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Water Reclamation Facility Modernization Study

FINAL
February 11, 2019

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Appendix J: UV Disinfection Engineering Report
Appendix K: Annual Solids Disposal Reports
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List of Acronyms

Abbreviation	Definition
A2O	Anaerobic / Anoxic / Oxic Configuration
ADWF	Average Dry Weather Flow
AO	Anaerobic / Oxic Configuration
ARCY	Anoxic Recycle
BFP	Belt Filter Press
BOD_5	Five Day Biological Oxygen Demand
CEC	Contaminants of Emerging Concern
COD	Chemical Oxygen Demand
ESD	Equivalent Single-Family Dwelling
GBT	Gravity Belt Thickener
MGD	Million Gallons per Day
MWC	Mark West Creek
NCRWQCB	North Coast Regional Water Quality Control Board
NRCY	Nitrified Recycle
SWRCB	State Water Resources Control Board
TMDL	Total Maximum Daily Load
Town	Refers to either Windsor Water District or Town of Windsor.
TSS	Total Suspended Solids
UCT	University of Cape Town Configuration
VSR	Volatile Solids Reduction
WRF	Water Reclamation Facility

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1. Purpose and Background

The Windsor Water District operates the Windsor Water Reclamation Facility (WRF). For simplicity "Town" will be used to refer to either the Town of Windsor, or the Windsor Water District. The WRF was constructed in 1964 to serve the Windsor area, which had a population of 772 people at the time. Today, the WRF serves a population of approximately 27,000 in the Town and some surrounding areas.

The Town recycles almost two-thirds of its tertiary treated wastewater, with almost half of that amount being used for energy production at the Geysers Steamfields. The remaining effluent flow is discharged to Mark West Creek. The current permitted capacity of the WRF for average dry weather flow conditions (ADWF) is 1.9 MGD, but the treatment design capacity is 2.25 MGD. The ADWF treatment capacity is constrained by the overall reclamation system reuse and disposal capacity. The current maximum permitted peak week wet weather capacity is 7.2 MGD. The Town is permitted to discharge to Mark West Creek between October 1 and May 14 of each year.

Over the past 50 years, the WRF has grown in size and has been upgraded several times, although some of the original structures are still in use. The WRF is located at 8400 Windsor Road, across from Windsor High School. The location of the WRF is shown in **Figure 1.1.**

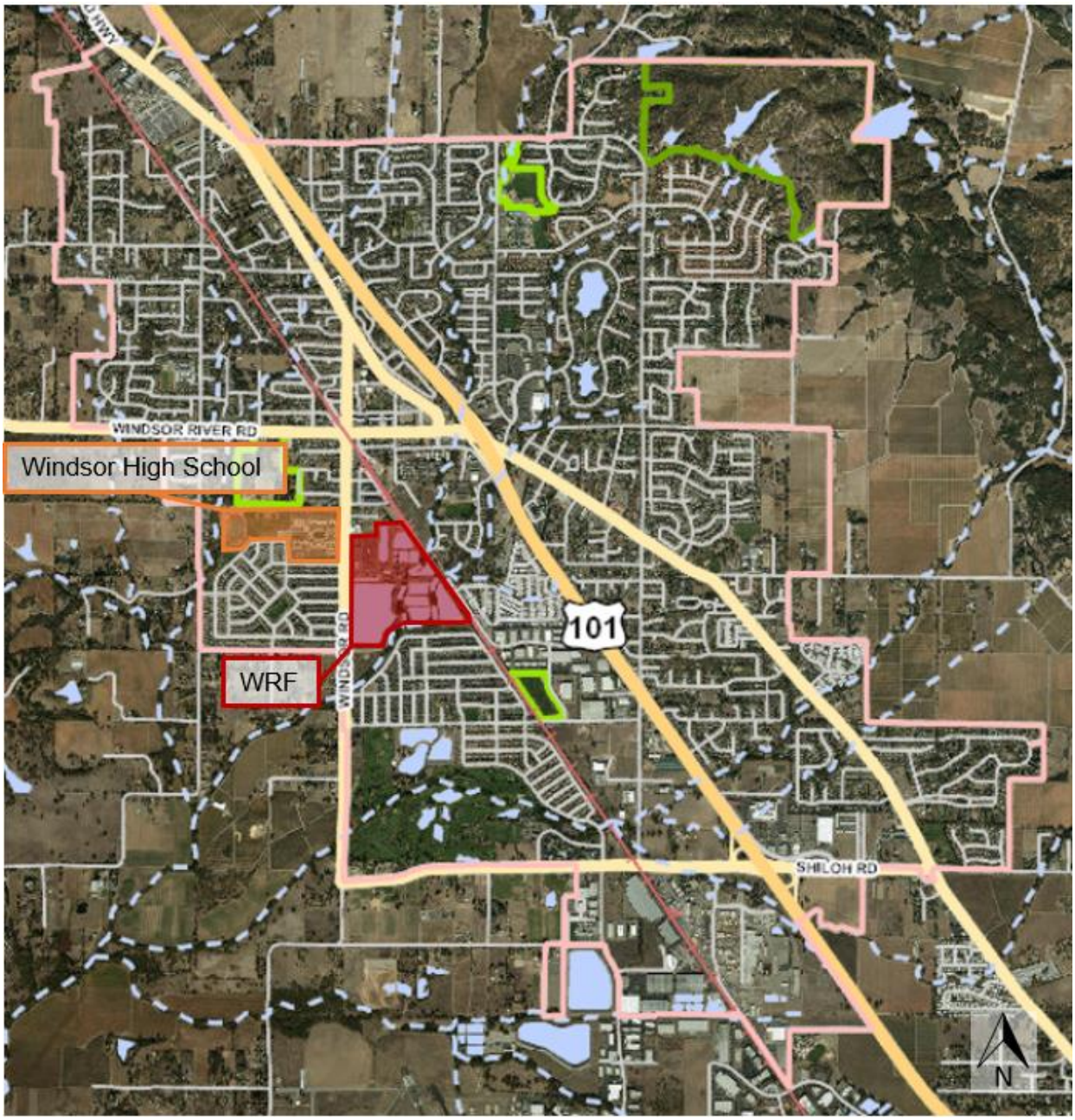


Figure 1.1: Location of WRF

The treatment process consists of the following:

