had added piloted bioxide previously.
    b. Mr. Campinha said that bioxide had not worked previously; he added that bleach had worked better than the bioxide.
    c. Mr. Campinha preferred the covered basin options over chemical options.

G. ODOR MONITORING

1. Ms. Greenberg explained the options to monitor odors.
    a. Mr. Campinha agreeded that he thought odor monitoring would be good.
        i. The monitoring could help the plant operators.
        ii. The monitoring could also help provide answers to residents and real-time data.
            (a) Using SCADA the WPCF could potentially make the data available to residents.
            (b) Including wind speed, direction, and odor levels in real-time could answer questions on if an odor complaint originated from the facility or not.

H. LONG TERM PLANNING

1. Mr. Drainville introduced long-term planning for the WPCF.
2. Mr. Campinha liked the idea to look into MBRs.
3. Mr. Campinha mentioned the issues with existing clarifiers.

☒ Attachments: PowerPoint Presentation 11-17-2020

Wareham WPCF Evaluation:

Denitrification Filter Addition and Odor Control Options

November 17, 2020

Session Agenda

1. Denitrification Filters
2. Odor Control – New Basins
3. Odor Control – Cover Existing Basins
4. Odor Control – Chemical Addition
5. Odor Monitoring
6. Long-Term Planning

Denitrification Filters

Background – Denitrification Filters

- 3 Leopold Filters
- Each Filter can treat 0.66 mgd
- Updated TR-16 guidelines require treatment of peak flow with one filter backwashing

Denitrification Filter Addition

	Zero Filters	1 Filter	2 Filters	3 Filters
Rational	Filters do not meet updated guidelines; filters are backing up; acting as bottleneck	Meet updated guidelines	Meet updated guidelines with additional redundancy	Meet updated guidelines with additional redundancy
System Design Peak Flow*	1.3 mgd	2.0 mgd	2.6 mgd	3.3 mgd
Cost	$0	$1,500,000	$2,700,000	$3,500,000

**Peak flow calculated with one filter backwashing*

Denitrification Filter Addition

Conclusions and Decisions:

- 3 Filters addition is preference
- Current filters are not meeting more than 1.1 – 1.2 mgd with all three operating – none in backwash
- Backwash with dirty water – can this be fixed/modified?
- Sump pump issue
- Clearwell
- Explore well for backup water

Odor Control

Background

- Odor complaints
- Storage of raw wastewater
- Low tide
- 2016 Odor Study Perimeter Sampling

New Covered Basins

5th Basin

- 250,000 gallon capacity

Advantages	• Aluminum cover to reduce odors • Stores daily peak events
Disadvantages	• Requires addition of odor control (biofilter) • Smaller capacity • Wet weather events still require using un-covered existing basins
Cost	$2,000,000
Operating / Maintenance Costs	• Open covers and clean basin periodically • Replace biofilter media after 2 to 3 years

Aquastore®

- 150,000 gallon basin
- Glass-fused-to-steel tank

Advantages	• Steel enclosed tanks • Stores daily peak events or large wet weather events • Sloped bottom – maintenance free
Disadvantages	• Requires addition of odor control
Cost	$2,500,000
Operating / Maintenance Costs	• Replace biofilter media after 2 to 3 years • Manway openings for maintenance as needed

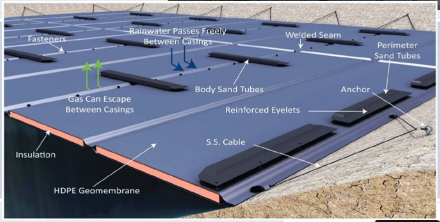
Aquastore®

- 1,200,000 gallon basin
- Glass-fused-to-steel tank

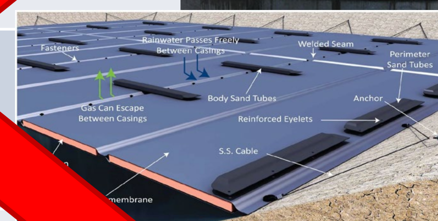
Advantages	• Steel enclosed tanks • Stores daily peak events or large wet weather events • Sloped bottom – maintenance free
Disadvantages	• Add pump station • Requires addition of odor control
Cost	$4,500,000
Operating / Maintenance Costs	• Replace biofilter media after 2 to 3 years • Manway openings for maintenance as needed

Cover Existing Basins

Floating Covers

Company	Cover	
Industrial & Environmental Concepts (IEC)	Modular Floating Cover	
Evoqua	Geomembrane Gas Collection Cover	
Lemna Technology	Modular Floating Cover	
AWTTi	HexaTile Cover	
Disadvantages	• Difficult to remove cover for cleaning (typically removed only every few years) • Cannot be used in applications with changing water elevations	

Floating Covers

Company	Cover	
Industrial & Environmental Concepts (IEC)	Modular Floating Cover	
Evoqua	membrane Gas ction Cover	
Lemna Technology	Modular Floating	
AWTTi	ile Cover	
Disadvantage	• Difficult to remove cover for cleaning (typically removed only every few years) • Cannot be used in applications with changing water elevations	

Aluminum Covers

Advantages	• Aluminum Cover to reduce odors • Stores daily peak events and wet weather events
Disadvantages	• Requires addition of odor control (biofilter) • Expensive
Cost	$4,700,000 (1 Basin) $9,400,000 (2 Basins)
Operating / Maintenance Costs	• Open Covers and Clean Basin Periodically • Replace biofilter media after 2 to 3 years

Odor Control Options

Conclusions and Decisions:

- Covered tank
- 1.1 mgd capacity
- Conical bottom is preferential
- Will require pump station to pump into the tank (due to the height of the storage tank)

Chemical Addition

Chemical Odor Control

Chemical		Advantages	Disadvantages
Hydrogen Peroxide	Oxidizer	• Effective and simple	• Requires short detention times (< 4 hours) • High cost • Safety considerations
Magnesium Hydroxide (Thioguard)	pH Elevation	• Raises pH higher than 8 • Reduces amount of alkalinity addition • Reduction of fats, oils, and grease • Provides treatment during long detention time	• Slurry – difficult to store and feed • Requires freeze protection • Due to pH self-buffering, difficult to detect when too much product added • Difficult to control automatically
Calcium Nitrate (Bioxide)	Alternate Oxygen Source	• Curative and preventative H2S production • Dissolves fats, oils, and grease • Low freezing point • Safe to handle	• Adds nitrates to the WW • Lowers pH; requiring addition of alkalinity • Requires fine tuning to achieve sweet spot • Prevention vs. Removal

Calcium Nitrate (Bioxide) - Pilot

- **Pilot Study**
  - H2S measurements before
  - Chemical Pump
  - Storage/Feed tank
  - Mixer to dissolve dry product
  - Shed to house equipment
  - H2S measurements after
- **Costs**
  - Equipment, Startup, Chemicals:
    - $150,000 (1 month of chemical)
- **Operation Costs**
  - Chemical Cost:
    - $36,000 per year

Odor Control Options

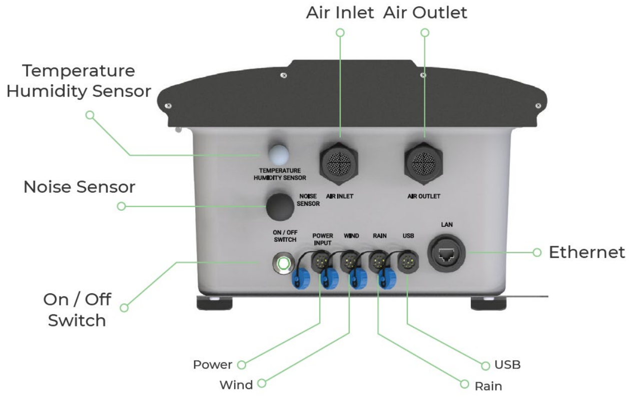
Conclusions and Decisions:

- Covered Basin at 1.1 to 1.2 mgd capacity is the preference
- Evaluate fully the Aquastore/conical tank
- Will require pump station to pump into the tank (due to the height of the storage tank)
- Connect to existing second basin for overflow

