

## TECHNICAL MEMORANDUM

**Date:** April 18, 2024  
**To:** Jack Robinson and Cory Prusha, City of Oak Harbor  
**From:** Tom Giese, PE and Soundarya Krishnamurthy, PE, BHC Consultants, LLC  
**CC:** Steve Schuller, PE and Alexander Warner, PE, City of Oak Harbor  
**Subject:** Oak Harbor Clean Water Facility Nutrient Optimization Analysis

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4/18/2024

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### 1. Introduction

The Washington State Department of Ecology (Ecology) issued a nutrient general permit effective January 1, 2022. The Puget Sound Nutrient General Permit (PSNGP) applies to 58 domestic wastewater treatment plants (WWTP) discharging to Washington waters of the Salish Sea. The purpose of this permit is to improve water quality in Puget Sound to improve the health and population of aquatic wildlife by reducing nitrogen loading. The City of Oak Harbor (City) with National Pollutant Discharge Elimination System (NPDES) permit number of WA0020567 has been identified as one of the WWTPs with small total inorganic nitrogen (TIN) loads. As one of the permittees with small TIN loads, the City must meet specific requirements pertaining to monitoring of nutrients in the influent and effluent, optimizing performance of the WWTP to reduce discharge of TIN, and submitting an approvable all known, available and reasonable treatment (AKART) analysis for evaluating alternatives capable of reducing TIN. With respect to optimization, the permit requires the City to evaluate current process performance, identify and evaluate a list of operational strategies to enhance performance of the existing WWTP with the goal of minimizing discharge of TIN during this first permit cycle. The City must prioritize and update a working list of alternatives for reducing the TIN levels with the existing treatment processes. Of the alternatives identified, those found to exceed a reasonable implementation cost or timeframe may be assigned a lower priority and excluded from the initial efforts but should still be retained on the working list for future consideration.

Once one or more alternatives are selected for implementation, the City must report on the implementation during the first annual report due March 31, 2026. This report must describe the alternative and how it was implemented including:

- Estimated TIN reduction.
- Implementation costs.
- Start date and duration of implementation.
- Description of anticipated and unanticipated challenges.
- Impacts to the overall treatment performance, as a result of implementing the alternative.
- Actual TIN reduction.

The permit also requires the small TIN load permittees to prepare and submit an AKART analysis to Ecology by December 31, 2025 that evaluates treatment process improvements to increase TIN reduction as required to meet proposed future permit limits. However, since the City currently maintains an annual TIN average of < 10 mg/L and does not expect to see an increase in effluent TIN load during the permit cycle based on data submitted with their DMRs, submission of AKART analysis should not be required.

The City hired BHC Consultants, LLC (BHC) to assist the City with identifying, evaluating, and predicting impacts of potential alternatives for optimization of TIN removal and providing information on anticipated costs and suggestions for implementation of the selected alternative(s).

### *1.1 Overview of the Oak Harbor Clean Water Facility*

The City's Clean Water Facility (CWF) utilizes a membrane bioreactor (MBR) system preceded by aeration basins for removal of biochemical oxygen demand (BOD), total suspended solids (TSS), and total nitrogen (TN). Wastewater enters the CWF at the headworks by gravity and passes through a 3/8-inch coarse bar screen to remove larger debris before entering the influent pump station. Wastewater is pumped up to a vortex grit chamber to separate grit from the influent and then passes through 2 millimeter (mm) fine screens to remove smaller debris that passed through the coarse screen and grit chamber. From the headworks, screened wastewater flows by gravity to the aeration basins, which are equipped with mixers and diffusers designed to provide oxidation, nitrification, and denitrification to the incoming influent. The aeration basins consist of parallel treatment trains and a third future train that can currently be used for equalization storage. Each train consists of one deoxygenation zone to reduce dissolved oxygen (DO) entering the anoxic zones, two anoxic zones, and two aerobic zones. The two anoxic zones receive nitrified mixed liquor that is recycled from the MBR tanks in addition to the influent wastewater. The influent wastewater provides a carbon source for bacteria and the nitrate provides a source of oxygen in the absence of DO from aeration, as the anoxic zones are only mixed and not aerated. The nitrate is denitrified to nitrogen gas, which escapes to the atmosphere, thereby removing nitrogen from the wastewater. The

wastewater is then aerated in the aerobic zones where BOD is oxidized, and ammonia is nitrified to nitrate. The mixed liquor from the aeration basins enters the four MBR tanks. The MBR tanks are designed to separate the bacteria treating the wastewater to provide clear effluent or permeate. The MBRs also typically provide some additional oxidation and nitrification, as they receive high volumes of scour air to help keep the membranes clean. Each MBR tank contains three membrane cassettes containing hollow fiber membrane modules. Two cassettes in each tank contain 48 membrane modules, each with 340 square feet of membrane area, and the third cassette contains 36 modules. Filtered effluent then undergoes ultraviolet (UV) disinfection prior to effluent pumping to Oak Harbor via Outfall 003. The outfall is a 24-inch HDPE pipe with a 200-foot-long diffuser on the end that discharges the effluent to Oak Harbor. Return activated sludge (RAS) pumps return mixed liquor from the membrane tanks to the aeration basin to keep the biomass distributed and active throughout the process. Sodium hydroxide is added to the RAS prior to re-entering the aeration basins to boost alkalinity, some of which is consumed during nitrification, to maintain a near neutral pH. Waste activated sludge is periodically pumped to a storage tank to control the amount of biomass in the process. These sludge solids are pumped to a centrifuge dewatering unit and the dewatered cake is pumped to a dryer. The dried sludge meets the requirements for Class A biosolids and is conveyed into a roll-off container before being hauled away.

A process flow diagram is presented in Figure 1.

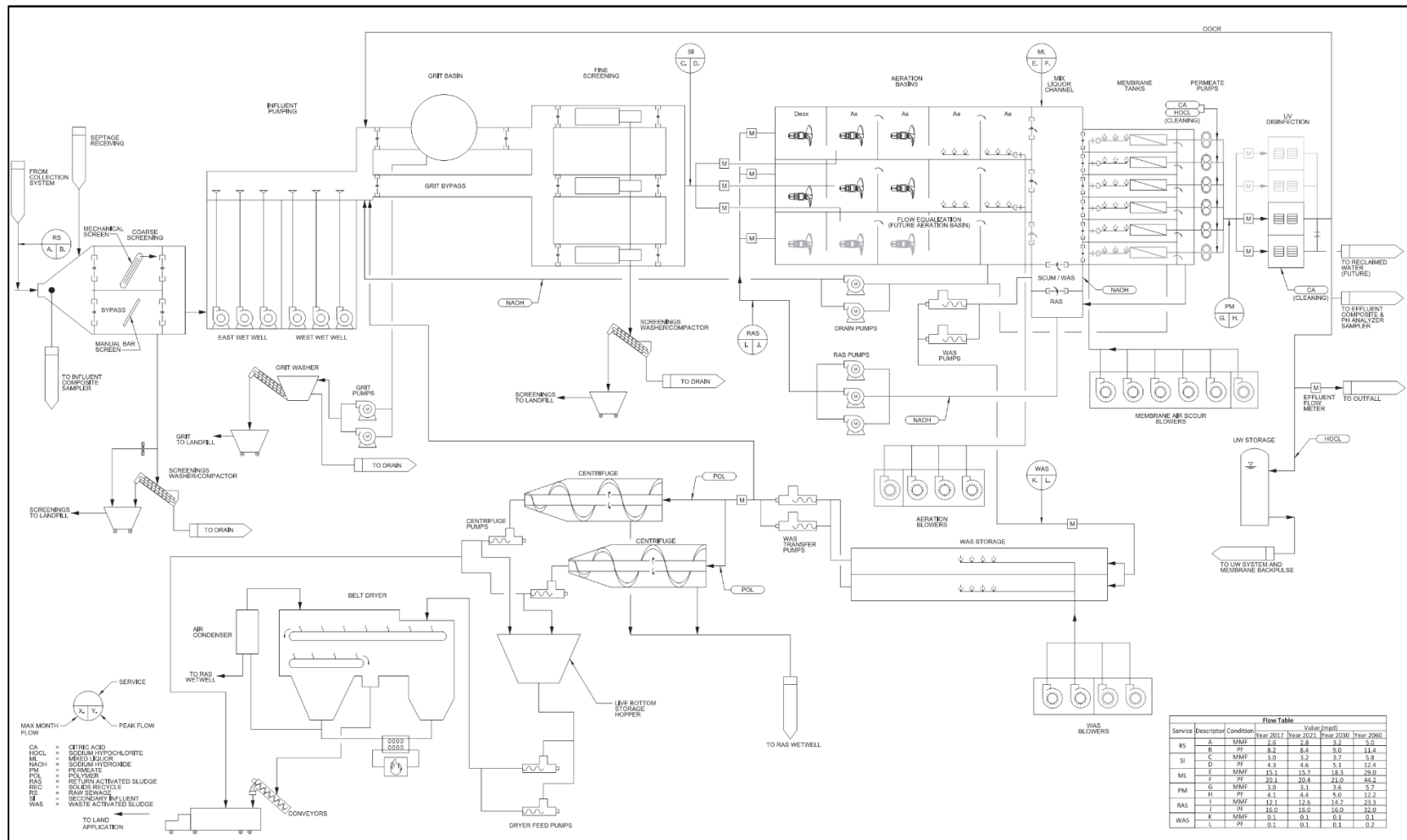


Figure 1 – WWTP Schematic

## 1.2 Current WWTP Performance

The current NPDES permit requires the WWTP to produce effluent with a monthly average BOD or TSS load of no more than 700 pounds per day (lbs./d) and pH between 6.0 and 9.0 when discharging to Oak Harbor. The NPDES permit does not currently include limits on nitrogen. The average effluent quality as it pertains to CBOD, TSS, nitrogen, and phosphorus for September 2022 through August 2023 is summarized in Table 1 below.

**Table 1**  
**WWTP Effluent**

<b>Effluent Parameter</b>	<b>September 2022 – August 2023 Average</b>
Flow, MGD	1.46
CBOD, mg/L	0.8
CBOD, lbs./d	9.7
TSS, mg/L	0.2
TSS, lbs./d	2.4
Ammonia-N, mg/L	0.1
Nitrate + Nitrite, mg/L	6.1
Total Kjeldahl Nitrogen (TKN), mg/L	1.3
TIN, mg/L	6.2
Total Nitrogen (TN), mg/L	7.4
Ortho-phosphorus, mg/L	1.6
Total Phosphorus, mg/L	1.7

As is evident from the data in Table 1, the WWTP is currently able to essentially complete nitrification and a relatively high level of nitrogen removal. This is helped by sufficient hydraulic residence time to grow a significant population of nitrifying organisms, sufficient dissolved oxygen to support the growth of nitrifying organisms, sufficient alkalinity to buffer against a drop in pH during nitrification that could cause inhibition of nitrification, sufficiently large anoxic zones for denitrification, and significant readily available carbon to drive denitrification in the anoxic zones. Based on an average influent TKN concentration of 45 mg/L, as discussed in Section 2.2 and shown in Table 3 below, the facility removes about 83.5% of total nitrogen on average.

## 2. Process Modeling

A process model of the existing WWTP was developed using the BioWin program developed by EnviroSim.

### *2.1 Process Model Setup*

Each anoxic zone and aerobic zone are added as a bioreactor element and modeled as separate continuous stirred tank reactors (CSTR) with assuming no settling of solids. The four MBR tanks were modeled as an MBR element as shown in Figure 2 below. Parameters related to operation and control of the MBR tanks and aeration basins like diffuser type, aeration rates or set points, and membrane characteristics are included in the model elements. The wasted sludge solids are directed to the waste activated sludge (WAS) storage tank, which supplies sludge to the centrifuge and dryer for processing. Biomass in the MBR tanks is returned to the deoxygenation zone in the aeration basin as RAS flow. Sodium Hydroxide addition for supplemental alkalinity is modeled as a state variable influent element into the RAS line. The BioWin process model diagram is shown in Figure 2 below.