- Influent pump station;
- Headworks with mechanical screening, grit removal via grit separator and classifier, and flow measurement;

- Extended activated sludge and secondary clarification;
- Chemical addition for tertiary flocculation, tertiary clarification, and sand filtration;
- Disinfection with UV light; and
- Biosolids stabilization ponds.

Section VI, C, 7.a, Table 9 of the Town's current NPDES permit no. CA0023345 (Order no. R1-2013-0042) issued in November 2013 and effective for 5 years commencing on February 1, 2014, required submittal of a treatment plant modernization and optimization study by February 1, 2016. This study was to include an evaluation of options to achieve mandated stepdown effluent phosphorus loading limits to Mark West Creek. The required submittal was completed by February 1, 2016.

The existing WRF meets the Town's current needs and regulatory requirements, however many of the mainstay treatment processes are approaching their 20th year in service. Additionally, improvements may be necessary to meet compliance with future pending regulatory requirements. While this Water Reclamation Facility Modernization Study (Study) is intended in part to address the regulatory requirements associated with the phosphorus removal, the Town is taking this opportunity to also assess the entire, whole treatment facility processes.

1.1 Goals and Objectives for the Study

The Study should include an approach to addressing phosphorus removal consistent with the North Coast Regional Water Quality Control Board (NCRWQCB) compliance schedule and identify opportunities to enhance and improve the liquid and solid phase treatment. Given the background and WRF history, the overall goals of this study are:

- Meet the requirements of the Town's current NPDES permit pertaining to the modernization and optimization study;
- Evaluate and identify processes for the modernization and upgrade of the Town's aged water reclamation facility which address process effectiveness, energy efficiency, and current and anticipated future regulations;
- Serve as a "roadmap" for the Town for making incremental and phased improvements to the WRF to reliably serve its customers for the next 20 years; and
- Develop cost and determine feasibility of major recommended projects.

1.2 Treatment Plant of the Future

As the Town looks to the future and developing a plant of the future, the following will be important to the overall analysis:

- Aging Infrastructure
- Treatment Capacity
- Regulations
- Sustainability/Optimization
- Feasibility and Cost
- Timing of Identified Projects and Upgrades

The plant of the future will be focused on proven treatment technologies that can be utilized to meet future nutrient limits, wholesale recycled water needs and technologies for solids handling and resource recovery that are sustainable and without adverse impacts to the environment. It is important to recognize the industry is rapidly changing and that emerging and innovative technologies that could offer improved life cycle costs and/or reduced footprint requirements are ever emerging. As part of the plant of the future the Town would want to stay abreast of these technologies to determine which options would best meet the future needs of the Town's WRF.

1.3 Scope of Work

The following is the Scope of Work for the Study.

- **Evaluation of Existing Conditions:** This task includes the review of reports, historical data, drawings, discharge operation model, and general WRF information. This task also includes field sampling, data analysis including a statistical analysis of historical WRF influent wastewater flows and loads data

- **Phosphorus Removal Strategy Development:** This task includes development of a BioWin™ process model based on WRF unit process data to evaluate the WRF's performance. This task also includes development of protocol for onsite jar testing of chemical coagulants. This information was used to evaluate various phosphorus removal strategies to identify modeling scenarios including future loading and operational changes to simulate reduction of effluent

- **Effluent Storage Evaluation:** This task includes collecting existing data from operations staff and other consultants familiar with the WRF (where appropriate) to consider options to increase storage capacity or utilization of the effluent ponds to support an increase in the permitted AWDF to meet the WRF's treatment system capacity. The task also includes options to increase storage capacity through addition of storage and through operational changes to optimize use of existing storage.

- **Grit and Fine Screening Facility Evaluation:** This task is focused on identifying improvements needed for the fine screening and grit removal facilities. The Town is currently in the process of undertaking the engineering design of a new coarse screen to protect the influent pumps and provide improved maintenance. The coarse screen project is expected to be underway during the modernization study.

- **UV Disinfection System Evaluation:** This task includes an evaluation of the capacity, energy efficiency, and retrofit costs for the existing system to determine the most appropriate future system and will provide a high-level analysis of three UV systems

(Ozonia 40HO, Calgon C3500D, and in-vessel technologies) based on the current NWRI "Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse".