Odor Monitoring

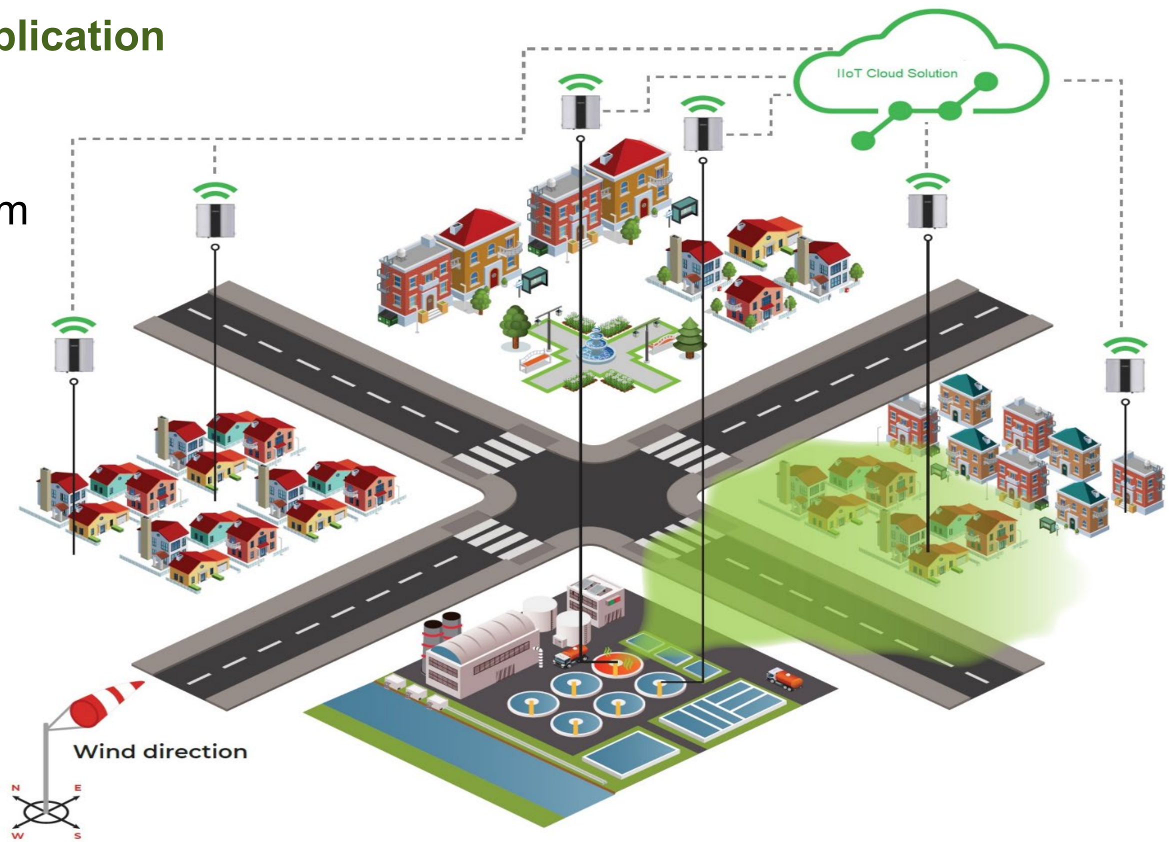
Odor Monitoring – Digital in Real-Time

- Real-time monitoring of meteorological parameters
  - wind speed and direction, solar radiation, season, mixing layer, depth, etc.
- Data-driven odor analysis
  - Odor sensors at the pumping stations
  - Air pressure chambers around the WPCF
- Identify and quantify specific odorful gases and identify potential odor source



Odor Monitoring – Proposed Solution

- Sensor Layout for Odor Detection
  - Real-time monitoring
  - Data sent to cloud application every 24 hours
- Alarms & Notifications
- Remote Monitoring Platform



Appendix C
Meeting Minutes December 3, 2020

Minutes

December 15, 2020

Subject/Client:	Wareham WPCF – Denitrification Filter and Odor Control	Ref. No.	11217251
From:	LQ Lenna Quackenbush	Tel:	774-470-1654
Venue/Date/Time:	Microsoft Teams; December 3, 2020 @ 9:00 a.m.		
Copies To:	All Attendees		
Attendees:	Guy Campinha (Town of Wareham) Russ Kleekamp (GHD) Marc Drainville (GHD) Sara Greenberg (GHD) Lenna Quackenbush (GHD)	Absent:	

This meeting included a PowerPoint presentation – please see attached.

A. ODOR CONTROL

1. Ms. Greenberg presented the odor control options and costs. The options considered included a fifth basin, a small aquastore tank, and a large aquastore tank. Costs for odor monitoring and denitrification are included in all odor control options as both will be pursued in the next phase of design.
2. Mr. Campinha asked about the legal, fiscal, and construction phase services. Mr. Drainville explained that the design costs were not included in the allowance.
3. Mr. Campinha asked about the odor monitoring monthly costs. Mr. Drainville explained that the standard is to use a cellular plan for remote locations, GHD is investigating wiring some of the monitoring units to reduce the need for cellular plans.
4. Mr. Campinha decided to continue with design of the fifth basin option.
5. A summary of decisions for the next stage of design are as follows:
    a. Addition of three denitrification filters
    b. Addition of a fifth Equalization Basin with an approximate 250,000 gallon capacity. This basin would be covered and be accompanied by odor control.
    c. Odor monitoring of the site (perimeter and select point locations to be determined in the next stage of design).

B. SOLIDS INVENTORY

1. Mr. Drainville presented the solids inventory graph from 2017 through October 2020. The graph showed MLSS concentration, percent solids concentration in thickened sludge, and sludge disposal

in pounds. The graph seemed to show a relationship between the decrease in thickened solids concentration and the rise in the MLSS concentration in the aeration tanks.

2. Mr. Campinha explained that adding additional PAC interfered with the polymer's ability to solidify. Mr. Campinha mentioned that a period of instability also coincided with increases in phosphorus influent.
3. Mr. Campinha mentioned that sludge disposal used to be 36,000 gal per day and is now up to 70,000 gal per day but the plant is unable to keep up with the disposal needs.
4. Mr. Campinha said that the plant is hauling two to three trucks a day, trucking twice as much as the plant wants to which increases cost. Mr. Campinha mentioned the cost per truck is about $2,000.
5. Mr. Campinha asked where the solids are coming from.
   a. Septage – increased septage delivery. Possibly due to more people being home and pumping septic systems. Oak Bluffs will start to deliver septage in the coming months
   b. Possibly increased grease.
   c. Wasting from clarifiers.
6. Mr. Campinha mentioned that the sludge storage tank volumes are maxed out. Because the plant is wasting so much and the sludge tanks are full, the time to settle in the sludge holding tanks is reduced and they cannot decant from the sludge holding tanks. The WAS to the thickener contains more water and therefore the final thickened sludge contains more water. The plant adds polymer to solids settling tank but the process needs 24 hours to settle with polymer.
7. Mr. Campinha mentioned new lab operators had joined the WPCFC staff. As such he would like to increase the data collection at the WPCF.
8. The gravity belt thickener may be running too fast which contributes to reduced sludge concentration. However, the GBT needs to run fast to be able to waste the sludge that the plant needs to waste within the daily operational hours. It was suggested that training for gravity belt thickening operation may be useful for new employees.
9. Mr. Campinha mentioned that currently there is five feet of solids blanket in the clarifiers; now that PAC addition has reduced, the clarifiers seem to be settling better.
10. Mr. Campinha mentioned that the internal recycle at the plant has been reduced.
11. Grit removal units are failing which may be adding to the solids in the system.

C. UNIT PROCESS EVALUATION

1. Ms. Greenberg explained the unit process evaluation and criticality matrix for the WPCF evaluation.
2. Mr. Campinha provided comments about all unit processes at the treatment facility. The comments were recorded in the PowerPoint.
3. The comments from this meeting will be used to develop a priority list for needs at the treatment facility.