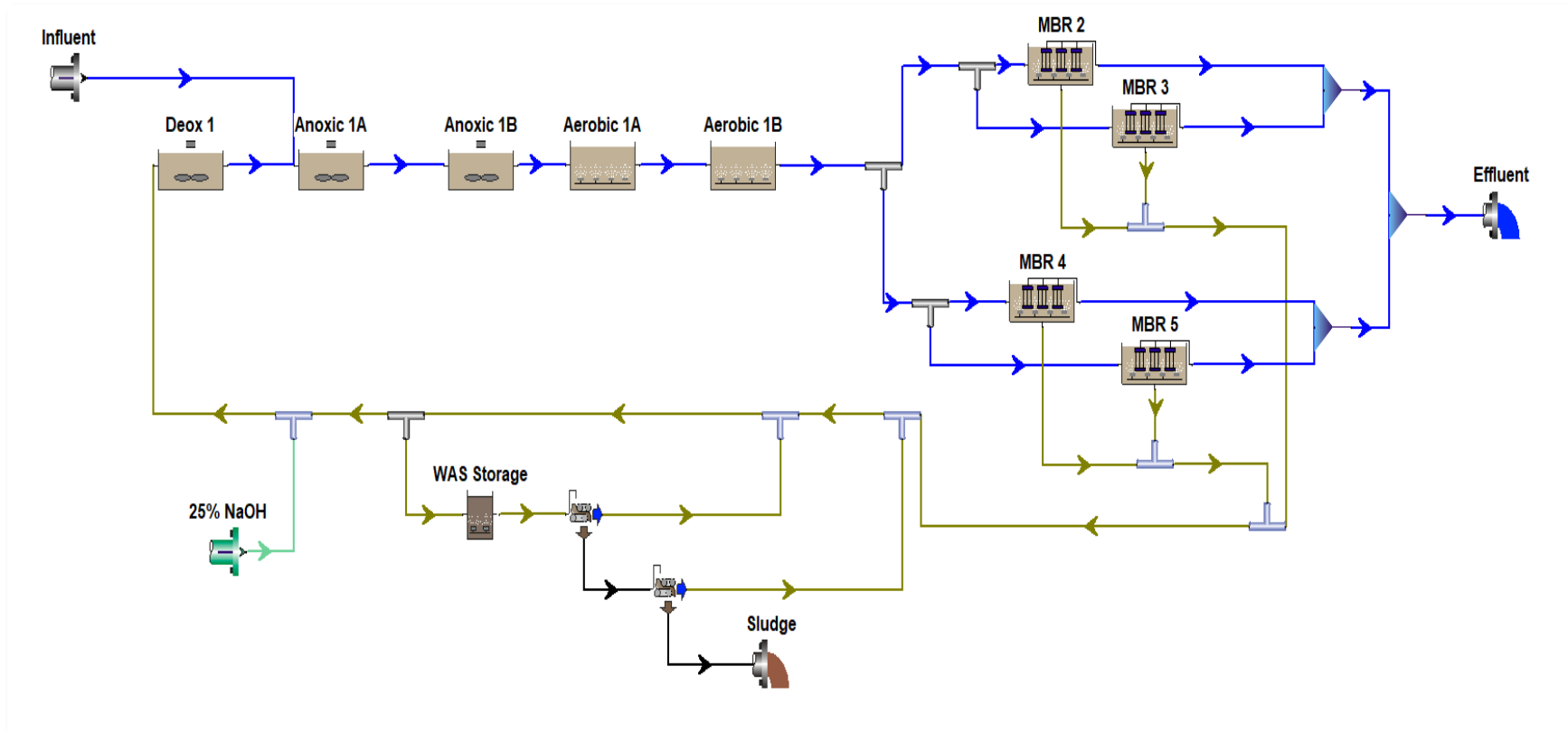


Figure 2 – BioWin Process Model Diagram

The available influent data from WWTP DMR reports and typical ratios for municipal wastewater were utilized as part of an initial effort to calibrate the model to accurately reflect current performance before evaluating the impacts of optimization alternatives. The calibrated model was then used to assess which of the alternatives identified (see Section 3) would be expected to have a positive and significant impact on process performance, particularly regarding increased removal of TIN. The influent data utilized for the model calibration and subsequent analyses was gathered from discharge monitoring reports for September 2022 through August 2023 (one full year of the most recent data at the time this effort was begun) and is summarized in Table 2 below.

**Table 2**  
**2022 – 2023 Average Influent Characteristics**

Influent Parameter	September 2022 – August 2023 Average
Flow, MGD	1.46
CBOD <sub>5</sub> , mg/L	289
TSS, mg/L	244
TKN, mg/L	41.5

## 2.2 Process Model Calibration

Because the process model requires more input data for influent wastewater characteristics than was available from the City, some assumptions needed to be made with respect to characteristics of typical municipal wastewater. Influent wastewater characteristics used in the process model are shown below in Table 3. Values for flow, TSS and 5-day carbonaceous BOD (CBOD<sub>5</sub>) were taken from historical data as shown in Table 2 above. Available influent TKN data was limited. Initially, the value from Table 2 was used. However, that average value is based on a fairly wide range. Given the limited data, it was reasonable to conclude that the actual average value could be higher so that the model predicted effluent quality more closely matched actual recorded effluent quality data. Therefore, the influent TKN value was increased from 41.5 to 45.0 milligrams per liter (mg/L). As there was no influent phosphorus data available, it was initially assumed that influent total phosphorus was 6.0 mg/L. This was then adjusted down to 5.0 mg/L so that the model predicted effluent quality more closely matched actual recorded effluent quality data. Values for filtered CBOD<sub>5</sub>, VSS, total chemical oxygen demand (COD), filtered COD, filtered-flocculated COD, acetate, effluent filtered COD, ammonia, and ortho-phosphorus were based on typical ratios to the other established parameters for municipal wastewater. Values for the remaining parameters were assumed based on what is typically seen at other wastewater facilities in the region.



**Table 3**  
**Process Model Influent Parameters**

Influent Parameter	Process Model Input Value
Flow, MGD	1.46
5-Day Carbonaceous BOD (CBOD <sub>5</sub> ), mg/L	289
Filtered CBOD <sub>5</sub> , mg/L	113
TSS, mg/L	244
Volatile Suspended Solids (VSS), mg/L	207
Total Chemical Oxygen Demand (COD), mg/L	548
Filtered COD, mg/L	206
Filtered-Flocculated COD, mg/L	103
Acetate, mg/L	12
Effluent Filtered COD, mg/L	30
TKN, mg/L	45.0
Ammonia-N, mg/L	34.2
Nitrate-N, mg/L	0.0
Total Phosphorus, mg/L	5.0
Ortho-Phosphorus, mg/L	2.5
pH	7.0
Alkalinity, mg/L (as CaCO <sub>3</sub> )	200
Dissolved Oxygen, mg/L	0.0
Calcium, mg/L	80.0
Magnesium, mg/L	15.0
Temperature, Celsius	16

Using the influent parameters as listed in Table 3 and operational parameters (e.g., recycle rates, aeration rates, solids retention time, etc.) as provided by the City, the process model predicted solids balance and effluent quality were compared with actual data to confirm a suitable calibration, typically defined as predicted values within 10% of actual values. As shown by the results summarized in Table 4, the model adequately predicted actual performance. The only significant outlier was the nitrate concentration in the mixed liquor. This is not a reported parameter but is used to track process performance. The City confirmed that the simple reagent test method used for monitoring nitrate in the mixed liquor is not very accurate by sending duplicates to two outside laboratories. Therefore, it is presumed the model predicted mixed liquor

nitrate value is sufficiently accurate and the discrepancy to the reported values using the reagent test method can be ignored.

**Table 4**  
**Process Model Calibration Overview**

Parameter	Actual 9/2022 to 8/2023 Value	Process Model Predicted Value	Difference
Effluent CBOD <sub>5</sub> , mg/L	0.8	1.0	0.2
Effluent TSS, mg/L	0.2	0.0	-0.2
Effluent pH	6.9	7.1	0.2
Effluent Ammonia-N, mg/L	0.1	0.1	0.0
Effluent TKN, mg/L	1.7	1.6	-0.1
Effluent Nitrate-N + Nitrite-N, mg/L	6.1	5.9	-0.2
Mixed Liquor Nitrate, mg/L	4.0	5.6	1.6
Effluent Total Phosphorus, mg/L	1.7	1.8	0.1
Effluent Ortho-Phosphorus, mg/L	1.6	1.8	0.2
Solids Retention Time, days	18.9	19	0.3%
Mixed Liquor Suspended Solids (MLSS), mg/L	7,960	7,687	-3.4%
RAS TSS, mg/L	9,699	9,600	-1.0%
Dried Solids, lbs./d	1,879	1,819	-3.2%
WAS Flow, gallons per day (gpd)	25,560	25,500	-0.2%

### 3. Identification & Screening of Nutrient Optimization Alternatives

Ten different alternatives were initially identified for consideration. These alternatives are listed below followed by a description, illustration, and initial assessment of each.

- Alternative 1 – Adjusting Dissolved Oxygen (DO) Set Points.
- Alternative 2 – Adjusting MBR RAS Rate.
- Alternative 3 – Adjusting Process SRT.
- Alternative 4 – Cycling Sludge Storage Aeration.
- Alternative 5 – Adjusting Dewatering/Drying Operating Schedule.
- Alternative 6 – Increasing Treatment Volume.

- Alternative 7 – Altering Number of MBR Tanks.
- Alternative 8 – Use of Available Flow Equalization.
- Alternative 9 – Supplemental Carbon.
- Alternative 10 – Supplemental Alkalinity.

Because supplemental alkalinity is already being added in the form of sodium hydroxide, the purpose of the last alternative was to examine whether or not the typical dose could be reduced to save money while still adequately buffering the pH.

### 3.1 *Alternative 1 – Adjusting Dissolved Oxygen (DO) Set Points*

Nitrification requires the presence of sufficient DO. Otherwise, the rate of nitrification can be limited when the DO concentration is too low to support a sufficiently high oxygen transfer rate. Lower levels of aeration and DO can result in heterotrophic bacteria that oxidize CBOD outcompeting slower growing nitrifying bacteria for the available oxygen. However, in some instances successful nitrification can be achieved at lower DO levels which can also promote some simultaneous denitrification in the aerobic zones. Figure 3 illustrates that adjustments to DO levels would be focused on the two aerobic basins.

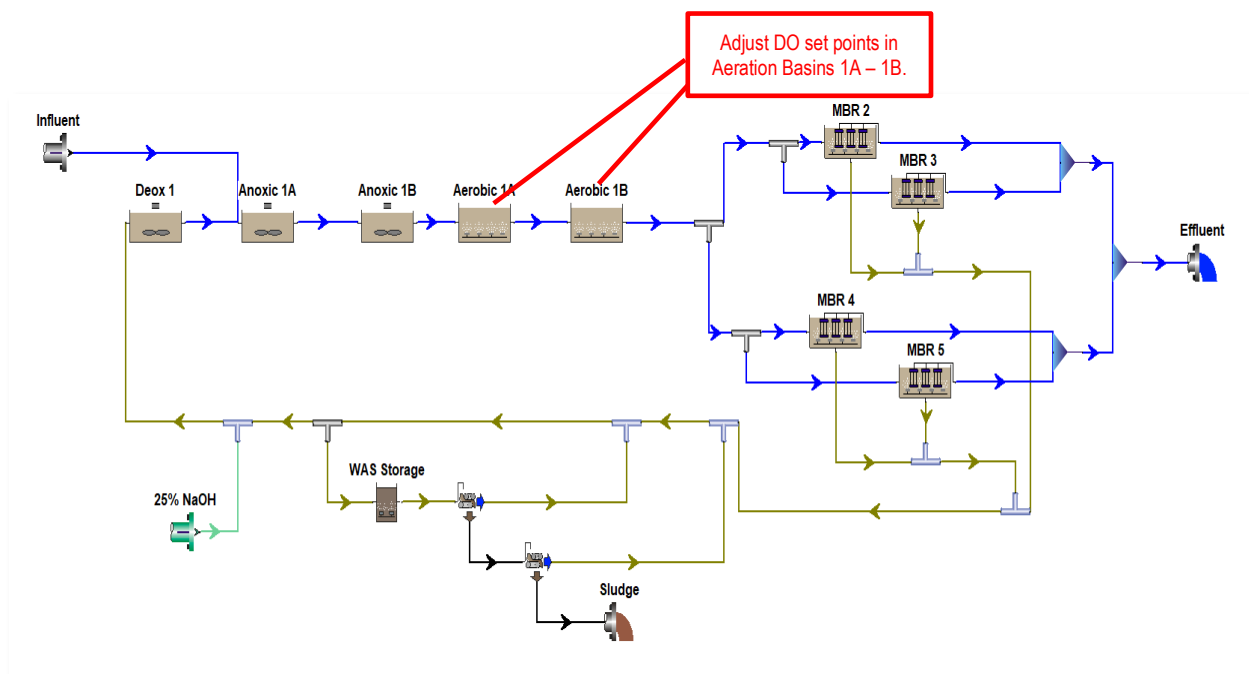


Figure 3 – Alternative 1 Illustration

As shown in Table 5, process modeling suggests that a lower DO concentration in the aerobic zones reduces effluent TN without impacting nitrification. The baseline scenario represents current aeration practice, which supplies about 950 standard cubic feet per minute (scfm) of air to each aerobic zone. This results in a DO of around 2 mg/L in Aerobic Zone 1A, which is fairly typical, but a significantly higher DO of over 4 mg/L in Aerobic Zone 1B. These DO concentrations support essentially complete nitrification as evident from the low ammonia concentration but yield less optimal denitrification. At lower DO set points, effluent TN does improve some, but appears to improve significantly at DO concentrations below 1 mg/L. This is likely due to achieving significant simultaneous denitrification in the aerobic zones at lower DO set points. This results in somewhat higher ammonia concentrations exiting the aeration basins (e.g., around 1.5 mg/L for the lowest DO set points in Scenario 17), but the high aeration in the MBR tanks nitrifies the remaining ammonia such that the effluent ammonia is still very low (predicted to be 0.2 mg/L or less). The minimum air requirement to keep the biomass suspended is 200 scfm per aerobic zone (400 scfm total), which corresponds to a DO level of about 0.1 mg/L in the aerobic zones. That level of aeration is insufficient to support adequate nitrification, such that any of the scenarios shown in Table 5 would have adequate aeration to keep solids in suspension. As shown in Table 5, the resulting decrease in power demand with lower DO set points can yield significant energy savings totaling more than \$20,000 per year, based on a unit cost of \$0.0865 per kWh from a recent electric bill provided by the City.