- **Sludge/Solids Management Evaluation:** This task is focused on the solids process evaluation based on reduction of the volume of sludge that must be dewatered and the associated truck trips that are required for off-site disposal. The options to be evaluated include sludge drying beds, mechanical dewatering, and mechanical thickening.
- **Alternatives Analysis:** This task includes the development of planning-level costs for process alternatives developed in Tasks 3 through 7. Costs will include initial costs, O&M costs, energy costs (where appropriate), and present worth.
- **Full Scale Alumer Testing and Sampling**: This task includes jar testing and the development of full-scale testing of Alumer for optimal phosphorus removal.
- **Toxicity Testing**: This task includes assistance to the own in determining the best approach for toxicity testing, developing a sampling plan and review of the test results and report to the NCRWQCB.
- **Nutrient Off-Set Assistance**: This task includes assistance to the Town in coordinating with the NCRWQCB on nutrient off-set and banking of credits as well as coordination with the City of Santa Rosa and the development of cost/benefit analyses.
- **Additional Capital Improvement and Repair Development**: This task includes identification and assistance in evaluating and providing costs for repair and rehabilitation, capacity related and regulatory projects.
- **WRF Capacity Analysis:** This task focuses on a review of the ultimate biological treatment capacity of the WRF based on the current loading after the change in sampling and develops a list of capital projects to address and shortfall in the WRF biological treatment capacity. This task also updates the previous study entitled "Small Winery Classification and Investigation Regarding Wastewater Capacity Fees".
- **Reporting:** This task includes the development of draft technical memorandum, and preparation of a draft final report that will also include introduction, Capital Improvement Program sections (with recommendation and suggested order, if appropriate), and appendices.

1.4 Report Organization

This report is organized into the following sections:

- Section 1 – Purpose and Background
- Section 2 – Service Area
- Section 3 – Flows and Loads
- Section 4 – Regulatory Requirements
- Section 5 – Existing Facilities
- Section 6 – Water Reclamation Facility Capacity Analysis

- Section 7 – Phosphorus Removal Strategy
- Section 8– Effluent Storage
- Section 9 – Fine Screen and Grit Removal - Headworks
- Section 10 – Ultraviolet Disinfection
- Section 11 – Sludge and Solids Management
- Section 12 – Capital Improvement Plan

Appendices include the following:

- A: Town of Windsor NPDES Permit
- B: Conditions Assessment
- C: Capital Replacement and Maintenance Plan – Brelje and Race
- D: Process Air Calculations and Review of Design and Current Loadings
- E: Secondary Clarifier State Point Analysis
- F: Modernization and Optimization Study
- G: Water Balance Model TM – Brelje and Race
- H: Effluent Storage Cost Estimates
- I: Grit and Fine Screens Cost Estimates
- J: UV Disinfection Engineering Report
- K: Annual Solids Disposal Reports

2. Service Area

The purpose of this section is to provide an understanding of the land uses within the Town of Windsor to better understand domestic, commercial, and industrial loads on the WRF treatment systems. Allowable land use affects population growth and potential industrial growth within the Town. Windsor is centrally located within the Sonoma County wine country and has strong economic and cultural ties to the larger region.

2.1 Service Area Description

The Town currently serves 27,000 people within its service area, as described in **Section 1**; this includes an estimated population of 1,000 persons who reside outside the Town limits to the southeast. The Town of Windsor 2040 General Plan describes long term plans for growth and land use within the Town's urban growth boundary/sphere of influence, as shown in **Figure 2.1**.

The Town prepares for growth in order to meet its goal to "operate and maintain the Town's sanitary sewer system, wastewater treatment plant and reclamation facilities to provide adequate capacity for existing residents, businesses and future development."

The Town includes residential, commercial, industrial, and mixed-use regions. Residential development is regulated by a maximum number of dwelling units per acre (DU/Ac) standard. Commercial and industrial development is regulated by a maximum floor area ratio (FAR) standard. Mixed use development is regulated by both standards.

A large portion of the Town is reserved for low density and very low density residential dwellings. Rural residential and estate residential regions sit along the edges of the urban growth boundary. For the most part, commercial, mixed use, and high-density dwellings lie adjacent to the Highway 101 corridor and along Old Redwood Highway. Commercial regions are located such that they are easily accessible.

Industrial users are predominately located along the south of the Town, adjacent to Highway 101. The beverage industry is a major part of the local industrial economy. The Town treats wastewater from six local wineries which together, generated an estimated 41,600 cases of wine in 2017. This number of cases is expected to increase to 56,600 cases in 2018. The Russian River Brewing Company also opened up within the Towns service area in late summer of 2018. It is anticipated that the beer and wine industry will continue to grow.