☒ Attachments: PowerPoint Presentation 12-3-2020

Wareham WPCF Evaluation:

Odor Control, Sludge Inventory, and Evaluation

December 3, 2020

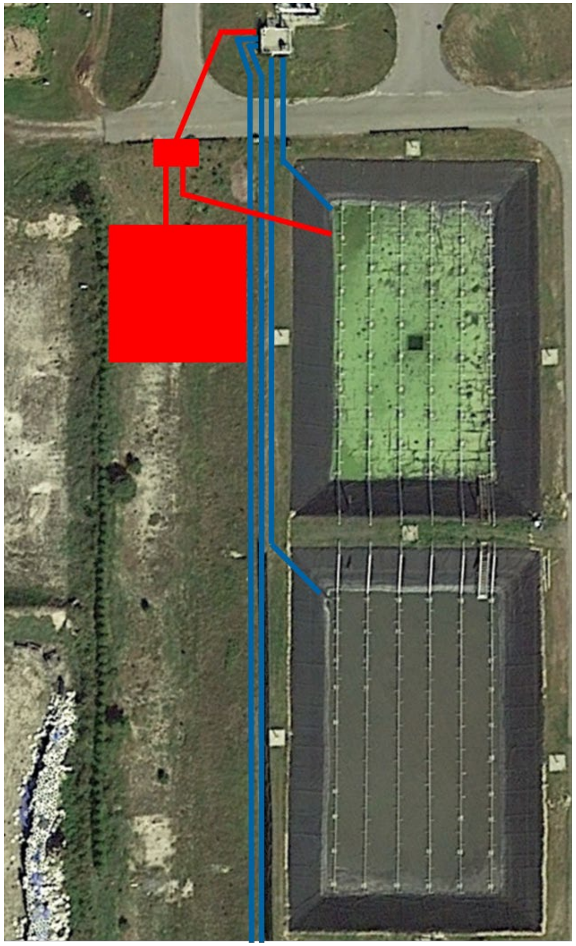
Session Agenda

1. **Odor Control Option Costs**
   - 5^{th} Basin
   - Small Aquastore Tank
   - Large Aquastore Tanks
     - Odor Monitoring
2. **Solids Inventory**
3. **WPCF Evaluation**
   - Equipment and Process Assessment

Odor Control

Odor Control Options

Covered 5th Basin
(250,000 gal)

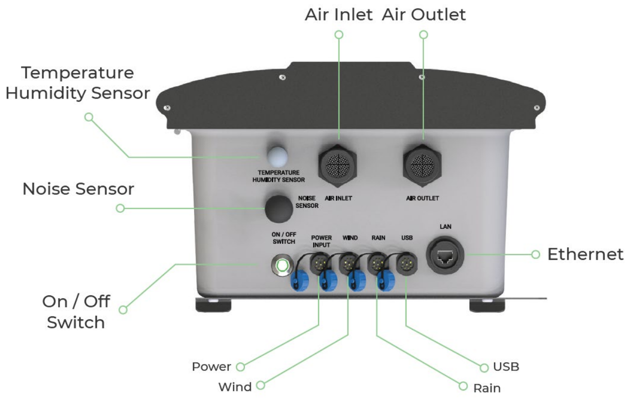


Aquastore Tank
(150,000 gal)

Aquastore Tank
(1,200,000 gal)

Odor Monitoring

- Real-time monitoring
  - Wind and Odor
- Perimeter and within WPCF
- Identify and Quantify:
  - Odors
  - Sources



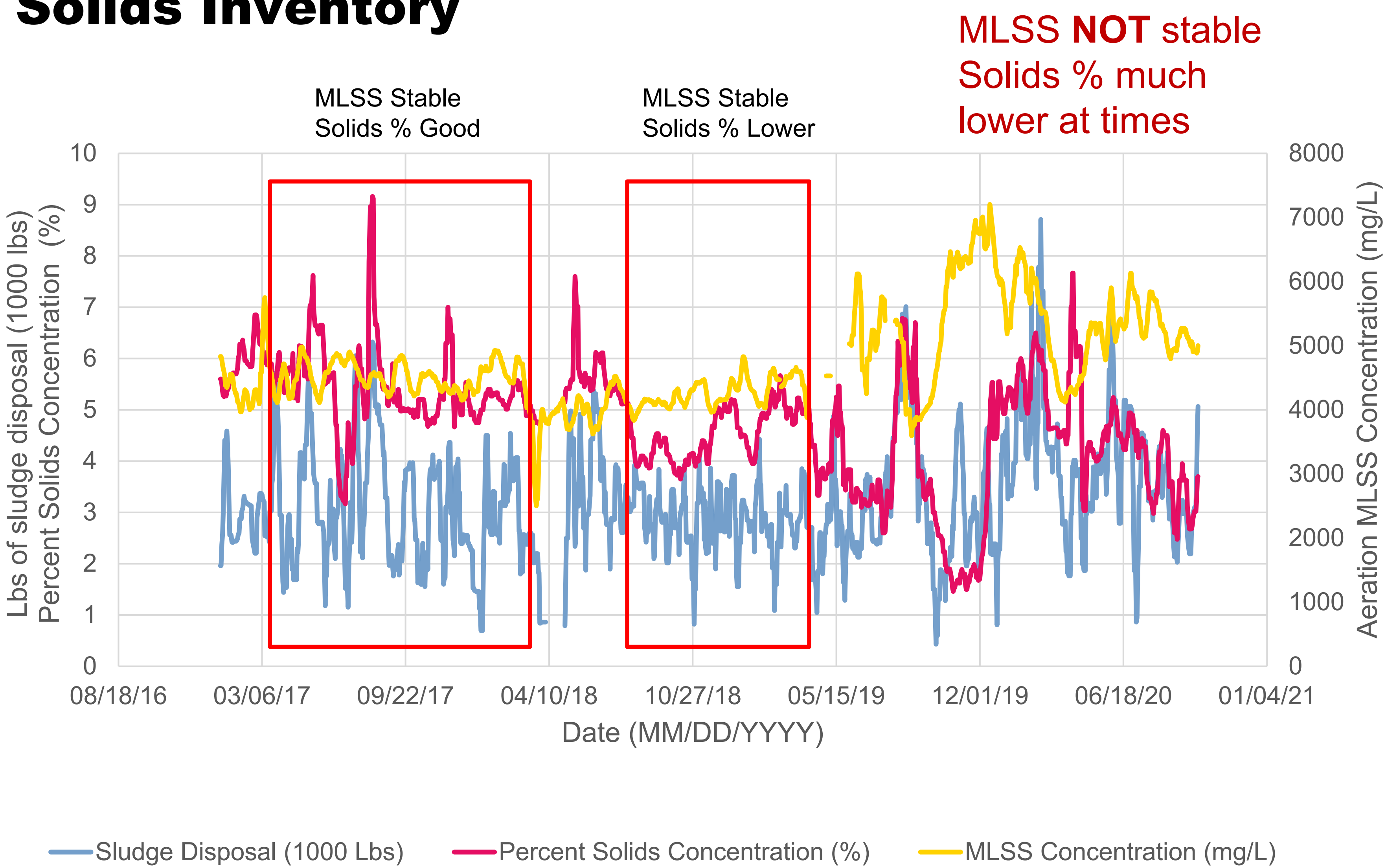
Engineers Opinion Of Probable Costs

Lump Sum Work	Covered 5th Basin (250,000 gal)	Aquastore (150,000 gal)	Aquastore (1,200,000 gal)
Odor Project Subtotal of Project Cost Estimate	**$1,972,000**	**$1,615,000**	**$5,194,000**
Contingency (30%)	$592,000	$485,000	$1,558,000
Odor Project Total Construction	**$2,564,000**	**$2,100,000**	**$6,752,000**
Legal, Fiscal & Construction Phase Engineering (25%)	$641,000	$525,000	$1,688,000
Odor Project Costs (2020 Dollars)	**$3,200,000**	**$2,600,000**	**$8,400,000**
Denitrification Filter Addition	**$3,500,000**	**$3,500,000**	**$3,500,000**
Total Project Costs (2020 Dollars)	**$6,700,000**	**$6,100,000**	**$11,900,000**
Midpoint Construction (Spring 2022)	**$6,950,000**	**$6,330,000**	**$12,350,000**
Costs / Gallon	**$28/gallon**	**$42/gallon**	**N/A**