Table 5  
Comparison of DO Set Points

Scenario	Zone 1A DO Setpoint (mg/L)	Zone 1B DO Setpoint (mg/L)	Zone 1A Airflow (scfm)	Zone 1B Airflow (scfm)	Total Airflow (scfm)	Effluent Ammonia-N (mg/L)	Effluent TN (mg/L)	Power Decrease (kW)	Cost Savings \$/yr.
Baseline	2.24	4.41	950	950	1900	0.07	7.47	0	\$ 0
1	0.25	0.25	594	445	1039	0.21	3.80	31.5	\$ 23,876
2	0.35	0.35	630	461	1091	0.14	4.29	29.6	\$ 22,437
3	0.4	0.4	647	469	1116	0.12	4.61	28.7	\$ 21,770
4	0.5	0.5	675	480	1155	0.1	5.11	27.2	\$ 20,633
5	1	1	769	515	1284	0.08	6.36	22.6	\$ 17,102
6	1.5	1.5	845	545	1390	0.08	6.88	18.7	\$ 14,170
7	1.8	1.8	892	565	1457	0.07	7.06	16.2	\$ 12,298
8	2.5	2.5	1013	618	1631	0.07	7.33	9.8	\$ 7,449
9	2.5	1	1045	443	1488	0.07	6.86	15.1	\$ 11,449
10	2	1	950	460	1410	0.07	6.77	17.9	\$ 13,594
11	1.5	1	860	483	1343	0.07	6.63	20.4	\$ 15,458
12	1.5	0.75	875	446	1321	0.08	6.41	21.2	\$ 16,057
13	1.5	0.5	900	398	1298	0.08	6.03	22.0	\$ 16,678
14	1.08	2.82	750	750	1500	0.08	6.93	14.6	\$ 11,093
15	0.57	1.59	650	650	1300	0.09	6.13	22.0	\$ 16,640
16	0.44	0.53	650	500	1150	0.11	5.04	27.5	\$ 20,800
17	0.21	0.32	550	500	1050	0.20	3.96	31.1	\$ 23,573

### 3.2 Alternative 2 – Adjusting MBR RAS Rate

Adjusting the RAS rate from the MBR tanks would introduce more nitrate to the anoxic zone that could boost denitrification. However, higher recycling rates will also introduce more dissolved oxygen into the anoxic zone, thus reducing the potential for denitrification. Therefore, an optimal recycling rate is to be selected by balancing both criteria. As illustrated in Figure 4, adjustments to the total RAS rate would be evenly split between each MBR tank.

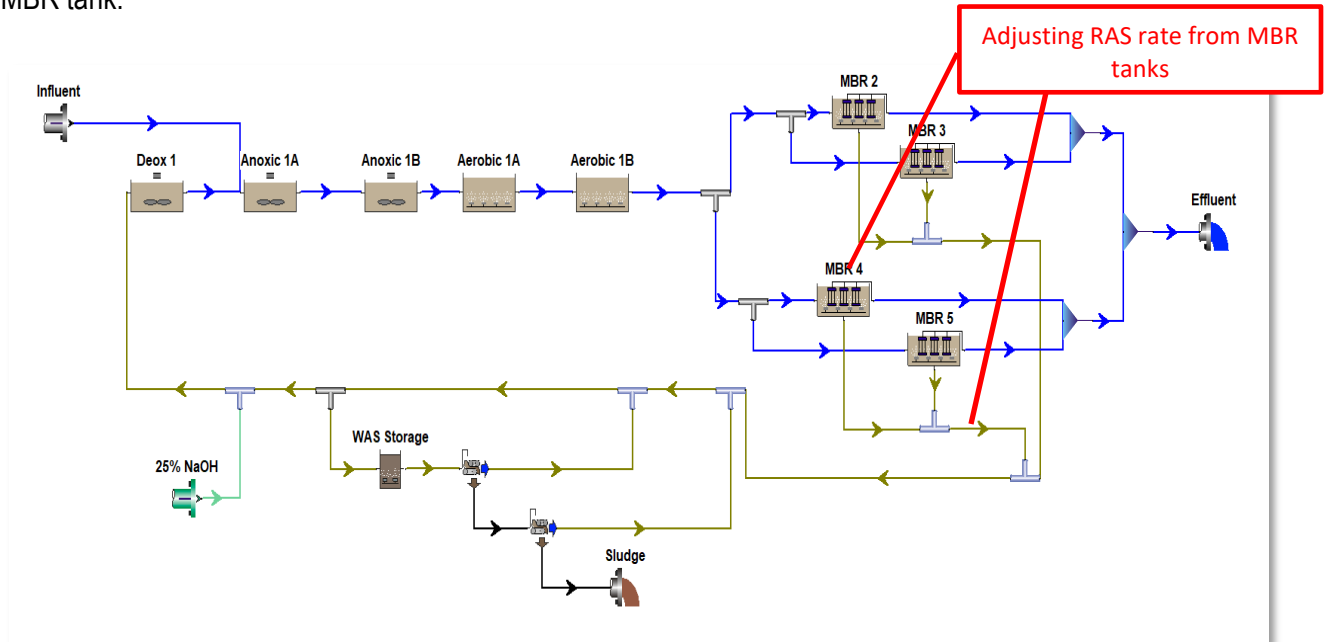


Figure 4 – Alternative 1 Illustration

The modeling results suggest that increasing the MBR RAS rate may increase nitrogen removal. A RAS rate of about 5Q (five times the influent flow) appears to be optimal compared to the current rate of 4Q. Increasing the RAS rate to 5Q appears to yield a significant reduction in effluent nitrogen with no negative impacts on nitrification or removal of CBOD<sub>5</sub>. However, the City reported that the existing RAS pumps are unable to maintain the current target of 4Q and need to be upsized. A summary of the process modeling results for different RAS rates is provided in Table 6 below.

Table 6  
Comparison of MBR RAS Rates

Scenarios	RAS Flowrate	Influent Flow (MGD)	Effluent NH <sub>3</sub> (mg/L)	Effluent TN (mg/L)	Reduction in TN (%)	Effluent CBOD <sub>5</sub> (mg/L)
Baseline	4Q	1.46	0.07	7.47		0.99
1	3Q	1.46	0.07	8.8	-17.8	1.00
2	5Q	1.46	0.08	6.83	8.6	0.98
3	6Q	1.46	0.08	6.96	6.8	0.97

### 3.3 Alternative 3 – Adjusting Process SRT

Solids retention time (SRT) plays a vital role in WWTP nutrient optimization. The SRT is the total weight of biomass in the aeration basins and MBR tanks (see Figure 5) divided by the rate of mass removed from the system via the effluent and sludge wasting. The microorganisms responsible for nitrification have a much slower growth rate than other heterotrophic bacteria. If SRT is too low, incomplete nitrification occurs and elevated levels of ammonia will be observed in the effluent. SRT that is too high can lead to release of nitrogen from bacteria due to increased endogenous decay and yield high MLSS in the MBR tanks that can increase membrane fouling. The current SRT for the system is about 19 days. BioWin can simulate different wasting rates to predict the impact of lower or higher SRT values.

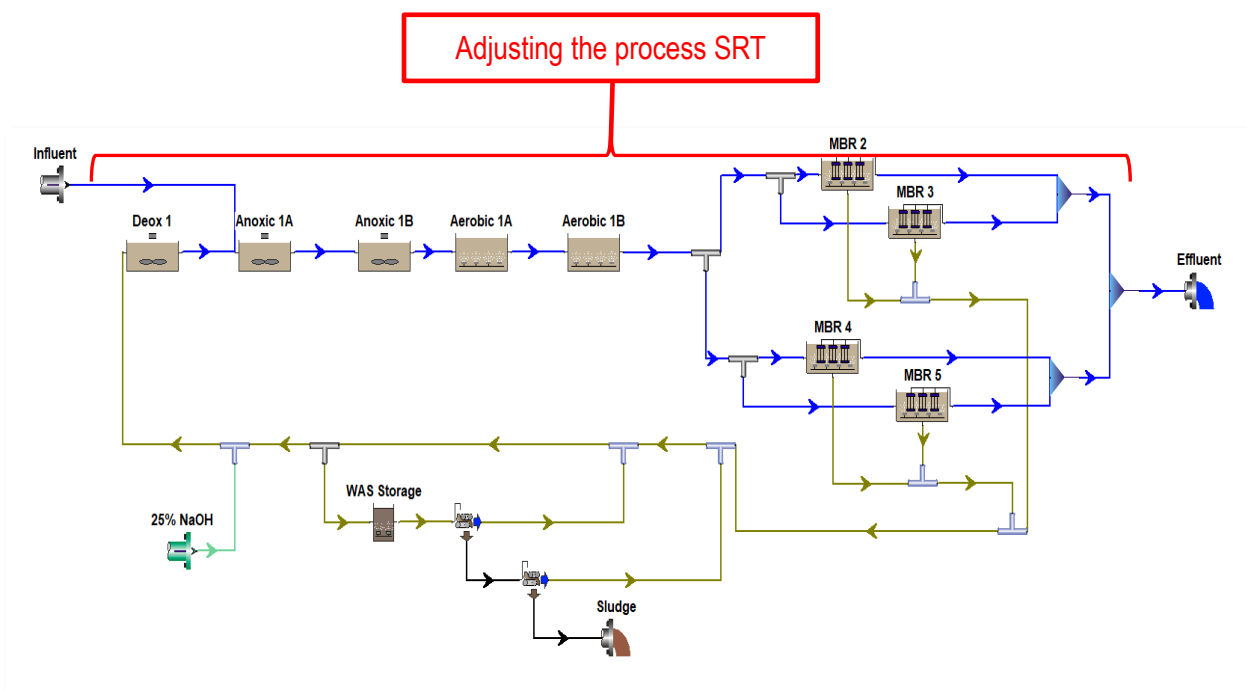


Figure 5 – Alternative 3 Illustration

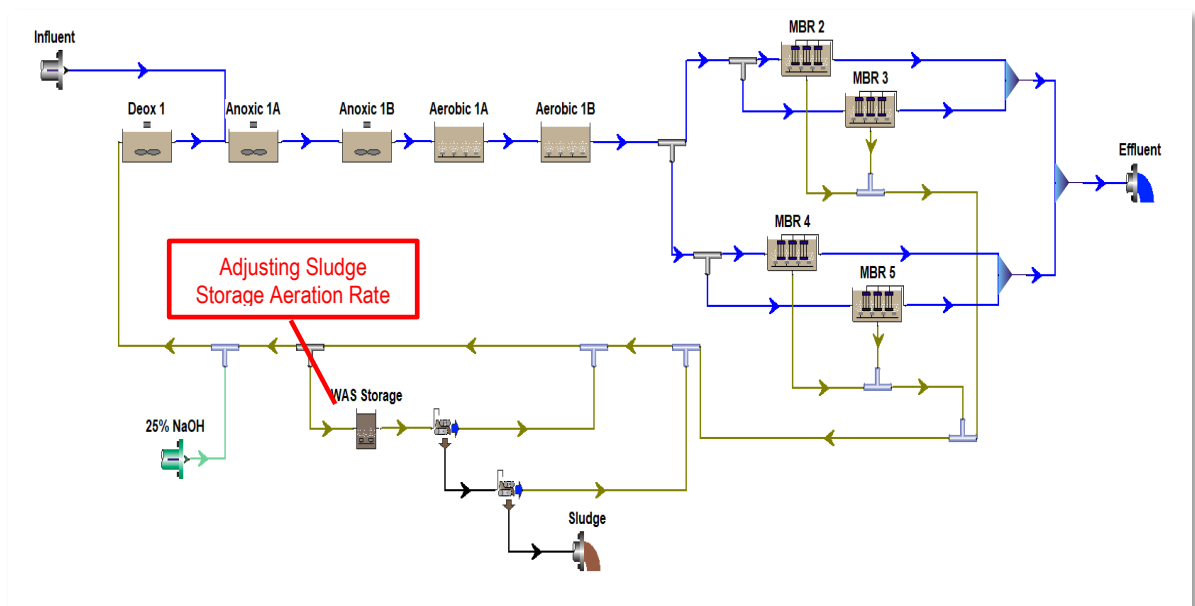
A comparison of different SRT values ranging from 15-25 days from initial process modeling, is summarized in Table 7 below. A range of 15-25 days was selected to avoid too low or too high values that would maintain an MLSS concentration in the MBR tanks of 8,000 to 12,000 mg/L. Based on the results shown in Table 7, it appears that there is no significant reduction in total nitrogen by changing SRT within a reasonable range.