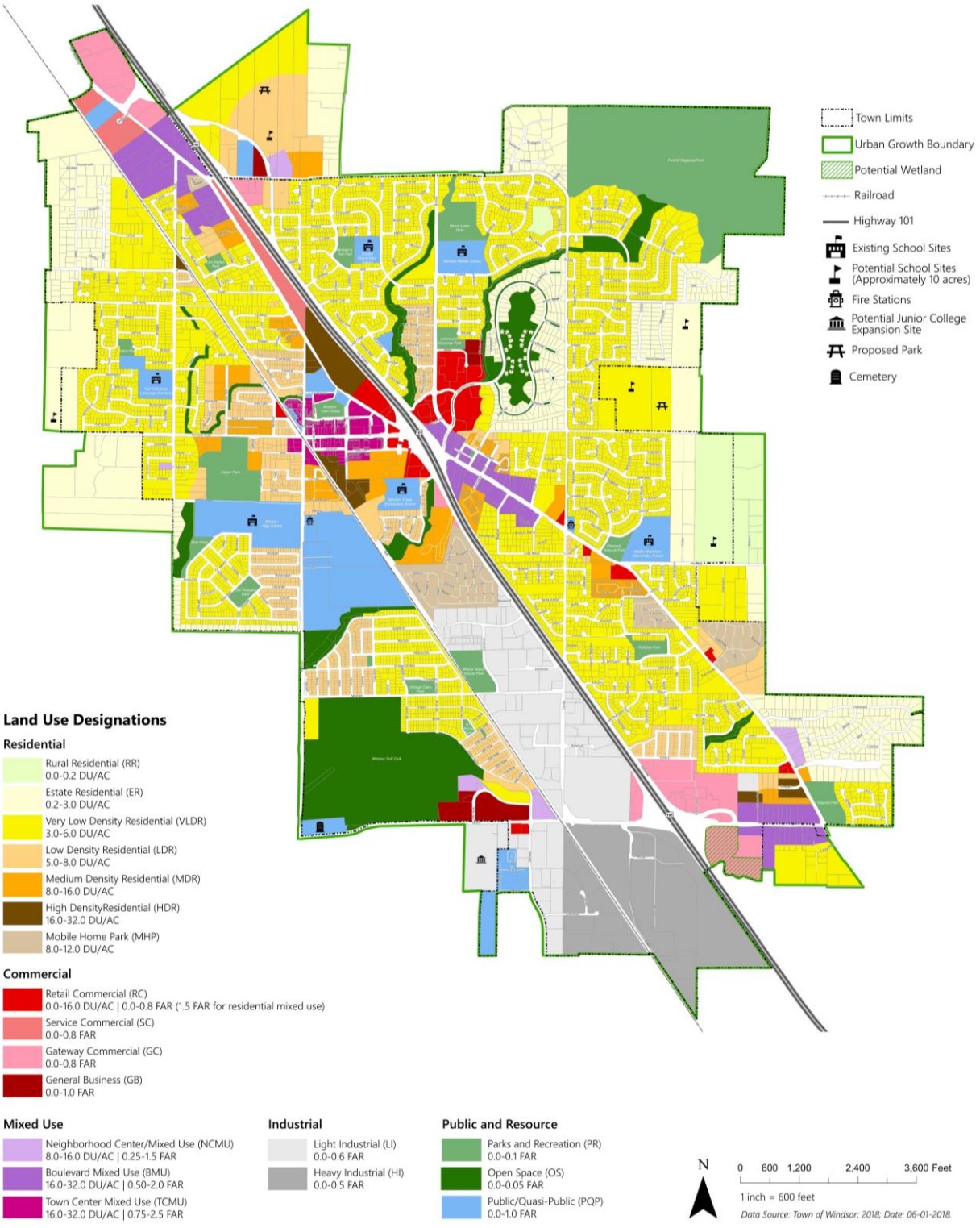


Figure 2.1: Land Use Diagram: 2040 General Plan

3. Flows and Loads

This section reviews historical data to determine the current flows and loads at the Windsor Water Reclamation Facility (WRF). This section will also use the projected growth from the 2040 General Plan to determine future flows and loads at the WRF that should be planned for treatment through 2040.

3.1 Background

Influent samples are taken after the headworks treatment processes, which include screening, a grit chamber, and a Parshall flume. It should be noted that influent composites include recycle streams such as sludge pond supernatant and filter backwash that are returned to the influent sewer. These loads are therefore captured in the influent sample to the facility.

The WRF measures influent composite samples for 5-day biological oxygen demand (BOD_5), total suspended solids (TSS), chemical oxygen demand (COD), ammonia (NH_3-N), and total phosphorus (TP) in addition to other parameters. These parameters provide valuable insight into wastewater influent characteristics. Comparing influent loading and post treatment water quality can demonstrate the overall performance of the facility.

In 2014 a change sampling protocol required by a change in NPDES permit requirements indicated that influent loading is greater than previously thought. Prior to February 2014, influent composite samples were collected between 10 pm and 6 am. A fixed volume of sample was collected hourly over the 8-hour period and was added to the influent composite sample. After February 2014, the influent sampling method was changed to 24-hr flow-weighted composite samples. The purpose of this change in sampling protocol was to provide data that was more representative of the daily loadings to the WRF. The change in sampling is noted on graphs in this section where influent loading data prior to February 2014 is presented.

3.2 Influent Flows

This section will review the historical flows to the WRF measured at the influent (INF-001 monitoring location). INF-001 is measured after the recycled stream flow enters the influent wet well, and therefore reflects the actual treatment stream. A calculated “influent adjusted flow” is also analyzed to estimate the influent flow prior to the addition of the plant recycle stream. The influent adjusted flow is used for the calculation of the WRF’s average dry weather flow. The INF-001 flow is used to calculate influent loading (**Section 3.3**). Historical influent flow is shown for the period of 2012-2017 in **Figure 3.1**. The change in sampling protocol did not affect the measured flow and is therefore not noted on **Figure 3.1**.