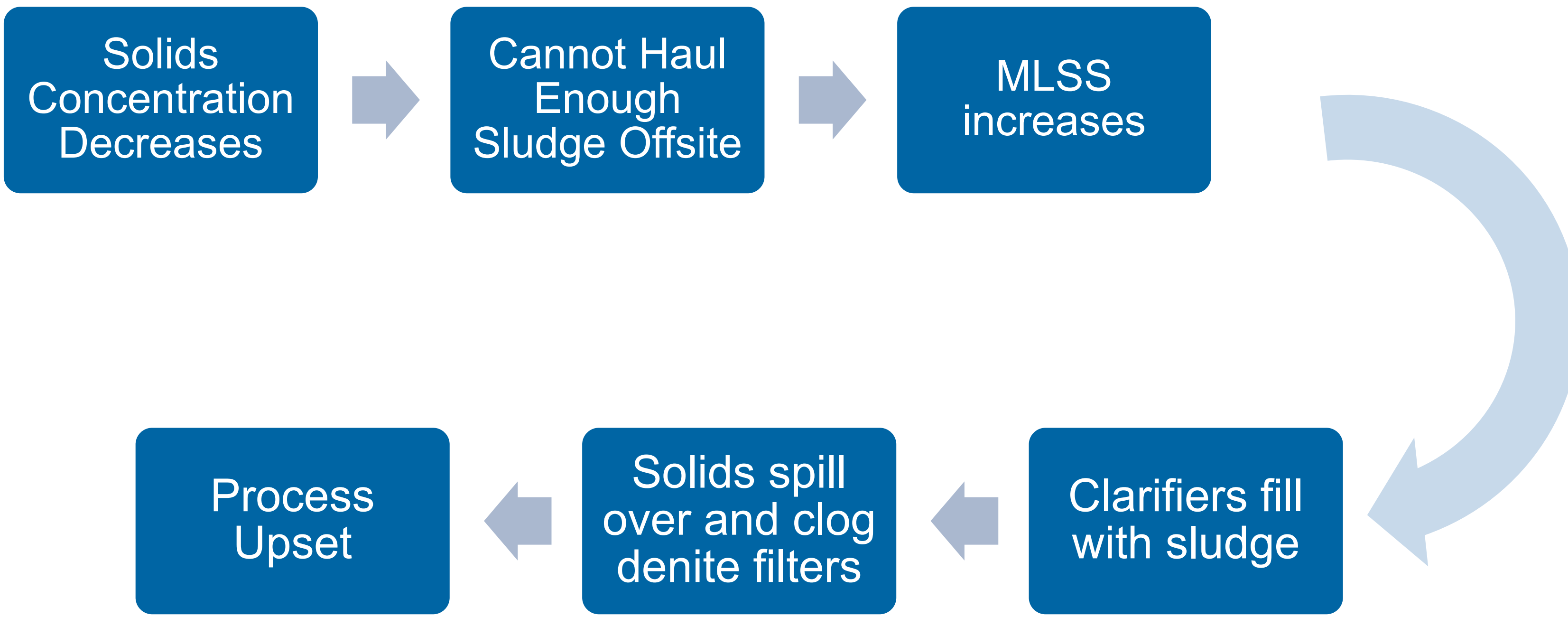
Target $9,000,000
August 2020 (11455)

Solids Inventory

Solids Inventory



Sludge Thickening Issue?



Sludge Thickening Conclusion

- Sludge not thickening enough?

WPCF Evaluation

Unit Process Evaluation

- **SRF Requirement**
  - Description
  - Evaluation
    - Desktop analysis of equipment sizing
    - Flow and load analysis
  - Operational Issues
    - Operator comments (in addition to January 2020 Meeting Comments)
    - Walk-through of plant
  - Recommendations

Criticality Matrix

- Likelihood of Failure
  - Condition Assessment
  - Performance Assessment
- Consequence of Failure

CoF Rating → ↓ LoF Rating	Negligible (1)	Marginal (2)	Critical (3)	Catastrophic (4)
Failing (5)	Medium	High	Very High	Very High
Poor (4)	Medium	High	Very High	Very High
Moderate (3)	Low	Medium	High	Very High
Good (2)	Low	Low	Medium	High
Excellent (1)	Low	Low	Medium	High

Headworks

- Influent D-Box corroded
- Parshall flume errors with reading – calibrated annually
- Influent screen – bearing replaced – working better
- Grit Unit operational issues – Grit in tanks
- Septage units are failing
- EQ Weir needs manual adjustment
- Flow split (at weir) corroded at location prior to going to basins
- Building HVAC (heating) issues (H2S as high as 100) – very corrosive environment
  - Equipment panels are corroding
- Odor control/biofilter status – in process of replacing the media (over winter)

Secondary Treatment

- Anoxic Tanks – tanks need taken down and cleaned, baffle tie-lines and water spray need replaced
  - Mixer not working?
- Aeration Tanks- aeration tank 2 needs rehabbed
  - Walls (leakage and infiltration) – they were re-lined (found exposed rebar in some locations) – structural concerns
  - Aeration Tank 1 was rehabbed (3-4 years ago)
- Secondary clarifiers are too shallow
- Internal recycle pumps leaking – a new pump is arriving (issue with seals). Replacing all of the pumps (in process). Existing pumps are only producing 3x flowrate not 4x flowrate.
- Secondary Clarifier Equipment replacement needed – they are near end of life (drums in 1 and 2 need replacement)

Denitrification Filtration

- Additional filters needed, existing filters are clogging
- Air valves do not operate well in the winter
- Winter issues:
  - Mist issues with false readings (open tanks – enclose?)
  - Backwash more of an issue in winter

UV Disinfection

- Issues addressed by Trojan:
  - Leakage
  - Hydraulics issues
- UV designed for 2 MGD – high flow issues

Septage

- Septage screen is broken
- Septage tanks are broken and leaking
- Structural Tanks from 1972 – some failing in the tanks
- Septage pumps that are old
- Blowers are not working – not mixing properly with lack of air flow/issues with air flow

Solids Handling

- Low solids concentration in 2019 (polymer issue?)
  - Use of emulsion – increase %
- Not enough capacity to store solids – additional tankage for storage
- Odor control/biofilter status – functional – need to rehab the fans – one of which is offline
- Pumps are older – replace as they go (past useful life)

Chemical and Ancillary Systems

- Soda ash lines are clogged – add new lines as they clog
- Does hypochlorite system work – leaking overhead pipes – not currently in use
- *Permag system issues – pumps are from 2000 approx.*
- *Plant water is inadequate – elaborate:*
  - Issues with plant water pumps – new pumps seizing quickly (Tigerflow pumps) – need new motors
  - Run out of water for denite and everything – plant water adequately sized? Run out of water at times – cannot use the plant water for all applications wanted.
    - *Clearwell for denite filters?*
    - *Plant water for other uses?*
    - *On site well? Well wanted to save on cost for constantly replacing equipment and parts.*

Other Issues - WPCF Evaluation

- HVAC issues?
  - Administration Bldg – half heats half stays cold – has had HVAC looked at and it is not evenly supplied throughout building
  - Headworks Building – not working (pumped from blower bldg.)
  - Dewatering building, ops building – issues with the system in general
- Alarm system – old wiring – update (new fire codes)

Questions

Appendix D
Basis of Design Memorandum for Denitrification Filter Addition

TECHNICAL MEMORANDUM

January 26, 2021

To	**Town of Wareham, MA**		
Copy to	**Town of Wareham Denitrification Filter and Odor Control**		
From	**Kyle King P.E.**	Tel	
Subject	**Basis of Design Memo** **Denitrification Filters**	Job No.	**11221642**

1.0 PURPOSE OF MEMO

The purpose of this memorandum is to establish the process basis of design for the addition of 3 new denitrification filters to the existing denitrification filter process. The basis of design for Electrical, Heating Ventilation and Air Conditioning (HVAC), Structural/Civil, and Architectural disciplines will be provided separately (as required).

2.0 CODES AND STANDARDS

The following design guidelines and standards have been adopted for this project:

- TR-16 Guides for the Design of Wastewater Treatment Works (2016 Edition)

3.0 BACKGROUND

The denitrification filter process of the Town of Wareham Water Pollution Control Facility (WPCF) currently consists of three filters. The filters were constructed in 2005 and were manufactured by Leopold/Xylem. The three filters are designed to treat a maximum flow rate of 0.67 MGD each, and a combined flow of 2.00 MGD. The filters are configured for continuous operation with automatic backwashes set to occur every twelve hours or when headloss from filter fouling causes the filter level to reach the high-level setpoint. The filters do not have a backup.