**Table 7**  
**Comparison of Solids Retention Times**

Scenarios	SRT (Days)	Influent Flow (MGD)	Effluent NH <sub>3</sub> (mg/L)	Effluent TN (mg/L)	Effluent CBOD <sup>5</sup> (mg/L)
Baseline	19	1.46	0.07	7.47	0.99
1	15	1.46	0.08	7.44	1.01
2	17	1.46	0.08	7.45	1.00
3	22	1.46	0.07	7.48	0.99
4	25	1.46	0.07	7.49	0.98

### 3.4 Alternative 4 – Cycling Sludge Storage Aeration

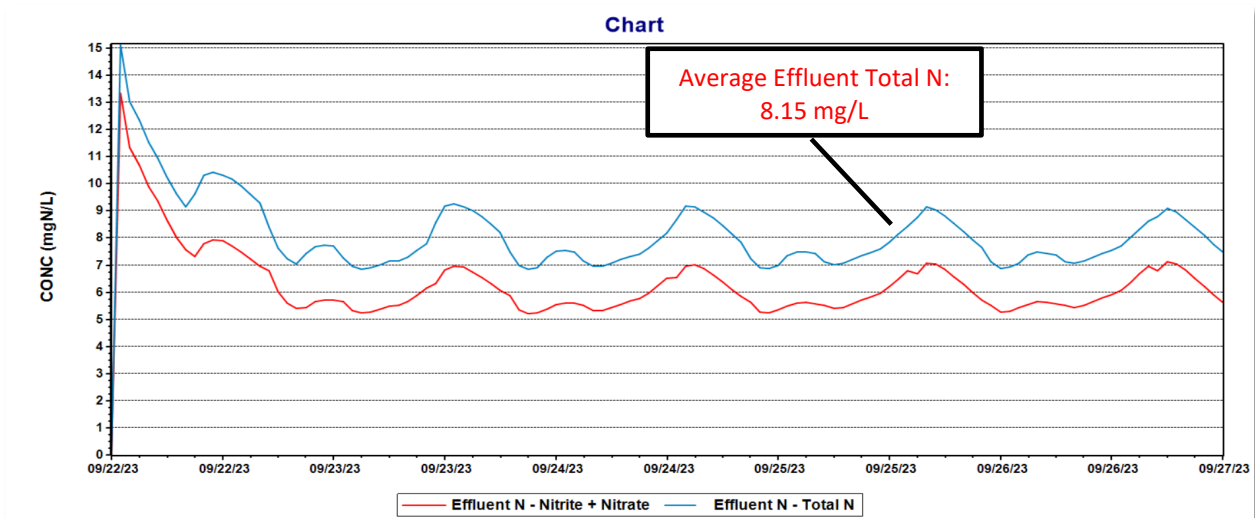
As shown in Figure 6, Alternative 4 considers the impact of cycled aeration in the WAS storage tank. Cycled aeration could allow for greater denitrification by providing intermittent periods of anoxic conditions when air is cycled off. The duration that aeration was off could not be too long resulting in substantial solids settlement that would reduce the effective volume and contact between the bacteria and nitrate.



**Figure 6 – Alternative 4 Illustration**



The process model was used to assess the impact of cycled aeration using an aeration pattern with a DO set of 2 mg/L for one (1) hour followed by 0 mg/L of DO for one (1) hour, with this pattern repeated. Similarly, cycled aeration was also modeled for aeration patterns of two (2) hours and four (4) hours respectively. To appropriately model these cycled aeration patterns a dynamic simulation was ran with an assumed diurnal influent flow pattern based on a typical diurnal curve. For simplicity, it was assumed that influent concentrations remained the same even as the flow increased and decreased. Generally, DO would deplete over a period of time when aeration was off, causing the bacteria to then utilize nitrate as an oxygen source, and then increase over time when full aeration returned. Figure 7 represents the effluent TN levels with the baseline continuous aeration of 450 scfm with a diurnal influent flow pattern. Figure 8, Figure 9, and Figure 10 represent the effluent TN levels with cycled aeration patterns of one (1) hour, two (2) hours, and four (4) hours, respectively.



**Figure 7 – Baseline with Diurnal Influent Flow Pattern**

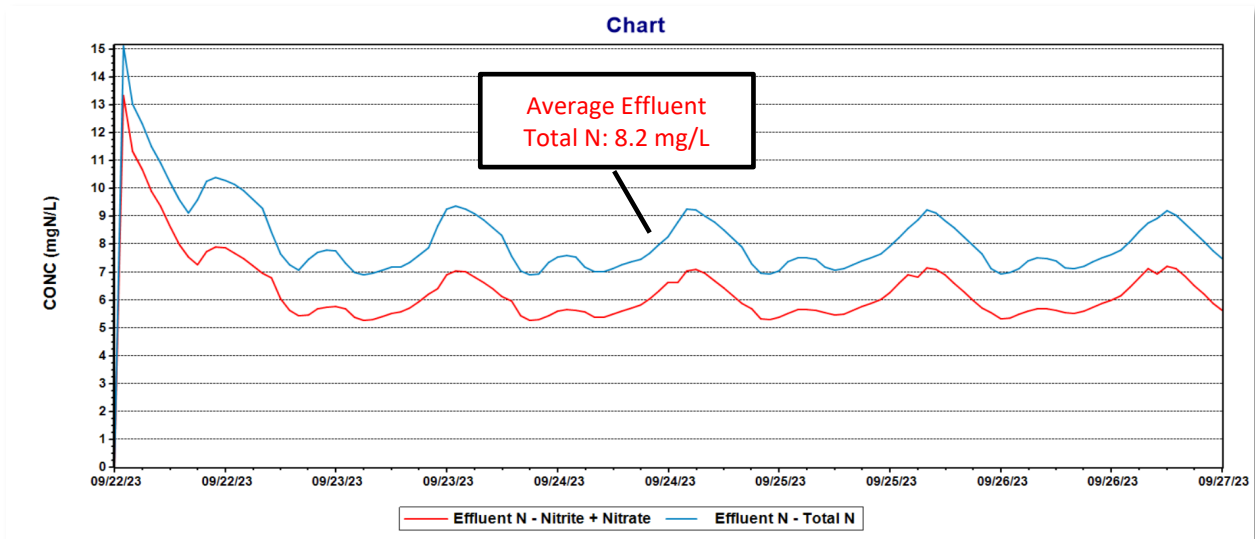


Figure 8 – Hour Cycle Time with Diurnal Influent Flow Pattern

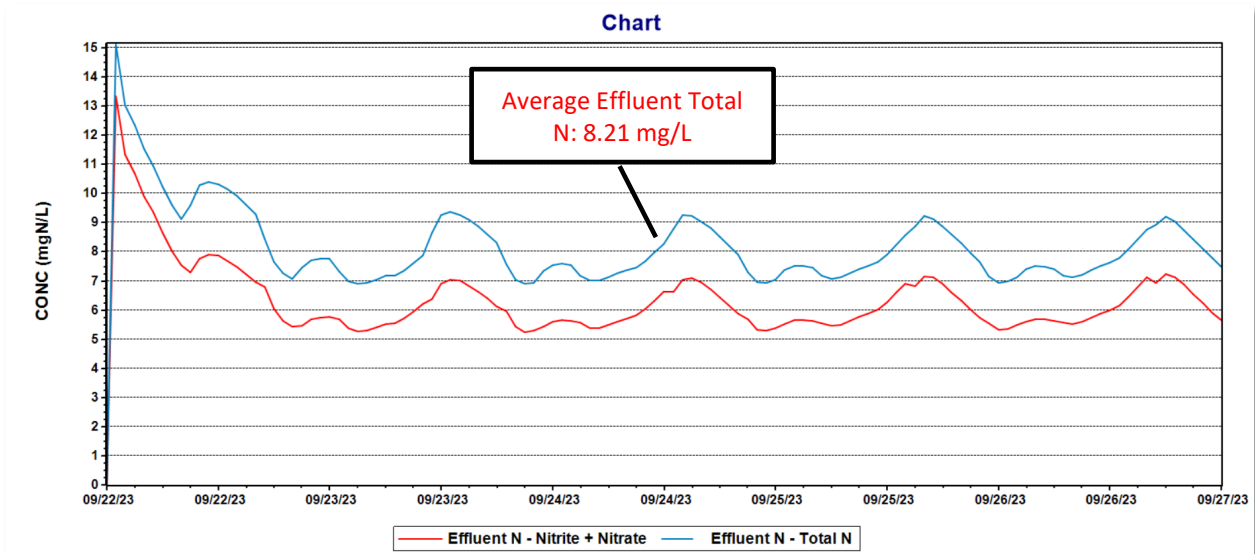


Figure 9 – Hour Cycle Time with Diurnal Influent Flow Pattern  
2-Hour Cycle Time with Diurnal Influent Flow Pattern

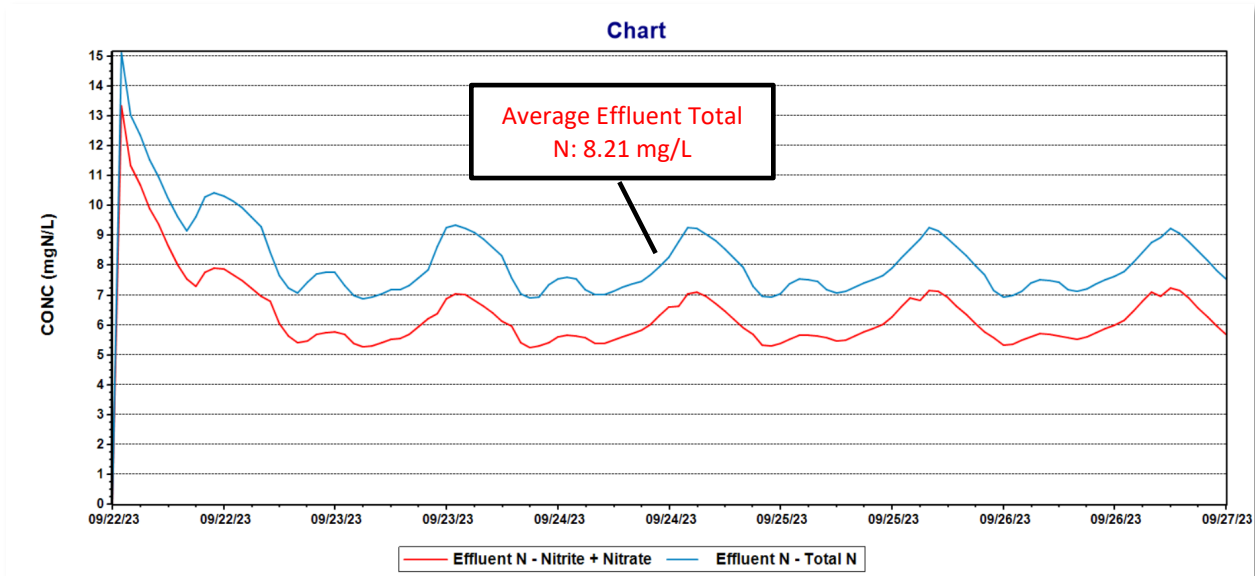


Figure 10 – 4-Hour Cycle Time with Diurnal Influent Flow Pattern

These results suggest that cycled aeration in the WAS storage tank would not have a noticeable impact on nitrogen removal. Under the baseline condition, the DO in the WAS storage tank is already very low, such that there is simultaneous nitrification and denitrification occurring. As a result, there is no is not significant additional denitrification occurring when the air is cycled off.

### 3.5 Alternative 5 – Adjusting Dewatering/Drying Operating Schedule

Alternative 5 considers adjusting operation of the dewatering and sludge drying systems (see Figure 11). Potentially, changing from less intermittent to more continuous operation could improve the efficiency of nitrogen removal by shifting from higher concentration recycle streams at shorter intervals to lower concentration recycle streams at longer intervals. The WAS storage tank was changed to a variable volume reactor in the process model to allow the same wasting rate and aeration of the WAS storage tank remained at 450 scfm.

To gauge the impact of adjusting the dewatering and drying systems operating schedule, dynamic simulations using a diurnal flow pattern were run for continuous operation and operation for only eight (8) hours per day.