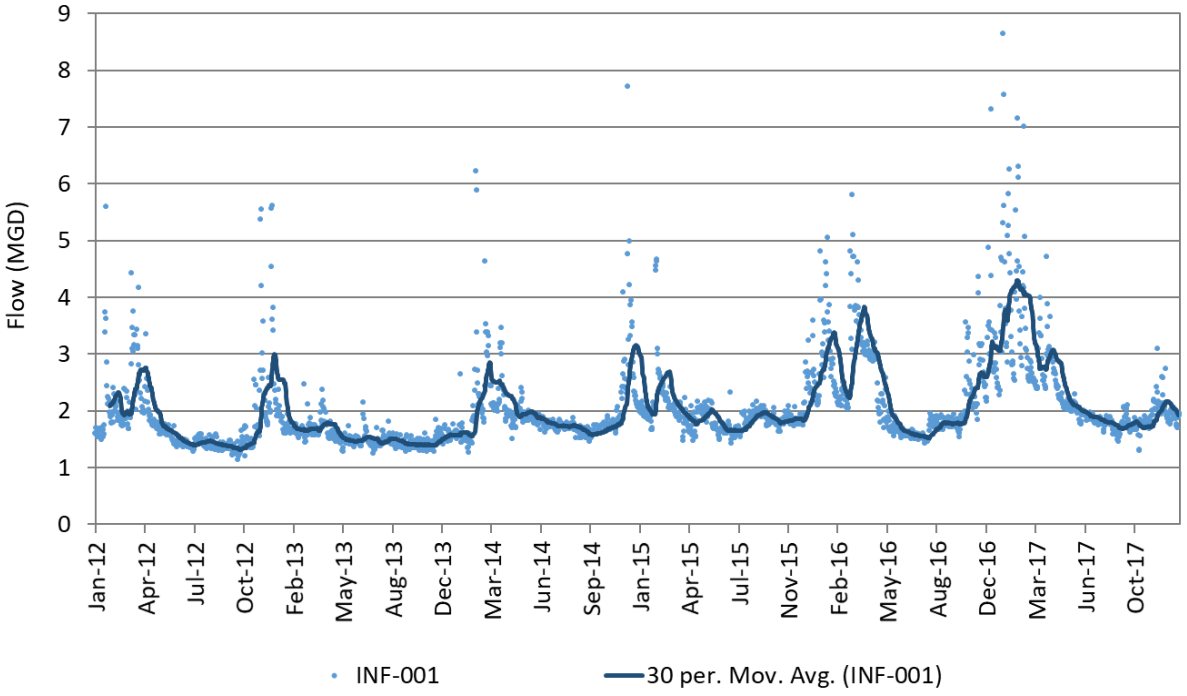


Figure 3.1: Influent (INF-001) Flow

Standard operating procedure at the WRF limits influent flows to a maximum of 5 MGD. This is controlled by utilizing influent equalization via high flow storage. Flows are diverted after the influent pump station but prior to headworks and INF-001. Flows during the wet season occasionally exceeded 5 MGD, however the standard operating procedure was generally followed.

Table 3.1 presents key historical influent flow parameters for the permit period. Although the change in sampling protocol did not affect the measured influent flows, a consistent set of data was used throughout this analysis. Thus, the calculated averages in this section do not include 2014 data, as only full year data taken after the change in sampling protocol was used. The loading calculations shown later in this section use a data set that is consistent with the average flows and peaking factors provided in **Tables 3.1** and **3.2**. The 2014 data is included in the table below for informational purposes.

Table 3.1: Historical Annual Influent Flows (INF-001)

Year	Annual Average (MGD)	Max Month (MGD)	Max 30-Day (MGD)	Max 7-Day (MGD)	Max Day (MGD)
2014	2.01	3.11	3.16	4.52	7.72
2015	1.97	2.71	3.06	3.87	4.66
2016	2.38	3.72	3.84	4.53	7.32
2017	2.41	4.13	4.30	5.74	8.64
AVERAGE [1]	**2.26**	**3.52**	**3.73**	**4.72**	**6.87**

[1]The average shown is for 2015 through 2017 in order to be consistent with the loading data set, which includes only full year data following the change in sampling in early 2014.

Annual flow peaking factors for the WRF are summarized in **Table 3.2**. Peaking factors are calculated as the ratio of max month, max 30-day, max 7-day or max day flows compared to the annual average flow.

Table 3.2: Historical Annual Influent Peaking Factors

Year	Max Month	Max 30-Day	Max 7-Day	Max Day
2014	1.55	1.57	2.25	3.84
2015	1.37	1.55	1.96	2.36
2016	1.56	1.61	1.90	3.07
2017	1.72	1.78	2.38	3.59
AVERAGE[1]	**1.55**	**1.65**	**2.08**	**3.01**

[1]The average shown is for 2015 through 2017 in order to be consistent with the concentrations and loadings data sets, which includes only full year data following the change in sampling in early 2014.

The annual average flow to the WRF is 2.26 MGD for the period evaluated (2015-2017) with an (average) maximum day flow peaking factor of 3.01. Average flow to the facility appears to have increased from 2014 to 2017, however it is likely that this trend is due to the drought years of 2014 and 2015 followed by wetter years in late 2016 and early 2017.

The average dry weather flow (ADWF) is defined as the lowest 30-day period of flow during a calendar year. ADWF is calculated using the influent flow (INF-001) minus the recycle flows to the WRF, known as the influent adjusted flow. A summary of average dry weather flows using the influent adjusted flow is provided in **Table 3.3.** The average ADWF for 2015 – 2017 was 1.34 MGD.

Table 3.3: Historical Average Dry Weather Flow

Year	Average Dry Weather Flow (MGD)
2014	1.36
2015	1.41
2016	1.34
2017	1.27
AVERAGE[1]	**1.34**

[1]The average shown is for 2015 through 2017 in order to be consistent with the concentrations and loadings data sets, which includes only full year data following the change in sampling in early 2014.

3.3 Wastewater Loads

This section discusses the historical daily loading of key parameters (BOD_5, TSS, COD, ammonia, and total phosphorus). These parameters are measured at INF-001 as 24-hour flow-paced composite samples and are analyzed once each week. The historical loading will be used in a later section for analyses of process performance and capacity (**Section 6**).

3.3.1 Influent Loading

3.3.1.1 Influent Biological Oxygen Demand

The Town takes an influent composite sample once a week to analyze for BOD_5; the BOD_5 analysis is not an inhibited BOD_5 test, which inhibits nitrification to limit interference with measurement of carbonaceous BOD_5. Influent load was calculated using the total (raw sewage plus treatment plant recycled flows) influent flow (INF-001) and recorded BOD_5 concentrations. **Figure 3.2** shows influent concentration and load for weekly samples of BOD_5 from 2012 through 2017 with 30-day rolling average trendlines. **Table 3.4** summarizes BOD_5 concentrations, loads and peaking factors. As influent composite samples are analyzed weekly, data is too infrequent to determine the 7-day maximum peaking factor for influent constituents.