Current design standards in TR-16 call for the filters to be designed with one being out of service. The current design does not allow for this. At times the flow entering the filters may contain elevated suspended solids due to periodic process upset conditions and the lack of a backup filter presents operational problems with the filters. The increased solids load to the filters causes accumulation of filter headloss at a higher rate than experienced under normal operating conditions. Filter headloss accumulation causes the filter levels to rise until the level in one of the filters reaches a high-level setpoint and automatic backwash of the filter is triggered. When one filter is backwashing, the plant must feed the full process flow through only two filters in forward operation. This situation creates a negative feedback loop where the filters in forward operation experience an additional increase in hydraulic and solids loading from the initial condition which triggered a backwash. The filters in forward operation rapidly become fouled to the extent that they fail to pass the full process flow. The level of the filters in forward operation rises when they cannot process the full forward

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flow and ultimately results in filter overflow. Overflow from the filters is an unpermitted discharge and could potentially result in penalty to the WPCF. Expansion of the denitrification filter process capacity is expected to help alleviate the hydraulic bottleneck caused by the negative feedback loop experienced under process upset conditions and reduce the likely hood of future filter overflows.

TR-16 guidelines have been updated since the design of the filters in 2001 and now recommend a level of filter redundancy which provides sufficient capacity for treatment of the maximum process design flow while one filter is backwashing. The WPCF will require at least two additional filters to meet the new TR-16 standard. The addition of 3 new denitrification filters is appropriate based on the hydraulic loading ranges provided in the Leopold Denitrification Filter Operations and Maintenance (O&M) Manual.

4.0 SUMMARY OF EXISTING FACILITIES

The denitrification filter process consists of three sand filters (Effluent Filters No. 1, No.2 and No. 3) in parallel configuration. The denitrification filters receive flow from the secondary clarifiers. There are two chemical injection points on the line from the secondary clarifiers to the denitrification filters; the first is for alum and the second is for methanol. Flow from the secondary clarifiers enters the center of the common filter influent channel through a penetration in the bottom of the channel. The common influent channel feeds the influent channel of each filter. Each filter is equipped with a slide gate (SLG 610, 620, and 630) which allows each filter to be isolated from the common influent channel during backwash. Filter influent travels from each filter influent channel into troughs on either side of the filter. Flow travels from the troughs over an adjustable weir and onto the filter sand media.

Suspended solids are removed via physical capture and nitrate is removed via biological denitrification process as water travels downward through the sand media. An undrain system collects water which has passed through the sand media and routes it to the filter effluent pipe. Effluent from each filter combines in a common header which is configured to send flow to the Clearwell or the UV System.

Filters are automatically backwashed on a set time interval (currently every 12 hours per operator input) or when filter level reaches a high-level set point. The backwash sequence initiates isolation of the filter in backwash wash by closing the influent and effluent valves of the filter and closing the slide gate which separates the filter influent channel from the common influent channel. The backwash process consists of air scour for approximately 2 minutes, followed by a combination of air scour and wash water for 15 minutes, and concludes with wash water for 5 minutes. Backwash air is supplied to the backwash air header by two blowers (Air Scour Blower No. 1 and No. 2) operating in a duty/standby configuration. Backwash water is pumped from the Clearwell to the filter effluent line by two backwash pumps (Filter Backwash Pumps No. 1 and No.2) operating in duty/standby configuration. Backwash water flowrate is measured by a magnetic flow meter and adjusted with a flow control valve. Backwash air and water flow upward through the filter dislodging solids and gas bubbles which have accumulated in the filter. The backwash water containing the dislodge solids flows back to the influent channel of the filter and exits to the backwash waste line through and opening in the bottom of the filter influent channel. Backwash waste is collected in the Mudwell and then pumped to the Septage and Sidestream Equalization Tank by two waste backwash pumps (Wash Backwash Water Pumps No. 1 & No.2) operating in a lead/lag configuration.

5.0 PROCESS/EQUIPMENT DESCRIPTION AND DESIGN CRITERIA

The purpose of this section is to describe the denitrification filter process and design criteria. This section has been divided into the following sections to facilitate review:

5.1 – FILTRATION

5.2 – BACKWASH

5.3 – METHANOL DOSING

5.4 – PIPE ROUTING & FLOW CONFIGURATION

5.5 – CONTROLS

5.1 FILTRATION

This section describes the denitrification filter process and design criteria in terms of forward flow only.

5.1.1 Denitrification Filter Process Description

The denitrification filters at the WPCF are gravity, downflow, packed-bed systems which utilize biological denitrification to remove nitrogen (primarily in the oxidize form i.e., nitrate) which was not removed in the upstream biological treatment system to below the permit level. The primary nitrogen removal mechanism is biological denitrification whereby microorganisms present in the filter convert the oxidized nitrogen to N_2 gas. N_2 gas which accumulates in the filter is removed during filter backwash (refer to section 5.2). The filter's packed bed, comprised of coarse sand media, provides the surface for attached-growth of the denitrifying microorganisms and also physically captures suspended solids. The deep-bed configuration provides the necessary hydraulic detention time for the denitrification process to occur. The majority of the degradable carbonaceous material in the process stream is removed in upstream biological treatment; therefore, addition of supplemental carbon to the denitrification filter influent is required. The supplemental carbon source currently used for the denitrification filters at the Wareham WPCF is methanol. Methanol dosing design considerations are captured in section 5.3.

5.1.2 Denitrification Filter Design Criteria

The purpose of the denitrification filtration process is designed to provide a final treatment step to ensure the WPCF meets its effluent total nitrogen (TN) limit and total suspended solids limit (TSS). The TSS and TN Limits per the plant National Pollutant Discharge Elimination Permit (NPDES) are provided in Table 5.1:

TABLE 5.1: NPDES Permit Limits for TSS and TN

Permit Parameter	Effluent Limits			
	Average Monthly	Average Weekly	Average Monthly	Average Weekly
TSS	130.1 lbs/day	195.3lbs/day	10 mg/L	15 mg/L
Total Nitrogen (Aptil 1 - October 31)	52 lbs/day	NA	4 mg/L	NA
Total Nitrogen (November 1 - March 31)	Report lbs/day	NA	Report mg/L	NA

The WPCF is designed such that the Denitrification Filters should not experience a peak flow rate in excess of 2.0 MGD, this is accomplished by diverting flow in excess of 2.0 MGD to equalization basis during periods of high flow. The existing filters were design to handle an average influent flow of 1.5 MGD and a maximum flow of 2.0 MGD. The TSS design load to the denitrification filters was determined based on historical data collected at the WPCF from October 1st, 2017 to Spethember 30th, 2020. The TN design load was determined based on Biowin Modelling performed by GHD as part of a WPCF Plant Evaluation and Fiscal Sustainability Report conducted in 2021. A summary of filter design flows and loads are presented in Table 5.2.

TABLE 5.2: Filter Influent Design Flows and Loads

Parameter	Average	Max Flow
Flow (mgd)	1.5	2.0
TSS, mg/L	--	9.4[(2)]
TN, mg/L	--	7[(3)]

GHD

5.1.3 Existing Filter Dimensions

A summary of the existing Filter Dimensions is provided in Table 5.3. The values provided in table 5.3 were used for the filter capacity evaluations in subsequent sections.

TABLE 5.3: Exiting Filter Dimensions

Parameter	Value	Units
Filter Length	16.0	ft
Filter Width	9.5	ft
Surface Area (1 Filter)	152.0	ft^2
Surface Area (3 Filters)	456.0	ft^2
Filter Sand Media Depth	6.0	ft
Sand Media Volume (1 Filter)	2,736.0	ft^3
Sand Media Volume (3 Filters)	8,208.0	ft^3

5.1.4 Filter Loading Design Criteria

The primary denitrification filter design parameters are hydraulic loading rates and nutrient loading rate. A summary of typical design criteria for downflow denitrification filters is presented in Table 5.4:

TABLE 5.4: Typical Denitrification Criteria

Parameter	Low End	High End	Units	Source
Hydraulic Surface Loading Rate	1.0	2	gpm/ft2	Leopold O&M Manual
	2.4	4.8	m/h	Metcalf & Eddy 5th Edition
Empty-Bed Contact Time	30.0	NA	minutes	Leopold O&M Manual
Nitrate Removed Per Filter Surface Area	NA	0.5	lbs/ft^2-day	Source: Leopold Manual
Nitrate Removed Per Filter Unit Volume	NA	70.0	lbs/1000 ft^3-day	Source: Leopold Manual

5.1.5 Existing Filter Nutrient Loading

The nutrient-load capacity of the existing filters was calculated under the maximum flow condition of 2.0 MGD and compared to the design criteria presented in Table 5.4. A summary of this comparison is provided in Table 5.5.