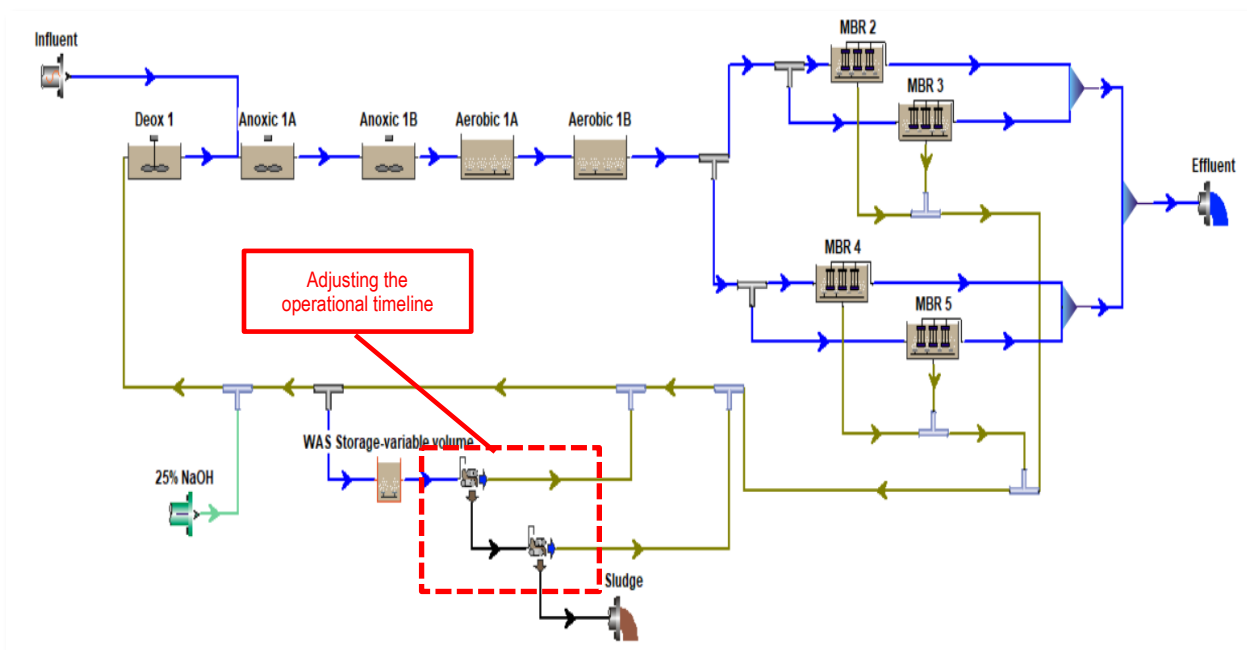


Figure 11 – Alternative 5 Illustration

Figure 12 illustrates the effluent TN values for continuous operation of solids handling processes and Figure 13 represents effluent TN levels for operation of the solids handling processes limited to eight (8) hours per day. No noticeable variation was observed in the effluent quality due to resulting changes in the return streams. Also, there was no impact to the effluent quality if the eight (8) hour period operation was at different times of the day. Therefore, adjusting the operating schedule of the solids handling process appears to have no significant impact on effluent quality under current conditions.

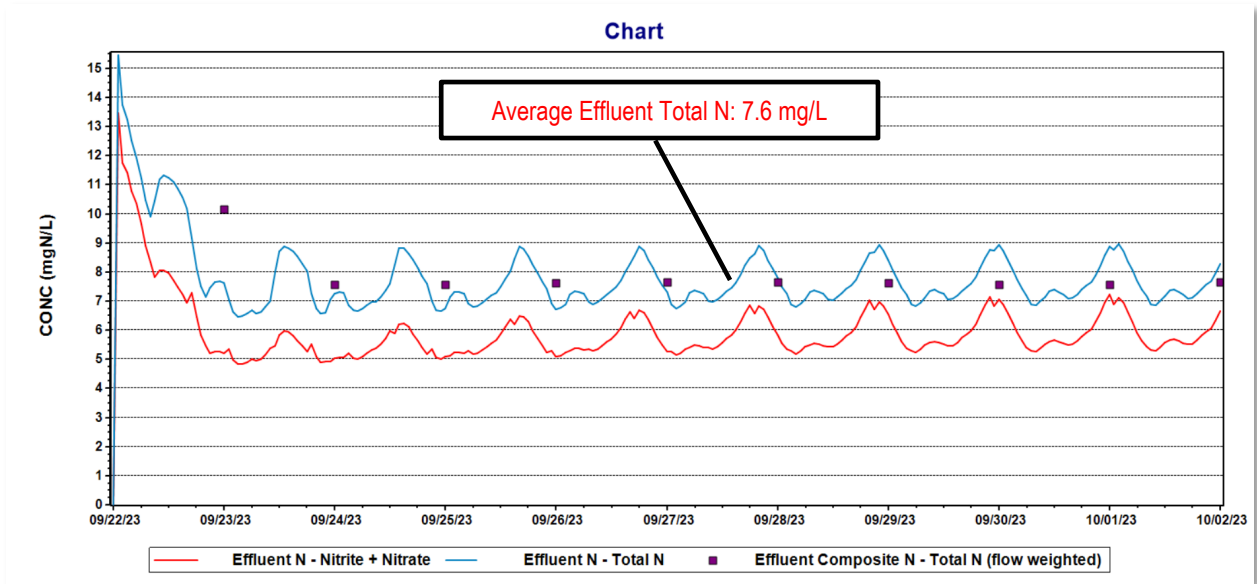


Figure 12 – Continuous Operation of Solids Handling

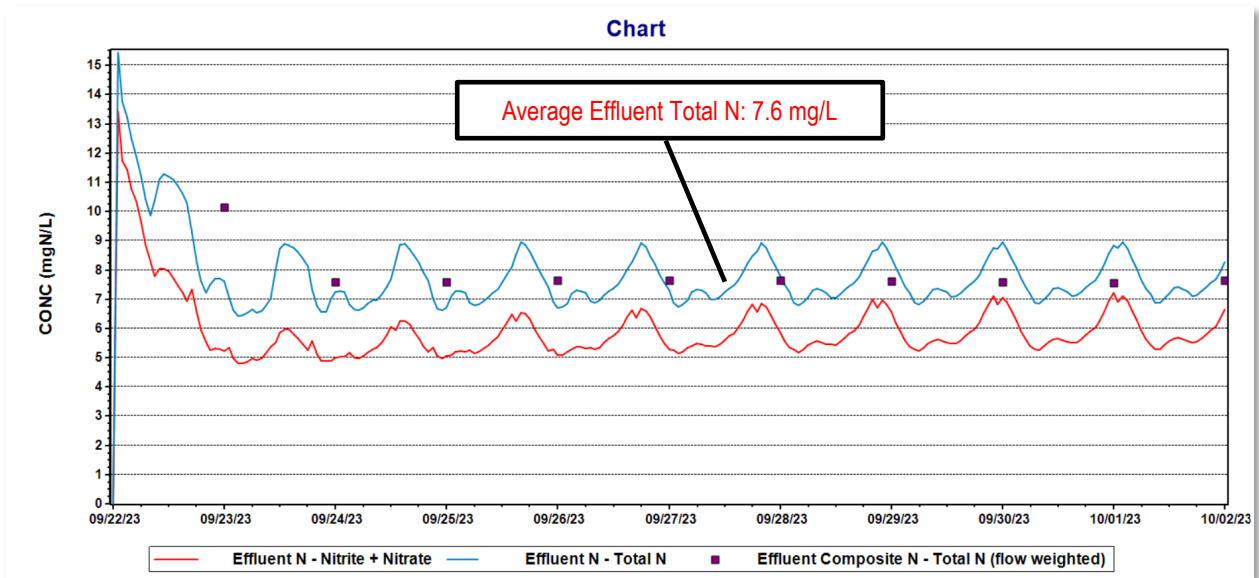


Figure 13 – Operating Solids Handling 8 hours/day

### 3.6 Alternative 6 – Increasing Treatment Volume

There is potential that increasing the treatment volume could increase nitrogen removal by providing additional aerobic volume for nitrification and additional anoxic volume for denitrification. As illustrated in Figure 14, the total aeration volume would be adjusted by utilizing a second or third aeration basin. For two aeration basins in service, the treatment volume would be twice the current baseline volume with just one aeration basin in service. Similarly, for three aeration basins in service, the treatment volume would be three times the current baseline volume.

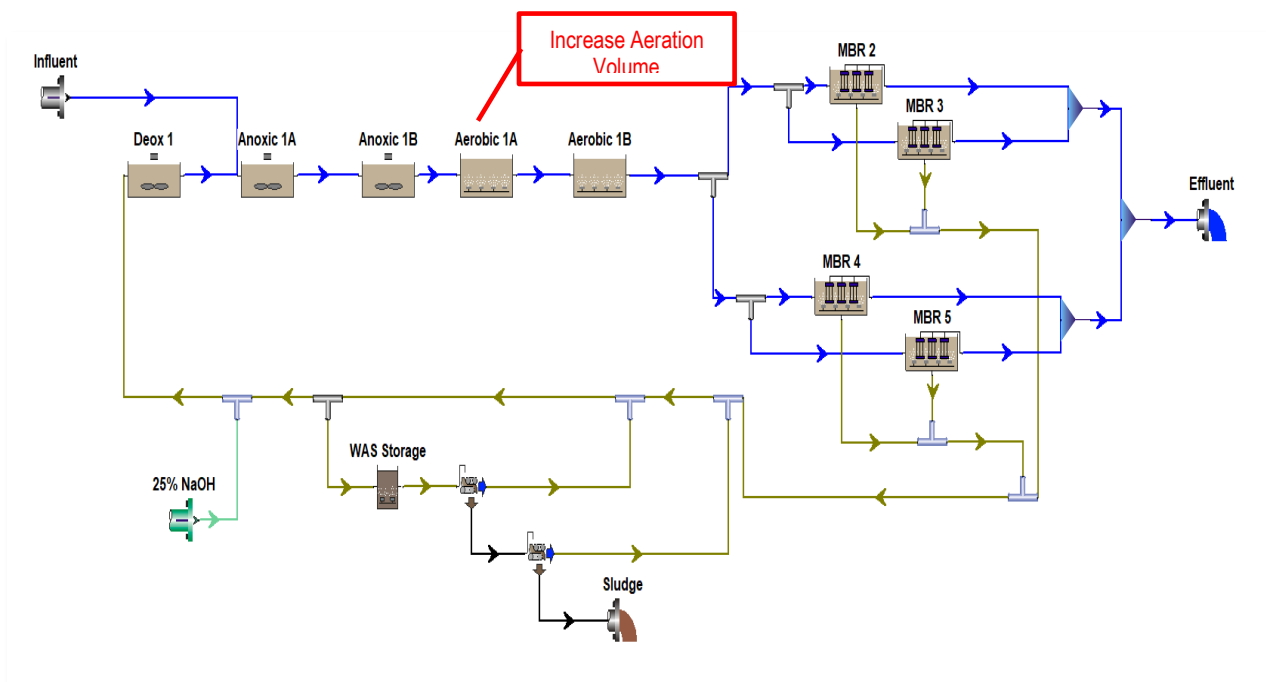


Figure 14 – Alternative 6 Illustration

Process modeling results suggest that increasing the treatment volume will not significantly improve effluent quality and would result in substantially higher operation and maintenance costs. A summary of the process modeling results for different treatment volumes is provided in Table 8 below. Although there is a slight reduction in effluent TN values and higher volumes, the results are not significant enough to warrant the additional labor and expense of operating a second or third aeration basin. Because the WWTP as a whole is operating well below its rated capacity, there is currently little benefit to be gained from operating with higher treatment volumes.

Table 8  
Comparison with Aeration Volume

Scenarios	Number of Aeration Basins in Service	Influent Flow (MGD)	Effluent NH <sub>3</sub> (mg/L)	Effluent TN (mg/L)	Effluent CBOD <sub>5</sub> (mg/L)
Baseline	1	1.46	0.07	7.47	0.99
1	2	1.46	0.07	7.30	0.93
2	3	1.46	0.07	7.22	0.89

### 3.7 Alternative 7 – Altering Number of MBR Tanks

Changing the Number of MBR tanks in service could potentially yield an improvement in nitrogen removal. Fewer MBR tanks could reduce the amount of DO recycled to the anoxic zones, thereby improving their efficiency. Additionally, fewer MBR tanks could reduce endogenous decay and the associated release of nutrients. Currently there are four (4) MBR tanks in service that have an average daily flowrate capacity of 2.8 MGD total, which is well above the current average flow rate of 1.46 MGD. As illustrated in Figure 15, the number of MBR tanks in service is simulated in the process model by changing the splitter flow percentages to each of the MBR tanks and the RAS rate from each MBR tank. The total RAS rate remained set at 4Q, such that with only two MBR tanks in service the RAS rate from each MBR tank in service was doubled.