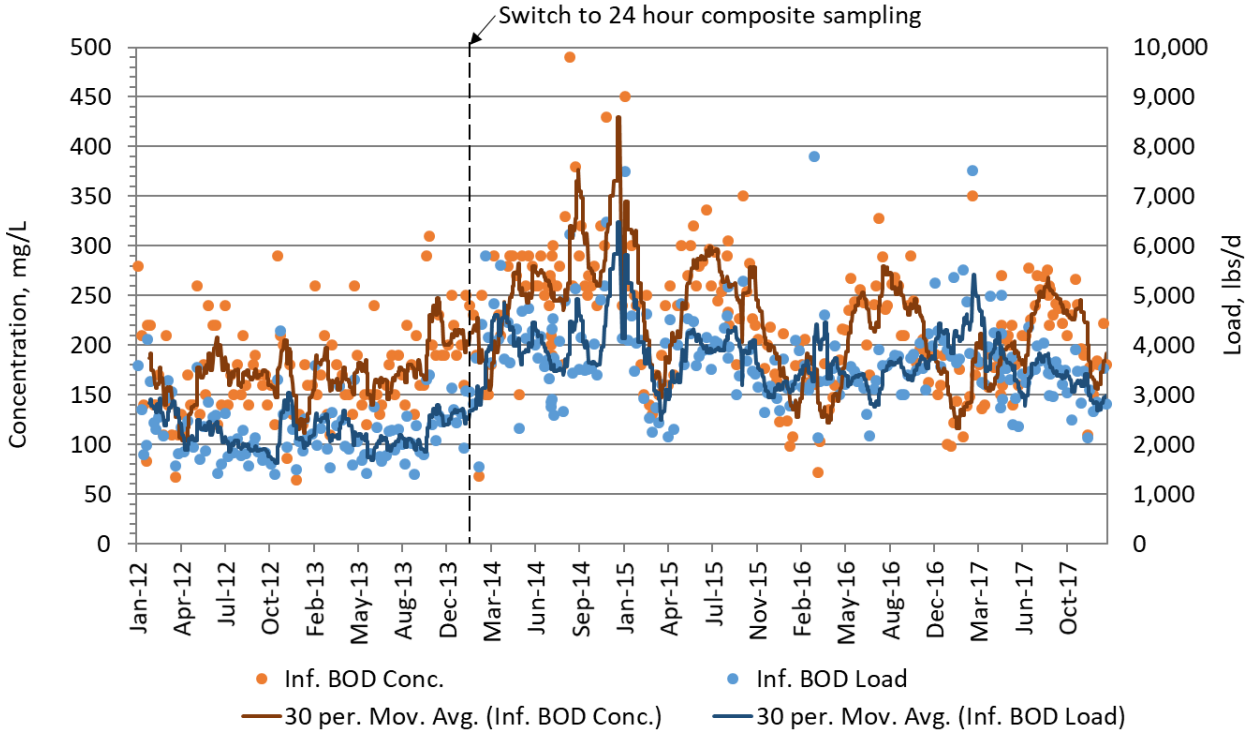


Figure 3.2: Influent BOD_5 Concentration and Load

Figure 3.2 shows an increase in measured BOD_5 concentration at the time the influent composite sampler was changed from an hourly composite taken between 10 pm and 6 am (8-hour composite) to a 24-hour flow-weighted composite. Prior to February 2014 influent BOD_5 averaged 173 mg/L and 2,290 lbs/day; after February 2014, average influent BOD_5 increased to 223 mg/L and 3,720 lbs/day. This increase is expected as influent loads are typically greater during the day, and eight-hour composite during the night hours did not capture daily loads accurately. Annual data following the change in sampling procedure are summarized in **Table 3.4.**

Table 3.4: Historical BOD_5

Year	Annual Average Conc.	Annual Average Load	Max Month Peaking Factor	Max 30-Day Peaking Factor	Max Day Peaking Factor
Feb. to Dec. 2014	252 mg/L	3,920 lbs/day	1.10	1.24	1.48
2015	234 mg/L	3,690 lbs/day	1.15	1.18	1.43
2016	202 mg/L	3,590 lbs/day	1.20	1.24	2.17
2017	199 mg/L	3,600 lbs/day	1.51	1.51	2.09
AVERAGE[1]	**212 mg/L**	**3,630 lbs/day**	**1.29**	**1.31**	**1.90**

[1]The average is for 2015 through 2017 which includes only full year data following the change in sampling in early 2014.

3.3.1.2 Influent Total Suspended Solids

Figure 3.3 shows weekly influent TSS sample concentrations and loads for 2012 through 2017 including a 30-day rolling average. **Table 3.5** summarizes TSS concentrations, loads and peaking factors.