TABLE 5.5: Existing Filter Nutrient Load Assessment

Parameter	Design Value	Existing Filter Load @ 2 MGD and 7 mg/L NO_3	Units
Nitrate Removed Per Filter Surface Area	0.5	0.26	lbs/ft^2-day
Nitrate Removed Per Filter Unit Volume	70	14.2	lbs/1000 ft^3-day

The comparison presented in Table 5.5 indicates the existing filters have adequate capacity for treating the design flows and loads, therefore nutrient loading does not need to be considered when evaluating the need for additional filters.

5.1.6 Existing Filter Hydraulic Loading

The hydraulic design requirements for filter systems under TR-16 Design Standards state that "Filter systems should be designed to accommodate peak hourly flows with one unit in backwash mode and to accommodate filters operating at design maximum headloss through filter media." A comparison of the hydraulic design criteria presented in Table5.4 versus the hydraulic loading to the filters under a max flow of 2.0 MGD and 2 filters in operation is presented in Table 5.6.

TABLE 5.6: Existing Filter Hydraulic Assessment

Parameter	Design Value	Existing Filter Load @ 2 mgd and 2 filters in operation	Units
Hydraulic Loading Rate	1.0-2.0	3.4	gpm/ft2
Empty-Bed Contact Time	30	39.3	minutes

The comparison presented in Table 5.6 indicates that, while able to meet empty-bed contact time design criteria, the existing 3-filter configuration cannot meet the design criteria for hydraulic loading rate under the required conditions stipulated by TR-16; therefore, additional filtration capacity should be added to the system.

5.1.7 Filter Hydraulic Loading with Additional Filters

The filter hydraulic loading rate was evaluated for three scenarios including 4, 5 and 6 filters operating in parallel. The number of filters considered to be in operation in each scenario, per TR-16 Standard, would be 3, 4, and 5, respectively. The hydraulic load to the filters is not being increased as part of this design; therefore, the hydraulic capacity of the additional filters was also evaluated at 2.0 MGD. The hydraulic loading rates calculated for each scenario are presented in Table 5.7.

TABLE 5.7: Hydraulic Loading Rates with Additional Filters

Parameter	Number of Filters		
	4 (3 in Operation)	5 (4 in Operation)	6 (3 in Operation)
Hydraulic Loading Rate at 2.0 MGD	2.3	1.7	1.4

The denitrification process cannot meet the design criteria with less than a total of 5 filters installed (4 in operation) at a flow of 2.0 MGD. The filter loading rate falls on the upper-end of typical design criteria with a total of 5 filters installed. The installation of 3 additional filters, for a total of 6 filters, will result in a hydraulic loading rate that falls near the middle of the design criteria. The addition of 3 new filters is appropriate because it comfortably meets the design criteria and allows design and construction of the existing system to largely mirror the existing filters. A mirrored-configuration simplifies the approach to design, construction, and operation and results in proportionally lower costs on a per filter basis relative to the addition of only 2 filters. The addition of 3 filters also provides additional capacity which can reasonably be expected to be utilized in the future expansion of the WPCF.

5.2 BACKWASH

5.2.1 Backwash Process Description.

Nitrogen gas and suspended solids accumulate in the filters as denitrification occurs which increases headloss through the filters over time. Nitrogen gas is typically removed through a process called “filter bumping” in which backwash wash water sourced from the clearwell is pumped upward through the filter for approximately 5 minutes. Filter bumping frequency varies based on filter influent loads but is typically conducted every four to eight hours per Leopold O&M Manual. Filter bumps are design to reduce headloss through the filter but do not completely restore the filter to clean-bed conditions. Headloss through the filters continues to accumulate gradually across consecutive filter bumps.

The gradual accumulation of filter headloss results in filter water level rise which eventually triggers a full backwash of the filter. A full backwash consists of a sequence of steps in which a combination of air from the air scour blowers and wash water from the Clearwell is passed upward through the filter for 20 to 25 minutes. Full backwashes are typically required every 24 to 48 hours per Leopold O&M manual. Successfully executed filter backwashes should return filters to the minimum level associated with normal operating conditions.

Backwash waste generated during the backwash sequences is routed to the mudwell. Backwash waste is then pumped from the mudwell to the Septage and Side Stream Equalization Tank and subsequently flows to the head of the primary biological treatment system.

The current denitrification filter system configuration allows for only one filter to be in backwash mode at any given time. The 3 existing filters and 3 new filters will operate in parallel and will also be configured such that only one filter will be permitted to enter backwash mode at any given time.

5.2.2 Backwash Flow and Air Scour Design Criteria

The Leopold O&M manual defines a typical denitrification filter backwash as consisting of the following steps and associated loading rates:

Step 1: Isolate the filter.

Step 2: Air wash at 5-6 scfm/ft^2 for about 1-2 minutes.

Step 3: Air/water wash at 5-6 scfm/ft^2 air and 6-8 gpm/ft^2 for about 15 minutes.

Step 4: Water wash at 6-8 gpm/ft^2 for 5 minutes.

The typical backwash flow and air scour rates defined above were used to calculate the required backwash flow rate and air scour rate required for the existing filters (surface area of 153 ft^2). A summary of the backwash design criteria and required backwash flow and air scour rates is provided in Table 8:

TABLE 5.8: Backwash Water and Air Scour Design Criteria

Parameter	Value		Units
	Low End	High End	
Backwash Air Design Criteria	5	6	scfm/ft^2
Backwash Airflow Rate Required	760	912	scfm
Backwash Water Design Criteria	6	8	gpm/ft^2
Backwash Water Flow Rate Required	912	1,216	gpm

The 3 existing filters and 3 new filters will be configured such that only one of the 6 filters may be permitted to enter backwash at a given time. The 3 additional filters will have the same surface area as the existing filters; therefore, no additional backwash water or air scour will be required and the capacity of the existing backwash pumps, air scour blowers, and backwash waste pumps does not need to be increased to accommodate the additional 3 filters.

5.2.3 Backwash Clearwell and Mudwell Design Criteria

The TR-16 states that “Clearwells should store, at a minimum, enough water to conduct at least two complete backwashes. Where needed for efficient plant operation, mudwells should have, at minimum, enough volume to store water from at least one filter backwash, with provisions to gradually introduce the backwash waste into the plant or primary clarifier influent.” The TR-16 criteria was used to calculate the required clearwell and mudwell volumes, a summary of the required volume is presented in Table 5.9.

TABLE 5.9: Required Clearwell and Mudwell Volumes

Parameter	Value		Units
	Low End	High End	
Clear Well Volume Required	3,6480	4,8640	gallons
Mudwell Volume Required	1,8240	2,4320	gallons

The capacities of the existing clearwell and existing mudwell were calculated based on the filter dimensions and operating setpoints. The clearwell and mudwell dimensions and operating capacities are summarized in Table 5.10.

TABLE 5.10: Dimensions and Operating Capacities of Clearwell and Mudwell

Parameter	Value		Units
	Clearwell	Mudwell	
Width	23.1	23.1	ft
Length	16.0	16.0	ft
Depth (1)	15.3	15.3	ft
Volume	4,2360	4,2360	Gallons
Low Level Set Point (2)	2	2	ft
High Level Set Point (3)	11	11	ft
Working Volume (4)	2,4864	2,4864	gallons

(1) Depth is average with the assumption that tank bottoms slope evenly from EL. 6.0 to 5.0 across the bottom of the tank. This is a conservative assumption which would result in slightly less total capacity available.

(2) Low level set point could not be confirmed with the plant, Leopold OIT is out of service and it is unclear whether the set points are displayed at the main SCADA screen. Low level was assumed to be 1'3" above the pump intake.

(3) High level set point could not be confirmed with the plant, Leopold OIT is out of service and it is unclear whether the set points are displayed at the main SCADA screen. High level was assumed to be 9' above low level, this coincides with the approximate elevations of the top of the filter media and the center-line of the backwash waste discharge to the mudwell.