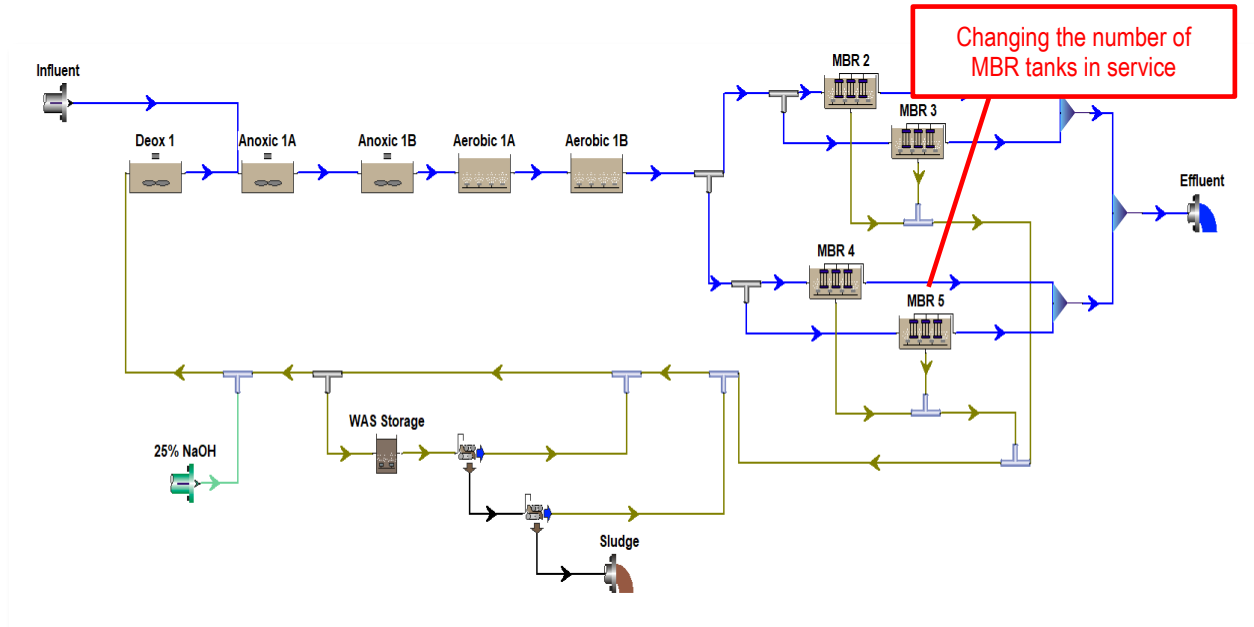


Figure 15 – Alternative 7 Illustration

Process modeling results indicated that decreasing the number of MBR tanks in service could yield a small improvement in nitrogen removal without impacting nitrification or removal of CBOD<sub>5</sub>. This small improvement is likely due to less DO being returned from the MBR tanks to the anoxic zones, resulting in more effective

denitrification, and possibly less endogenous decay. However, the slight potential increase does not warrant increasing membrane flux rates, which could yield more fouling and more frequent membrane clean-in-place procedures. The City does seasonally take a MBR tank offline at times for maintenance, though this is for a relatively short duration, and the process will automatically idle a tank when not needed, though there is still some delivery of scour air to an idle tank. A summary of the process modeling results for different numbers of MBR tanks in service is provided in Table 9 below.

**Table 9**  
**Comparison with Number of MBR Tanks in Service**

Scenarios	Number of MBR Tanks in Service	Influent Flow (MGD)	Effluent NH <sub>3</sub> (mg/L)	Effluent TN (mg/L)	Effluent CBOD <sub>5</sub> (mg/L)
Baseline	4	1.46	0.07	7.47	0.99
1	3	1.46	0.07	7.32	1.02
2	2	1.46	0.08	7.14	1.04

### *3.8 Alternative 8 – Use of Available Flow Equalization*

Flow Equalization can be utilized to minimize variations in wastewater flow and composition, thereby allowing the process to operate closer to steady state conditions. In many cases, this can yield improved performance by avoiding periods of time when higher flows and loads would otherwise decrease process efficiency. Currently, the WWTP has a third aeration basin which can be used as an equalization (EQ) tank. An EQ tank with volume and dimensions equal to the aeration basin was added into the model before the influent is introduced into the anoxic zones as illustrated in Figure 16. As discussed prior, a typical diurnal pattern was input for the influent to allow for running a dynamic simulation to assess the impacts of equalization.



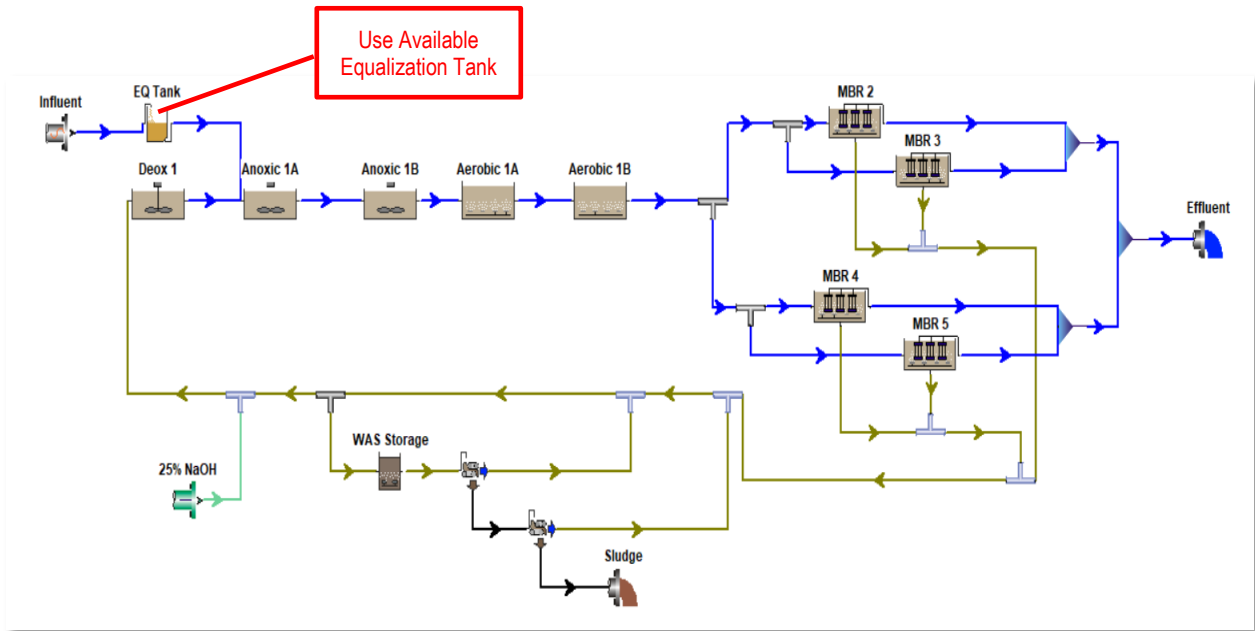


Figure 16 – Alternative 8 Illustration

Process modeling indicates that there is no effluent quality benefit to utilizing the available tankage to equalize influent flow. As shown in Figure 17, the flow weighted average effluent TN for operating without an EQ tank is predicted to be 7.9 mg/L. As shown in Figure 18, the process model predicts a flow weighted average effluent TN of 8.56 mg/L when operating with the available EQ volume. It appears that effluent total nitrogen actually may increase with use of the EQ volume under current conditions. This may be because the process is currently underloaded, such that it can achieve better performance when loading is higher, rather than performance degrading at higher loadings). Additionally, the available EQ volume is not quite large enough to fully equalize current average flows, such that peak loads would still pass into the process for periods of time.

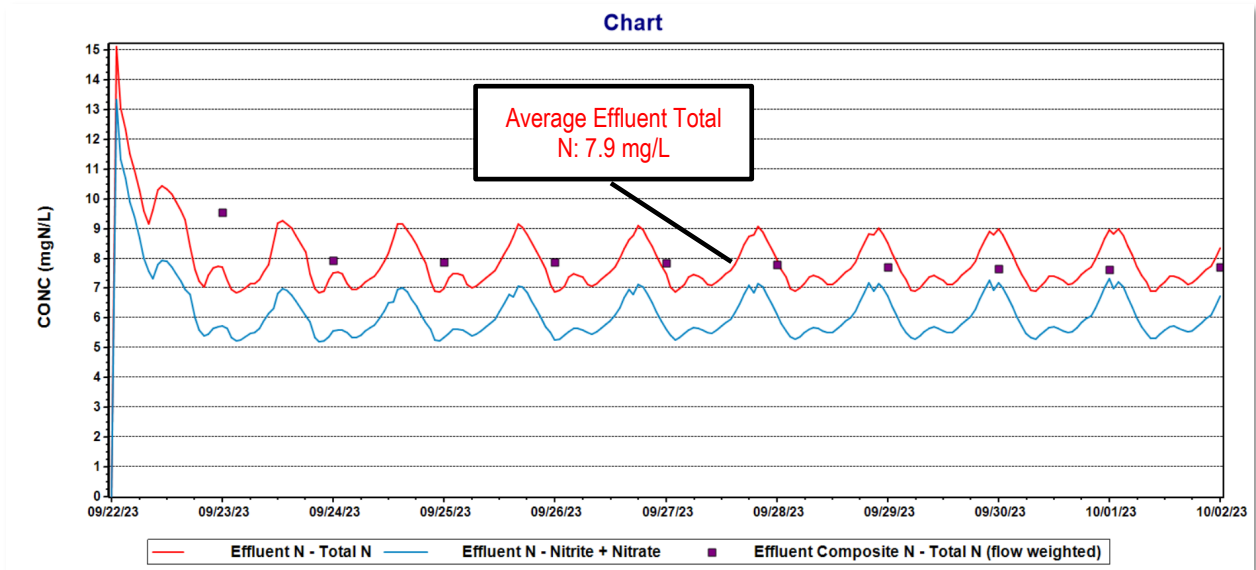


Figure 17 – Operating with No EQ Tank

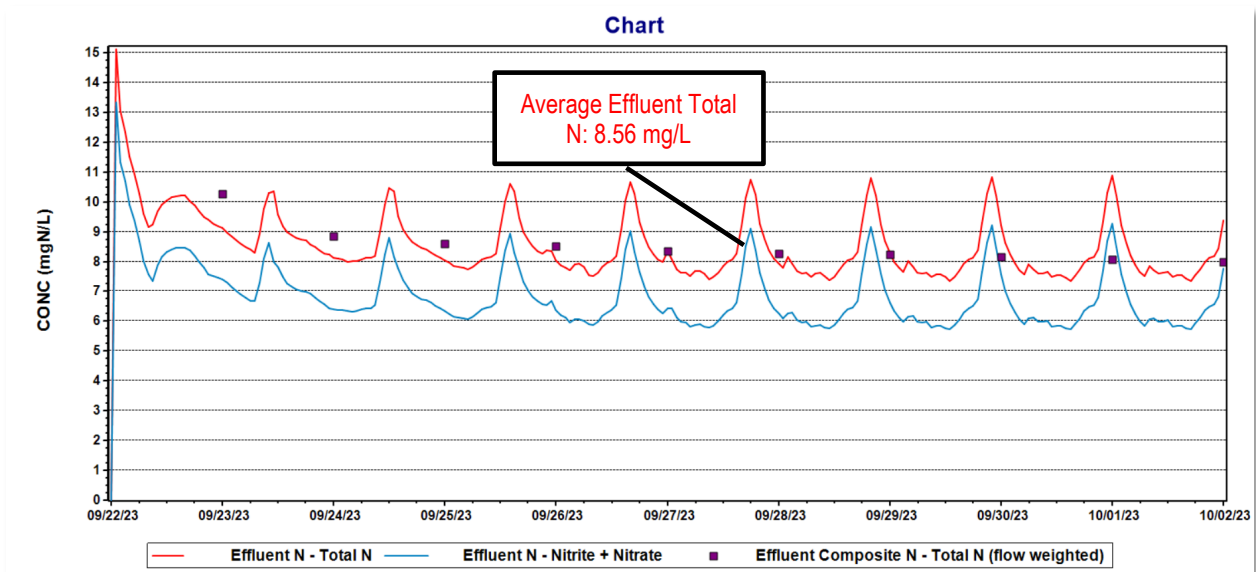


Figure 18 – Operating with EQ Tank

### 3.9 Alternative 9 – Supplemental Carbon

It is possible that there is insufficient readily available carbon in the influent wastewater to maximize denitrification in the anoxic zones. In such instances, the addition of a synthetic carbon source can be used to enhance denitrification. This would require purchase of synthetic carbon source (e.g., Micro-C) and a metering pump to pace dosing of the synthetic carbon with influent flow. Because the synthetic carbon source has a very

high concentration, a single tote could last a week or so at lower doses, such that the delivery totes themselves could be used for storage and the synthetic carbon metered directly from the totes. However, at higher doses, a bulk storage container would likely be necessary to store sufficient supplemental carbon. As illustrated in Figure 19, synthetic carbon could be dosed into the influent prior to flow entering the anoxic zones.

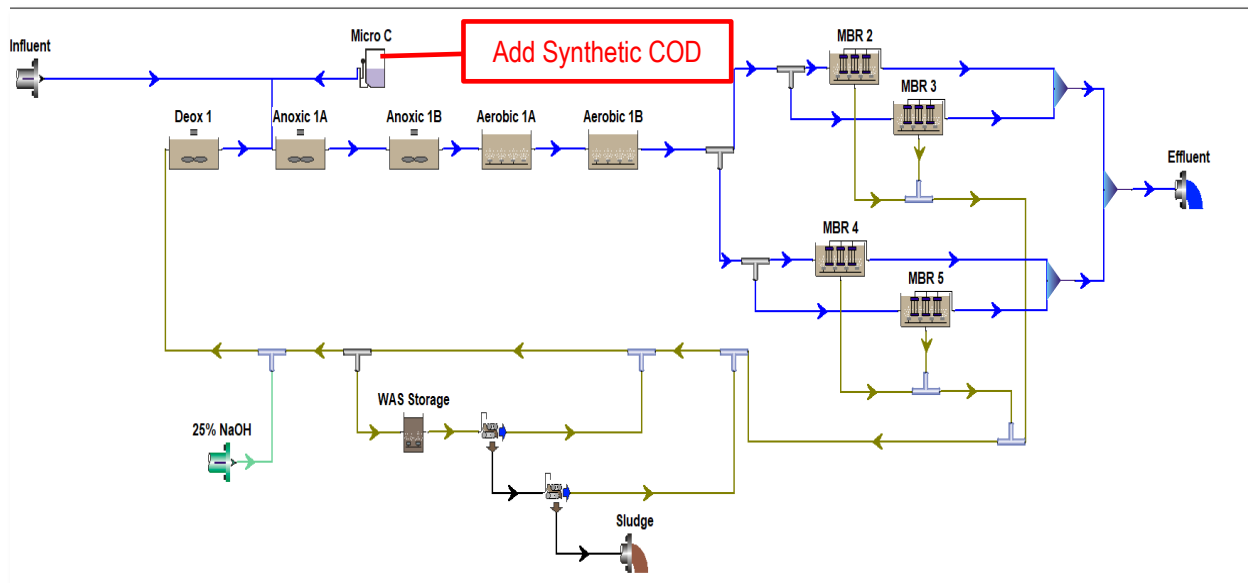


Figure 19 – Alternative 9 Illustration

As shown in Table 10 below, process modeling suggests that even a dose as low as 50 gallons per day of synthetic supplemental carbon could yield a significant reduction in effluent TN. A dose of up to 250 mg/L of supplemental carbon could increase nitrogen removal by approximately 35 percent. Doses above 250 gpd do not appear to provide further benefit. Higher doses due yield a slightly higher effluent ammonia and effluent COD, but neither increase is of concern. The higher effluent ammonia is likely due to the higher supplemental carbon doses consuming more oxygen, as not all of the supplemental carbon will be consumed in the anoxic zone. This is also why the effluent COD shows a slight increase, as some of the supplemental carbon will also pass through the aerobic zones.