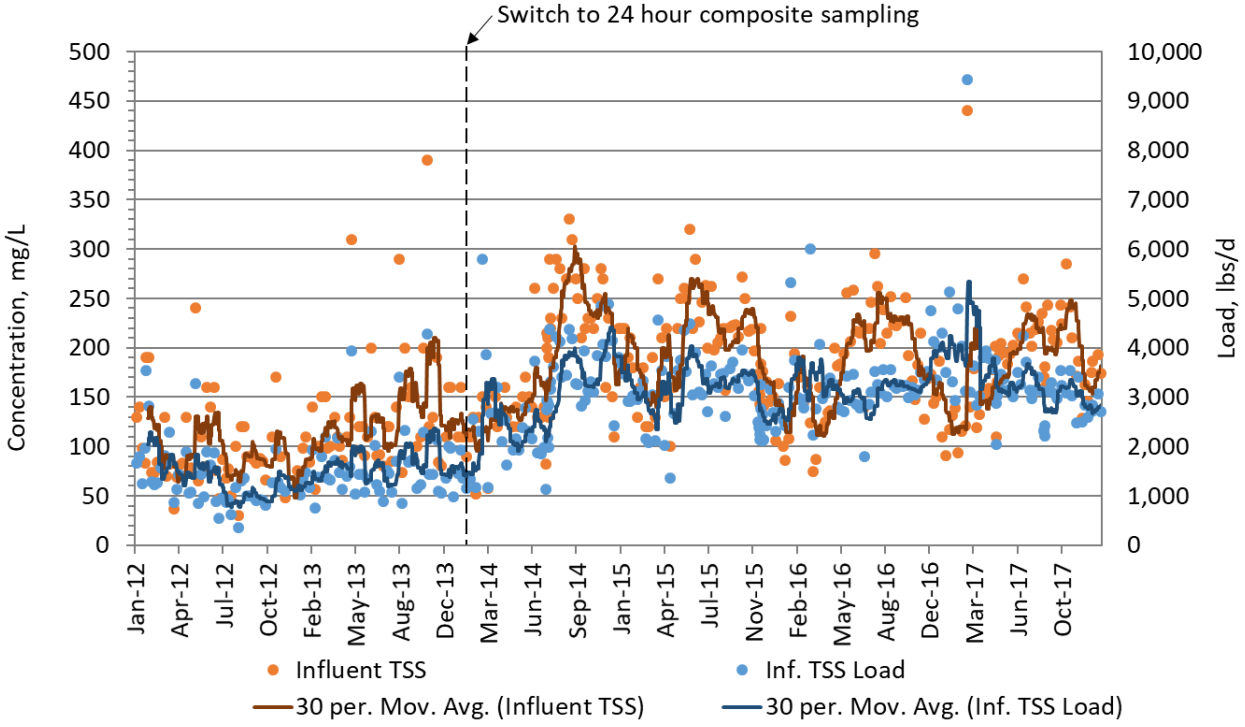


Figure 3.3: Influent TSS Concentration and Load

Table 3.5: Historical TSS

Year	Annual Average Conc.	Annual Average Load	Max Month Peaking Factor	Max 30-Day Peaking Factor	Max Day Peaking Factor
Feb. to Dec. 2014	188 mg/L	3,010 lbs/day	1.32	1.46	1.48
2015	195 mg/L	3,060 lbs/day	1.22	1.32	1.49
2016	185 mg/L	3,280 lbs/day	1.24	1.24	1.83
2017	179 mg/L	3,210 lbs/day	1.20	1.32	1.60
AVERAGE[1]	**186 mg/L**	**3,180 lbs/day**	**1.22**	**1.29**	**1.64**

[1]The average is for 2015 through 2017 which includes only full year data following the change in sampling in early 2014.

Figure 3.3 shows an increase in influent TSS concentration at the time the influent composite sampler was changed from an hourly composite taken between 10 pm and 6 am (8-hour composite) to a 24-hour flow-weighted composite. Influent TSS averaged 114 mg/L and 1,500 lbs/day prior to February 2014 and increased to 188 mg/L and 3,160 lbs/day after February 2014.

3.3.1.3 Influent Chemical Oxygen Demand

Figure 3.4 shows the weekly influent COD concentration and load from October 2014 through December 2017 as well as 30-day rolling averages. It should be noted that COD data was unavailable prior to October 2014.

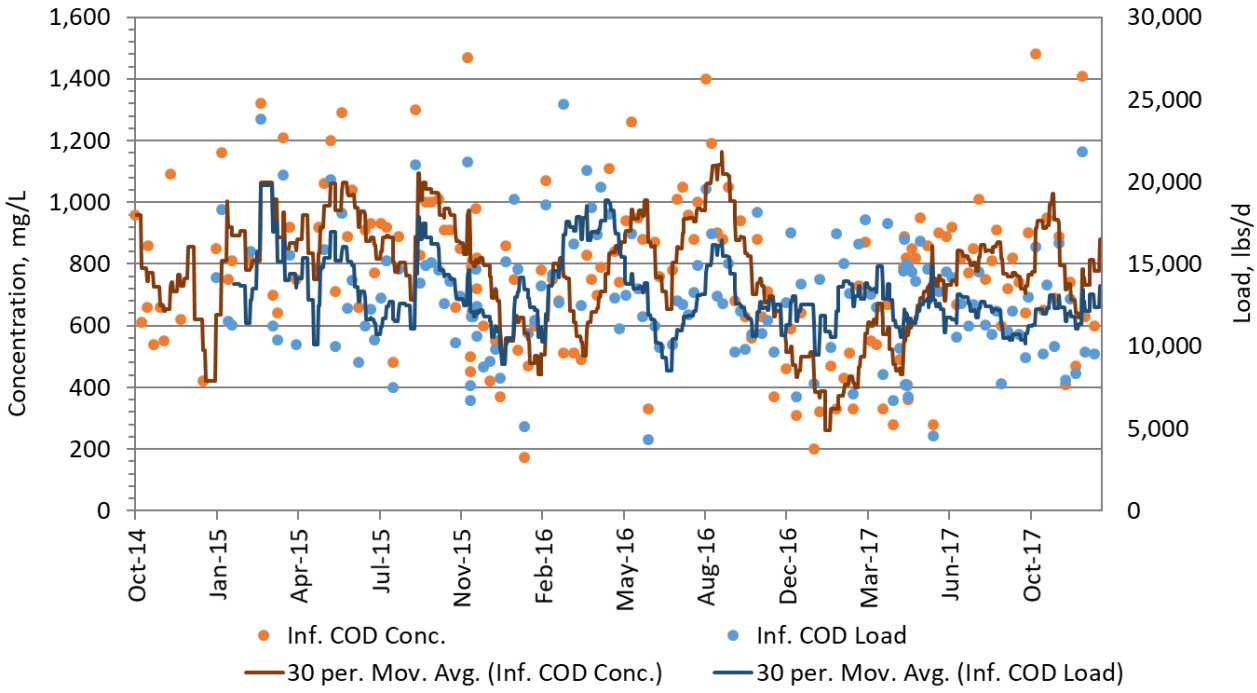


Figure 3.4: Influent COD Concentration and Load

Table 3.6 summarizes influent COD data for October 2014 through December 2017.