(4) Working volume was calculated based on the assumptions made in notes (2) and (3)

Table 5.10 indicates that the clearwell and mudwell are both sized to accommodate approximately one backwash at high-end of the backwash design flow rate. The mudwell meets the TR-16 standard for backwash waste capacity while clearwell has approximately half of the required volume. Although the clearwell is undersized per TR-16, the current configuration of the backwash pumps at the WPCF allows the backwash pumps to make use of water from a city water line if the clearwell is empty or cannot provide enough water for a full backwash. Additionally, a groundwater pump will be installed at the facility to provide an additional source of backwash water which can be used to supplement the clear well (the groundwater well design is described in a separate basis of design document). The WPCF can adequately meet the backwash water demand with the current storage capacity and the availability of alternative water sources. The required volume for backwash supply storage (clearwell volume) and backwash waste storage (mudwell volume) will not increase with the addition of three filters; therefore, no additional mudwell or clearwell capacity is required.

5.3 METHANOL DOSING

The methanol dose required to achieve adequate nitrogen removal in the denitrification filters is based on the nitrogen load to the filters. Typical methanol dosage per the Leopold O&M manual is 3 pounds of methanol per pound of nitrate removed. Methanol dosing requirements may vary from one denitrification filter application to another based filter hydraulics, loading, water quality, and climate. The Leopold system allows methanol dosing to be set manually or paced based on flow through the filters and filter influent/effluent nitrate concentrations.

The Wareham WPCF has historically demonstrated its ability to control methanol feed to the denitrification filters through its strong record of compliance with its total nitrogen discharge permit limit. The WPCF's historic success with methanol dosing control, coupled with the fact that influent flows and loads to the filters will not be changed as part of the project, indicate that no change to methanol dosing capacity or control configuration will be required with the addition of 3 filters

5.4 PIPING CONFIGURATIONS AND FLOW ROUTING

The purpose of this section is to describe changes to process piping and valves around the denitrification filters which will be made to accommodate the addition of three new filters. This section also describes design considerations taken to optimize the new configuration for proper process function.

5.4.1 Influent Flow

Influent piping will be rerouted to accommodate the existing filters and to facilitate the connection of additional filters which may be installed as part of future plant upgrades or expansions. A tee will be installed on the influent pipe adjacent to where it currently enters the clearwell. Both the new and existing filter influent lines will be connected to this tee. A butterfly valve will be installed on the influent line to each filter which will allow isolation of the new or existing filter for maintenance. The influent valves will be equipped with electric actuators and configured for local control or remote control from the filter control panels.

The flow rate to each individual filter is controlled by the elevation set point of the filter weirs. Each filter weir is equipped with mounting slots which allow the weir height to be adjusted as required to achieve equal flow distribution. The influent weirs of the new filters will be installed at an elevation which results in equal flow split between the new filters and existing filters.

5.4.2 Effluent Flow

Effluent flow from the existing filters flows into a common header which discharges to either the UV system or the Clearwell. Direction of effluent flow is controlled by a valve on the inlet to each flow path. Effluent from new filters combine in a common header in the same fashion as existing filters. The effluent header from new filters will be routed through the clear well and connect to the existing effluent header upstream of the valve which controls discharge to the clear well. This effluent piping configuration will allow the new filters to discharge to the UV system or the Clearwell and maintained the same operational flexibility of the existing filter effluent configuration.

5.4.3 Backwash Air

Backwash air piping for new filters will be connected to backwash air header of the existing filters. The backwash air piping for the new filters will be routed through the clearwell to the pipe gallery of the new filters where it will connect to the air header of the new filters.

5.4.4 Backwash Water

Backwash water piping for the new filters will be connected to backwash water header of the existing filters. The backwash water piping for the new filters will be routed through the clearwell to the pipe gallery of the new filters where it will connect to the backwash header of the new filters.

5.4.5 Backwash Waste

The backwash waste piping of the existing filter will remain unchanged. The backwash waste piping of the new filters will discharge to the mudwell on the opposite side of the existing filter backwash waste line.

5.5 FILTER CONTROLS

Functional operation of the existing filters will remain unchanged; however, the following control panel changes will be made to accommodate operation of all 6 filters:

- Existing components of the denitrification filter process which will be common to the new and existing filters (e.g. air scour blowers, backwash pumps, backwash waste pumps, et.) will be relocated to a new control panel which will communicate with the control panels of both the new and existing filters. The common control panel will also house any new common components such as influent control valves.
- The existing filter control panel will be upgraded to equipment which meets current standards.
- A new filter panel will be installed at a location local to the access to the pipe gallery for the new filters.
- The existing control panel and new control panel will be configured in a manner which allows for monitoring and control of all six filters.

6.0 DEMOLITION AND MAINTENANCE OF PLANT OPERATIONS

6.1 YARD PIPING

- Influent piping to the denitrification filters will need to be demolished and reconfigured to accommodate the new filters. Secondary Clarifier effluent must be pumped to the existing filter influent channel to maintain treatment through the denitrification process.
- The methanol injection location will also be demolished when the influent piping is demolished/reconfigured. Methanol feed must be temporarily configured to feed the into the line used to re-route denitrification filter influent.

6.2 MUDWELL

- The drain line from the new filters will be routed through the mudwell and the backwash waste line of the new filters will be connected to the mudwell. It is anticipated that the mudwell will need to be taken out of service for this portion of construction. Backwash wastewater must be pumped directly from the filters to EQ Basin XX while the mudwell is offline.

6.3 CLEARWELL

- The effluent line from the new filters, backwash water line to new filters, and backwash air line to the new filters will be routed through the clearwell. It is anticipated that the clearwell will need to be taken offline for this portion of construction. Backwash capability must be maintained through the use of city/potable water while the clear well is offline.

6.4 CONTROL PANELS

- The existing filter control panel must remain in operation until the new control panel is operational
- During relocation of common filter components, filter backwashes must be executed manually. This will require manual operation of filter backwash pumps, air scour blowers, and backwash waste pumps.
- During relocation of common filter components methanol feed must be maintained. This will require manual operation of methanol feed pumps.

Appendix E
Basis of Design Memorandum for Equalization Basin Odor Control

Memorandum

January 25, 2021

To:	Town of Wareham	Ref. No.:	11217251
From:	Marc Drainville, P.E. BCEE; Russ Kleekamp; Doug Mayer	Tel:	774-470-1647
Subject:	**DRAFT Memorandum for Equalization Basin No. 5**		

1. Introduction

The purpose of this memorandum is to provide a basis of design for the new Equalization Basin No. 5 at the Wareham Water Pollution Control Facility (WPCF).

2. Existing Equalization Basins

Wareham WPCF currently operates two Equalization Basins (EQ Basins), with a third and fourth Basin currently under construction under a separate project. The existing EQ Basins Nos. 1 and 2 were constructed in 2005 and each consist of an excavation with sloping perimeter, a shallow sloped bottom supporting aeration diffuser grids (no longer functional), and a 12-inch thick sand layer covered by an HDPE liner. 6-inch thick concrete slabs reinforce the areas surrounding the influent pipe joints. EQ Basin Nos. 1 and 2 are operated independently of one another and on an as-needed basis, however they are connected by two 12-inch DI overflow pipes.

The existing EQ Basins provide emergency storage for the plant when influent flows exceed the maximum safe capacity of the activated sludge secondary process, approximately 1.7 MGD. When flows downstream of the grit vortex in the Headworks Building exceed 1.7 MGD, wastewater overtops a weir gate and is allowed to flow by gravity to either EQ Basin No. 1 or 2, depending on which manual gate is open.

The new EQ Basins No. 3 and 4 will provide additional relief storage for the secondary treatment process, which has a history of overflows due to excessive hydraulic loading within individual process tanks and channels.

2.1 Discharge Pumping

When high influent flows subside, the EQ Basins are drained over a period of hours to days by a pair of vertical centrifugal pumps located in the Filter Building. Flow is withdrawn from a sump in the center bottom of each EQ Basin, and an 8-inch DI discharge pipes run underneath each EQ Basin to the pumps in the Filter Building. EQ Basin effluent flow joins a 16-inch DI header pipe, and is pumped north across the plant yard, back to a distribution box immediately downstream of the flow meter in the Headworks Building.