Table 10  
Supplemental Carbon

Scenarios	Micro C Dose (Gal/Day)	Influent Flow (MGD)	Effluent NH <sub>3</sub> (mg/L)	Effluent TN (mg/L)	Effluent COD (mg/L)	Effluent CBOD (mg/L)
Baseline	0	1.46	0.07	7.47	31.57	0.99
1	50	1.46	0.08	7.08	31.57	0.97
2	100	1.46	0.09	6.63	31.40	0.94
3	150	1.46	0.10	5.96	31.49	0.92
4	200	1.46	0.12	5.21	32.71	0.92
5	250	1.46	0.15	4.76	33.57	0.94

### 3.10 Alternative 10 – Supplemental Alkalinity

As illustrated in Figure 20, alkalinity can be added to support stable nitrification. Nitrification consumes alkalinity, which can lead to a reduction in pH if sufficient alkalinity is not present. Adding supplemental alkalinity increases buffering against a reduction in pH. The City already adds supplemental alkalinity in the form of sodium hydroxide (NaOH) to the process, so this alternative looks at the optimal alkalinity dose to provide adequate buffering with the least amount of chemical.

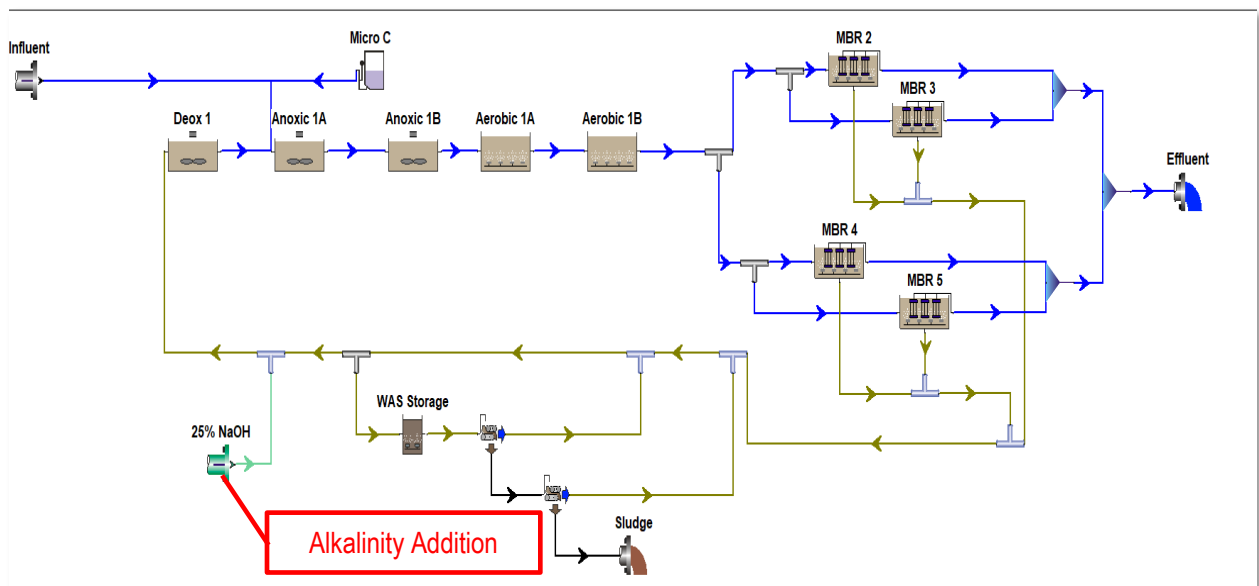


Figure 20 – Alternative 10 Illustration

The City currently doses about 586 gpd of 25% NaOH to the RAS prior to it being reintroduced into the process stream and combining with the influent. As shown previously in Table 3, it was assumed the influent alkalinity was 200 mg/L as calcium carbonate ( $\text{CaCO}_3$ ). This was estimated based on the supplemental alkalinity dose of 586 gpd and the effluent pH data provided. Subsequent influent alkalinity data provided by the City for 2023 had an average of 197 mg/L as  $\text{CaCO}_3$ . Therefore, the initial assumption was retained as it was representative of actual data.

To gauge the impact of NaOH specifically at different doses, the model was run simulating addition of sodium hydroxide at different doses, as illustrated in Figure 21. These model results indicate that the NaOH dose can be reduced without negatively impacting effluent pH or nitrification. At a dose of 300 mg/L as  $\text{CaCO}_3$ , which is nearly half the current dose, the process model predicts a slight drop in pH to only about 6.9 and effluent ammonia still below 0.1 mg/L.

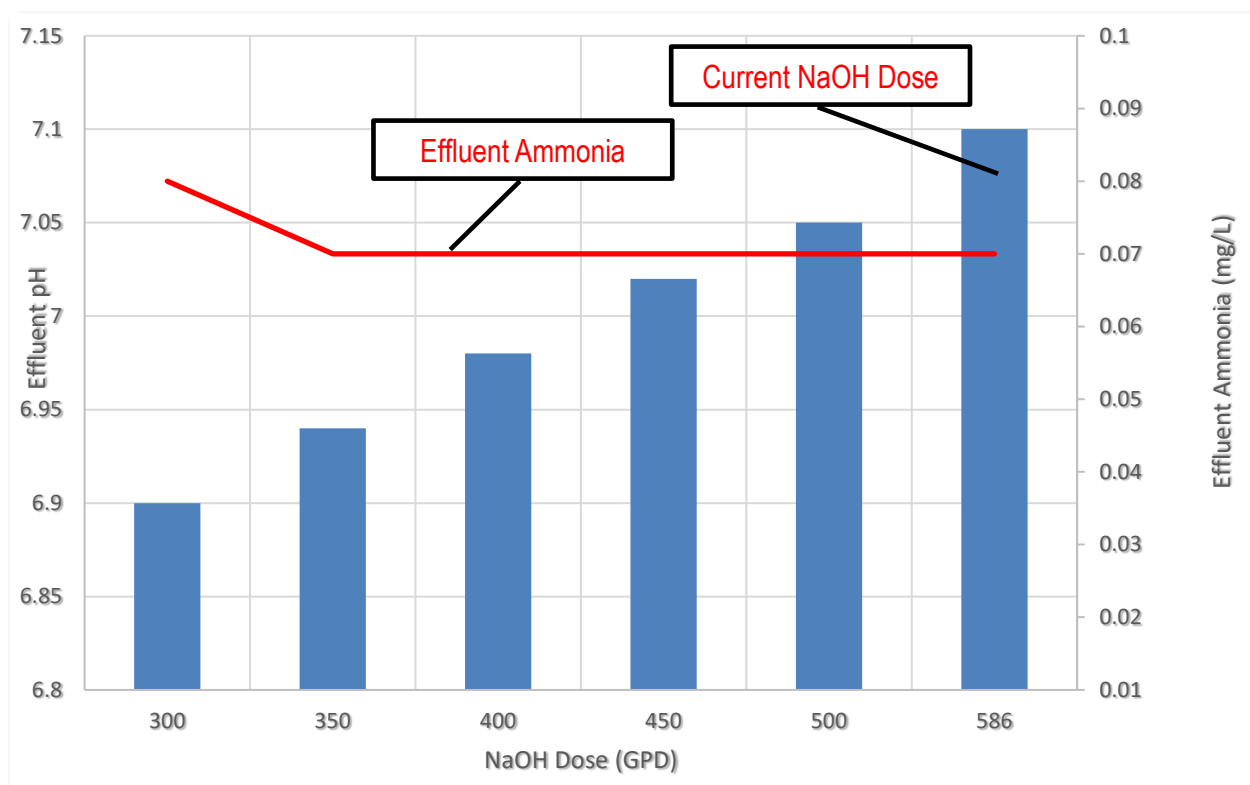


Figure 21 – Addition of Supplemental Alkalinity

A summary of the performance at current and reduced doses of NaOH are also shown in Table 11 below, including the associated cost reduction. Based on the current price of about \$1.54 per gallon of 25% NaOH, the annual cost savings could be up to about \$160,000 at a dose of 300 gpd, which is a reduction of 286 gpd from the current average dose of 586 gpd. It should be noted that the results shown in Figure 21 and Table 11 will

vary with seasonal changes in temperature, influent alkalinity concentration, and influent nitrogen load, all of which impact the amount and efficiency of nitrification.

**Table 11**  
**Sodium Hydroxide Addition**

NaOH Dose (gpd)	Influent Flow (MGD)	Effluent TN (mg/L)	Ammonia (mg/L)	pH	Cost Savings (\$/year)
586	1.46	7.47	0.07	7.10	-
500	1.46	7.47	0.07	7.05	\$48,341
450	1.46	7.47	0.07	7.02	\$76,446
400	1.46	7.47	0.07	6.98	\$104,551
350	1.46	7.47	0.07	6.94	\$132,656
300	1.46	7.47	0.08	6.90	\$160,761

### *3.11 Alternatives Selected for Further Evaluation*

The following is a summary of the discussion and process modeling results for the identified alternatives:

- Alternative 1 – Adjusting DO Set Points: Lower DO set points appear to reduce effluent nitrogen without impacting nitrification and save energy.
- Alternative 2 – Adjusting MBR RAS Rate: Increasing the RAS rate to 5Q appears to yield a significant reduction in effluent nitrogen.
- Alternative 3 – Adjusting Process SRT: Adjustments to SRT do not appear to have a significant benefit on effluent quality. Therefore, Alternative 3 will not be evaluated further, except as it may relate to other alternatives. However, it appears longer SRT can reduce WAS without too high MLSS or negatively impacting effluent quality.
- Alternative 4 – Cycling Sludge Storage Aeration: Cycling aeration does not appear to improve effluent quality but might save some energy without negatively impacting effluent quality. Therefore, this alternative will not be considered further.
- Alternative 5 – Adjusting Dewatering/Drying Operating Schedule: Adjusting the operating schedule for solids handling appears to have no noticeable impact on effluent quality, so continuing to operate in the most efficient manner seems appropriate.
- Alternative 6 – Increasing Treatment Volume: Putting the second or third aeration basin in service does not improve effluent quality significantly and would yield additional labor/expense.
- Alternative 7 – Altering Number of MBR Tanks: Taking one or more MBR tanks offline does not significantly improve effluent quality and would result in higher fluxes and possibly increased fouling/cleaning.

- Alternative 8 – Use of Available Flow Equalization: EQ does not appear to improve effluent quality and may actually yield slightly worse performance.
- Alternative 9 – Supplemental Carbon: Addition of supplemental carbon could be implemented to further increase nitrogen removal as required.
- Alternative 10 – The dose of supplemental alkalinity could be reduced to save cost without negatively impacting effluent pH or nitrification.

Alternatives 1, 2, 9, and 10 have been selected for further evaluation.

## 4. Evaluation of Selected Nutrient Optimization Alternatives

The evaluation of selected Alternatives 1, 2, 9, and 10 involves further process modeling to predict performance based on recommendations for implementation.

### 4.1 *Implementation of Selected Nutrient Optimization Alternatives*

The implementation of selected Alternatives 1, 2, 9, and 10 is divided into 3 phases. Phase 1 is for immediate implementation. Phase 2 is for implementation in the near future. Phase 3 is for implementation in the later future if needed.

#### 4.1.1 *Phase 1 – Immediate*

It is recommended that Alternatives 1 and 10 be implemented immediately. Because both alternatives could yield a large change in effluent quality compared to current operations, it is recommended that the changes be made incrementally and only for one alternative at a time. It is recommended that the City first incrementally reduce the DO set point for the two aerobic zones starting with set points of 2.0 mg/L with an ultimate target of 0.5 mg/L. It is recommended that the DO set point not be reduced in increments larger than 0.5 mg/L and that about one (1) month (nearly two full SRTs) pass between adjustments to ensure that the process has stabilized, and sufficient nitrification is still occurring before proceeding to the next incremental drop. Once 0.5 mg/L has been reached and a month has passed, the performance can be assessed to see if a further reduction makes sense, since the model predicts the DO set points could be as low as 0.25 mg/L.