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Table 1 – Basis of Design Information for Existing Equalization System

Parameter	Value
No. of Existing EQ Basins	4
Basin Storage Volume (each)	1.1 million gal
Basin Storage Volume (total, 4 basins)	4.4 million gal
No. of EQ Basin Pumps	2
EQ Basin Effluent Pump Capacity (each)	700 gpm
Typical EQ Effluent Flows	700-900 gpm
EQ Basin Effluent Pumps Motor Power	30 HP (each)
EQ Basin Effluent Pump Drive Type	VFD

From discussions with the Owner, the EQ Basin effluent pumps are started manually when an Operator is aware that influent flows have reduced to a safe level. The pumps are controlled by a program in SCADA that modulates their speed – and accordingly effluent flow rate – according to a signal from the influent flow meter. As influent flows reduce, the EQ Basin effluent pump speed ramps up, and vice versa. At approximately 1.7 MGD (or 1180 gpm), the effluent pumps shut down. The EQ Basin pumps are also interlocked with the Main Pump Station pumps so as not to run concurrently.

Typical discharge flows from the effluent pumps were indicated to be between 700 to 900 gallons per minute (gpm).

2.2 Odor Complaints

Wareham WPCF has received complaints and criticism from the Public for foul odors; the source of the odor complaints has yet to be conclusively determined. The Owner has noted that during wet weather conditions, all of the available equalization capacity is utilized and complete drawdown of an Equalization Basin can take a matter of days. With the aeration systems not functioning, it is possible that the uncovered, unmixed, and unaerated Equalization Basins could be contributing to the odor complaints.

As such, one of the primary design objectives of EQ Basin No. 5 is to provide a cover and an active odor control system.

3. EQ Basin No. 5 Preliminary Design

Alternatives were developed to address the Owner's need for a covered, first-priority EQ Basin with odor control as part of a conceptual design phase. The selected alternative is to provide a fifth EQ Basin to supplement and support the existing EQ Basins. EQ Basin No. 5 will be of a smaller volume than EQ Basins No. 1 through 4, however the design will incorporate the following design elements in order to provide effective, flexible, equalization volume that addresses the potential odor issues during most of the year:

- A new distribution box to select EQ Basin 5 (primary), or hydraulic overflow to Basins 1 or 2 (overflow)
- Slide gates to isolate EQ Basins 1, 2, and 5
- Approximate EQ5 volume of 250,000 gal
- Fiberglass reinforced polymer (FRP) basin cover
- Powered blower and activated carbon vessel odor control system
- Submersible mixers to keep solids in suspension
- Sloped tank bottom and collection sump
- Submersible pump station and control panel
- Variable frequency drives (VFDs) to drain basin at controlled flow rate
- Continuous liquid level monitoring by an pressure transducer
- Lifting equipment (e.g. portable davit cranes and bases)

The preliminary Basis of Design information is summarized in Table X.

Table 2 – EQ Basin No. 5 Basis of Design

Parameter	Value
Operational Volume	250,000 gal / 33,500 ft^3
Dimensions	75' L x 60' W x 11' D
High Water Level	24.0 ft (NGVD29)
Low Water Level	15.0 ft (NGVD29)
Approximate basin drain time (from full)	6 hours

3.1 EQ Basin No. 5 Discharge Pump Station

The basis for the decision to install a new discharge pump station for EQ Basin No. 5 is as follows:

1. The destination for wastewater pumped out of EQ Basins 1 and 2 the Headworks effluent distribution box. A clear path to pump directly from EQ Basin No. 5 to the Headworks exists under the road.
2. To connect to the existing pumps in the Filter Building, the EQ Basin effluent piping would have to navigate a crowded area in the yard approximately 15 ft below grade.
3. GHD investigated using an abandoned 24-inch concrete plant sewer pipe as a sleeve to navigate the yard piping congestion. However, the elevation of this pipe is too high in the area that it would need to connect to a gravity drain pipe from EQ Basin 5. Running to and through this sewer pipe would require the Basin bottom to be too shallow to allow for the submersible mixing that the owner has requested.

A pair of submersible pumps will be installed in a sump in the EQ Basin No. 5, with fill sloping toward it. The pumps will be provided with guide rails, a discharge coupling, and hatches in the cover to allow for removal for maintenance and repair. The pump discharge piping will run through the sidewall of the Basin and into a valve and meter box before traversing North under an existing access road, turning to enter the distribution box in the Headworks Building.

The pumps will be controlled by a PLC-based control panel, integrating signals from a continuous level control system (ultrasonic or pressure transducer) and the plant SCADA communication system. Programming will limit the modulate the speed of the pumps based on the influent flow meter signal in order to not overload the secondary process, and an interlock will be programmed to stop the EQ Basin pumps when the Main Influent Pump Station pumps are running.

Table 3 – Pump Station Basis of Design

Parameter	Value
Submersible Pumps, Qty	2
Discharge Pump Max Flow Rate	700 gpm (each)
Approximate Total Dynamic Head	17 ft
Motor power	5.5 HP
Discharge Pipe Diameter	6-inches
Approximate basin drain time from full	6 hours
Pumping termination	Headworks distribution box, downstream of influent flow meter
Proposed Manufacturers	ABS/Sulzer, KSB, Xylem/Flygt

3.2 Mixing and Solids Management

The Owner has stated that the aeration system in the existing EQ Basins has not functioned since it was installed. Allowing raw wastewater to sit in storage for long periods of time with no mixing or aeration can cause the wastewater to become septic and be a source of problematic odors. Additionally, suspended solids can settle out of suspension and become difficult to remove from the shallow-bottom EQ Basins 1 and 2.

For these reasons, it was determined during the conceptual design phase that submersible mixers will be provided for mixing energy to keep solids in suspension. Additionally, the tank floor sloping toward a sump, with a minimum 1:12 slope to keep solids and debris moving downhill as EQ Basin No. 5 is pumped down. A concrete coating will be specified to further reduce friction on the floor surface and provide sealing protection for the concrete Basin. A spray-down system fed by an internal recycle valve on the discharge force main is being considered.

Similar to the submersible pumps, the submersible mixers are mounted on brackets and guide rails for easy lowering and raising out of the Basin, traveling vertically along the side wall and through hatches provided in the Basin cover. The mixers will run when the level in the Basin reaches a setpoint that corresponds/correlates to the minimum liquid cover over the mixer propeller (≈24 inches).

Table 4 – Mixing Basis of Design

Parameter	Value
Number of mixers	2
Mixer Type	Submersible
Minimum water level over propeller top	≈2 ft
Drive Type	Direct
Motor Power	4.7 HP
Starter Type	Full voltage (i.e. constant speed)
Proposed Manufacturers	ABS/Sulzer, KSB, Xylem/Flygt

3.3 Odor Control System

One of the primary features of EQ Basin No. 5 will be its odor control system. The odor control system will consist of the Basin cover, Distribution Box No. 3 cover, a blower to draw air out of the enclosed Basin, FRP/stainless steel ductwork, a single/dual stage activated carbon vessel, and an exhaust stack. Upon reaching a level setpoint, the odor control blower will start and begin drawing air from the variable headspace above the liquid level.

While the Wareham WCPF operates and maintains two sets of biofilter cell odor control systems (for the Headworks and Dewatering processes), activated carbon has several advantages. First, it is better suited than a biofilter for removing hydrogen sulfide (H_2S) odors from raw wastewater. Second, the vessel can be sized for long carbon service life, and when needing replacement, can be performed by an outside vendor. In contrast,

Sizing of the odor control system depends on the ambient H_2S concentration, the volume and flow rate of air to be treated, and the frequency of use.

Basis of design information for the odor control system is summarized in Table 5:

Table 5 – Odor Control Basis of Design

Parameter	Value
Media type	Activated carbon
H_2S Concentration	5 ppm average, 10 ppm peak
Air volume min/max	9,000 ft^3 / 45,000 ft^3
Air changes per hour	6
Required air flow rate min/max	900 SCFM / 4,500 SCFM
Carbon volume	228 ft^3
Odor capacity	0.3 gH_2S/cm^3
FRP vessel dimensions	8’ dia x 9.5’ H
Blower Power	7.5 HP
Proposed Manufacturers	EcoVerde, Integrity Municipal Systems, Daniel Mechanical, Purafil

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