Once the new DO set points are established, the supplemental alkalinity dose can be reduced incrementally in the same fashion from the current dose that averages 586 gpd. It is recommended the City reduce the supplemental alkalinity dose to a target of about 300 gpd. This adjustment should be made in relatively small steps (e.g., 50 gpd increments) to ensure sufficient alkalinity remains for buffering pH. In addition to monitoring pH, it is also suggested that effluent alkalinity be monitored to ensure it remains above at least 80 mg/L as CaCO<sub>3</sub>, which is typically suitable for buffering pH. The two DO set points and supplemental alkalinity dose should not be reduced simultaneously, as it would be difficult to determine which parameter should be adjusted up if there is a noticeable increase in effluent ammonia. It should not be necessary to wait a long period

between adjustments to supplemental alkalinity as long as the effluent pH remains near neutral and effluent alkalinity remains above 80 mg/L as CaCO<sub>3</sub>. Waiting about a week between adjustments should be sufficient.

There is no cost associated with implementing Alternatives 1 or 10 and in fact both should yield significant savings.

#### *4.1.2 Phase 2 – Near Future*

It is recommended that Alternative 2 be implemented in the near future. As mentioned previously, the existing RAS pumps do not have sufficient capacity to achieve the optimal 5Q recycle rate and struggle to maintain the current recycle rate target of 4Q at high flows. Therefore, the RAS pumps must be upgraded first to facilitate a higher recycle rate.

The existing RAS pumps are listed as having a capacity of 8 MGD each, which should be plenty of capacity. Therefore, further investigation is needed to determine why the pumps are underperforming. The following is recommended to troubleshoot the RAS pump underperformance:

- Review set points and control strategy to ensure pumps aren't being unknowingly constrained.
- Review shop and or field performance test results to ensure pumps met the original design criteria.
- Review operating discharge pressure to ensure pressures are not higher than anticipated. If pressures are higher, assess headloss and determine if the actual operating pressures should be higher or if there might be a blockage or unintended constriction.
- Pull a pump and review its condition to determine if there has been significant wear of the impeller and/or casing. Mixed liquor in the MBR tank is typically saturated with dissolved oxygen. The change in pressure that occurs in the pump causes some of the dissolved oxygen to come out of solution, and therefore could cause cavitation that would degrade the casing and impeller. If this is the case, a replacement impeller can be fabricated of a harder material to extend the life of the impeller and/or a protective coating applied for added protection.

If it is determined that the pumps need to be replaced, they can be sized and swapped out. Presumably the existing discharge piping is adequately sized given the pump's design flow. Therefore, upgrading the pumps as a worst case would involve new pumps, motors, and variable frequency drives (VFDs) to accommodate the larger motors. Assuming a cost of about \$150,000 for each new pump and VFD, the total cost would likely be on the order of \$500,000. The cost could be considerably lower if it is determined that the pumps only require rehabilitation, not replacement, or simple modifications to settings or controls.

#### *4.1.3 Phase 3 – Later Future (If Needed)*

It is recommended that Alternative 9 only be implemented if necessary to achieve greater nitrogen removal than can be achieved by implementing Alternatives 1, 2, and 10, since Alternative 9 requires the ongoing purchase and use of supplemental carbon at considerable cost. Additionally, there would be a relatively small capital cost



to purchase and install a chemical dosing system for metering the supplemental carbon. Assuming bulk storage was not required and instead totes were utilized for storage of the supplemental carbon, the capital cost would likely be on the order of \$150,000 for purchase and installation of a chemical dosing system and programming modifications for automated metering.

## *4.2 Predicted Results of Selected Alternatives*

The alternatives selected for evaluation were modeled in order of the recommended implementation to predict what the resulting nutrient removal is anticipated to be at each phase of implementation and the resulting costs in dollars per pound of additional TIN removed. Additionally, potential risks or consequences associated with implementation of these alternatives are discussed.

### *4.2.1 Phase 1 – Immediate*

Initial modeling for Alternative 1 indicated that lower DO set points would reduce effluent nitrogen without negatively affecting nitrification. As shown in Table 13, the process model predicts nearly a 50 percent reduction in effluent total nitrogen with a DO set point of 0.25 mg/L for both aerobic zones and more than a 30 percent reduction in effluent total nitrogen with a DO set point of 0.50 mg/L for both aerobic zones compared to the baseline scenario. As discussed previously, based on results of this process modeling, it is highly likely the process will perform effectively with a DO set point of 0.50 mg/L and there is good reason to believe adequate performance could be achieved at DO set points as low as 0.25 mg/L. Therefore, DO concentrations of 0.25 mg/L and 0.5 mg/L in the aerobic zones were utilized in model runs estimating the anticipated TN reduction. These model runs also included a lower NaOH dose of 300 gpd for supplemental alkalinity to confirm the model did not predict any negative impacts with lower DO set points in combination with a lower NaOH dose, which was confirmed. The results of these model runs are shown in Table 12.

On average, it is estimated that the effluent TN load could be reduced by about 28.8 pounds per day (lbs./day) using DO set points of 0.5 mg/L and potentially reduced by 44.2 lbs./day using DO set points of 0.25 mg/L. Because there is no capital or ongoing operation and maintenance (O&M) cost associated with reducing the DO set point or supplemental alkalinity dose, the cost per additional pound of TN removed (\$/lb. TN) is \$0/lb. TN.

Although the process model does not predict any issues or complications resulting from these operational changes, the changes should be made one at a time, incrementally, and with sufficient time between changes to confirm no adverse effects. This will avoid the potential for a significant process upset should there be unknown impacts that could not be predicted by the model or were not predicted based on available information input into the model.

Table 12  
Phase 1 Performance Predictions

Phase	Zone 1A DO Set Point (mg/L)	Zone 1B DO Set Point (mg/L)	NaOH Dose (gpd)	RAS Recycle Rate	Carbon Dose (gpd)	Effluent NH <sub>3</sub> (mg/L)	Effluent TN (mg/L)	Effluent TN Load (lbs./day)	TN Reduction (%)
Baseline	2.24	4.41	586	4Q	0	0.07	7.47	91.0	0
1	0.50	0.50	300	4Q	0	0.11	5.11	62.2	31.6
1	0.25	0.25	300	4Q	0	0.22	3.84	46.8	48.6

#### *4.2.2 Phase 2 – Near Future*

Once the RAS pumps are rehabilitated or upgraded to provide capacity per the original design intent, the RAS recycle rate can be increased to 5Q, which previous process modeling indicated to be optimal. As shown in Table 13 below, this change is estimated to further increase TN reduction by about 7.7% with DO set points of 0.50 mg/L and about 4.0% with DO set points of 0.25 mg/L reducing the effluent TN load by 7.0 lbs./day and 3.7 lbs./day, respectively.

In addition to the capital expenditure discussed previously, increasing the target RAS recycle rate from 4Q to 5Q will also use a little more energy. Although detailed information on the RAS pumping system was not provided, it is estimated that the increase energy cost would probably be less than \$2,000 per year, such that the increased energy use would yield an ongoing cost of less than \$2/lb TN removed with DO set points of 0.25 mg/L and less than \$1/lb TN removed with DO set points of 0.50 mg/L.

Although the process model does not predict any issues or complications resulting from this operational change, effluent and mixed liquor nitrate should be monitored to ensure it is having a positive impact and predicted by the model and that the increased recycle is not reducing the effectiveness of the anoxic zone due to greater amounts of DO being recycled. It is possible that as flows and loadings increase over the years, the optimal RAS recycle rate may eventually be lower as there may be more benefit to having less DO recycled and a greater proportion of the anoxic zones as truly anoxic. Currently, the anoxic zones are so large compared to the flow and loads that even with increased DO there is still sufficient anoxic volume to achieve efficient denitrification.

Table 13  
Phase 1 and 2 Performance Predictions

Phase	Zone 1A DO Set Point (mg/L)	Zone 1B DO Set Point (mg/L)	NaOH Dose (gpd)	RAS Recycle Rate	Carbon Dose (gpd)	Effluent NH <sub>3</sub> (mg/L)	Effluent TN (mg/L)	Effluent TN Load (lbs./day)	TN Reduction (%)
Baseline	2.24	4.41	586	4Q	0	0.07	7.47	91.0	0
1	0.50	0.50	300	4Q	0	0.11	5.11	62.2	31.6
1	0.25	0.25	300	4Q	0	0.22	3.84	46.8	48.6
2	0.50	0.50	300	5Q	0	0.12	4.53	55.2	39.3
2	0.25	0.25	300	5Q	0	0.24	3.54	43.1	52.6

#### *4.2.3 Phase 3 – Later Future (If Needed)*

As shown in Table 13 above, the CWF could achieve effluent TN below 5 mg/L with implementation of Phases 1 and 2. If future permit limits required further reduction, supplemental carbon could be dose as needed, whether seasonally or to boost performance during periods of high flows and loads.

A summary of potential CWF performance with implementation of Phases 1, 2, and 3 is shown in Table 14 below. The process model was run at doses of supplemental carbon between 100 and 250 gpd for both DO set points of 0.50 and 0.25 mg/L. It was assumed that MicroC® 2000 was used as the source of supplemental carbon. MicroC® has a very high COD value (1,100,000 mg/L) and is easy and safe to handle. As shown in Table 14, there is some increase in TN reduction with supplemental carbon, but the increased reduction quickly diminishes as the supplemental carbon dose increases.

At a cost of about \$4 per gallon, the cost to increase TN reduction using MicroC® is substantial, not including the cost to purchase and install a chemical dosing system and make programming modifications for automated metering. A summary of the increased TN reduction and associated cost for the MicroC® is shown in Table 15 versus what would be achieved implementing only Phases 1 and 2. With a DO set point of 0.5 mg/L, the cost of using MicroC® varies from about \$93 to \$99 per pound of TN. The cost more than doubles with a DO set point of 0.25 mg/L because it is more difficult to reach the lower concentrations of effluent TN. In neither case is use of supplemental carbon cost effective. As a result, it should only be used if necessary and only periodically if possible. It is much more cost effective to optimize DO set points and the RAS recycle rate for TN reduction prior to design any supplemental carbon.

If dosing of supplemental carbon became necessary, care should be taken not to overdose supplemental carbon. Not only will this unnecessarily increase costs, but could also result in bleed through of the carbon and increase aeration requirements and/or yield higher effluent BOD.

Table 14  
Phase 1, 2, and 3 Performance Predictions

Phase	Zone 1A DO Set Point (mg/L)	Zone 1B DO Set Point (mg/L)	NaOH Dose (gpd)	RAS Recycle Rate	Carbon Dose (gpd)	Effluent NH <sub>3</sub> (mg/L)	Effluent TN (mg/L)	Effluent TN Load (lbs./day)	TN Reduction (%)
Baseline	2.24	4.41	586	4Q	0	0.07	7.47	91.0	0
1	0.50	0.50	300	4Q	0	0.11	5.11	62.2	31.6
1	0.25	0.25	300	4Q	0	0.22	3.84	46.8	48.6
2	0.50	0.50	300	5Q	0	0.12	4.53	55.2	39.3
2	0.25	0.25	300	5Q	0	0.24	3.54	43.1	52.6
3	0.50	0.50	300	5Q	100	0.13	4.18	50.9	44.4
3	0.50	0.50	300	5Q	150	0.14	4.03	49.1	46.0
3	0.50	0.50	300	5Q	200	0.14	3.86	47.0	48.4
3	0.50	0.50	300	5Q	250	0.15	3.70	45.1	50.4
3	0.25	0.25	300	5Q	100	0.25	3.37	41.0	54.9
3	0.25	0.25	300	5Q	150	0.26	3.30	40.2	55.8
3	0.25	0.25	300	5Q	200	0.27	3.24	39.5	56.6
3	0.25	0.25	300	5Q	250	0.28	3.19	38.8	57.4

Table 15  
Supplemental Carbon Cost

Phase	DO Set Point (mg/L)	Carbon Dose (gpd)	Effluent NH <sub>3</sub> (mg/L)	Effluent TN (mg/L)	Effluent TN Load (lbs./day)	TN Reduction (lbs./day)	MicroC® Cost (\$/lb.) TN
1 and 2	0.5	0	0.12	4.53	55.2	0	\$ 0.00
3	0.5	100	0.13	4.18	50.9	4.3	\$ 93.02
3	0.5	150	0.14	4.03	49.1	6.1	\$ 98.36
3	0.5	200	0.14	3.86	47.0	8.2	\$ 97.56
3	0.5	250	0.15	3.70	45.1	10.1	\$ 99.01
1 and 2	0.25	0	0.24	3.54	43.1	0	\$ 0.00
3	0.25	100	0.25	3.37	41.0	2.1	\$ 190.48
3	0.25	150	0.26	3.30	40.2	3.1	\$ 193.55
3	0.25	200	0.27	3.24	39.5	3.6	\$ 222.22
3	0.25	250	0.28	3.19	38.8	4.3	\$ 232.56