Table 3.6: Historical COD

Year	Annual Average Conc. (mg/L)	Annual Average Load (lbs/day)	COD/BOD_5 Ratio	Max Month Peaking Factor	Max 30-Day Peaking Factor	Max Day Peaking Factor
Oct. to Dec. 2014	697 mg/L	10,130 lbs/day	2.53	- [1]	- [1]	1.65
2015	817 mg/L	12,580 lbs/day	3.74	1.31	1.35	1.89
2016	766 mg/L	13,670 lbs/day	3.84	1.38	1.38	1.81
2017	670 mg/L	12,000 lbs/day	3.42	1.24	1.24	1.82
AVERAGE[2]	**751 mg/L**	**12,750 lbs/day**	**3.67**	**1.31**	**1.32**	**1.84**

[1]Not enough data in 2014 for proper calculation of this parameter
[2]The average is for 2015 through 2017 which includes only full year data following the change in sampling in early 2014.

COD/BOD_5 ratios are a characteristic of the influent coming into the WRF and are not affected by treatment processes. They are site-specific and provide an understanding of the biodegradability of organic compounds in the WRF influent. Raw wastewater COD/BOD_5 ratios typically observed by Hazen range from 1.8 to 2.5. The limited data indicates that the COD/BOD_5 ratio of the WRF influent may be significantly greater than typical values. This was confirmed during special sampling where the average COD/BOD_5 ratio was 3.7.

3.3.1.4 Influent Ammonia

Influent samples were analyzed weekly for ammonia concentration starting in January 2013**. Figure 3.5** shows influent ammonia concentrations and loads with a 30-day moving average trendline. **Table 3.7** summarizes average values.

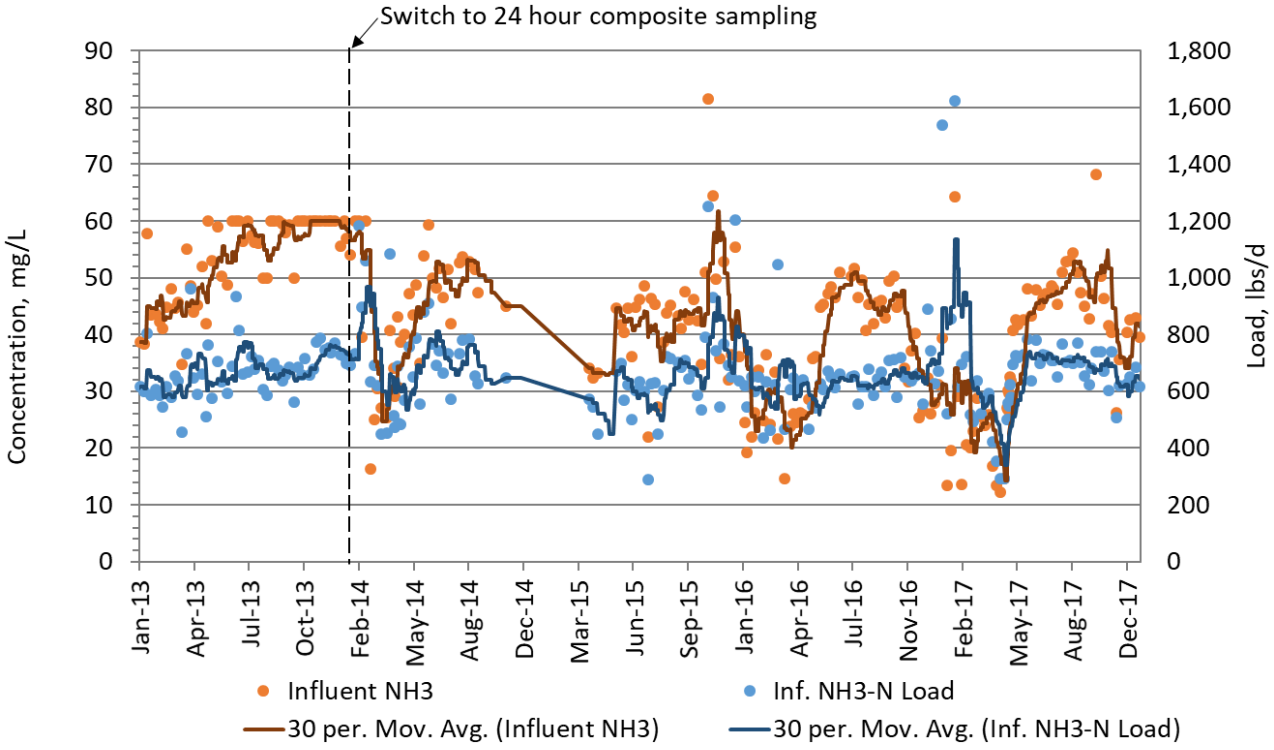


Figure 3.5: Influent Ammonia Concentration and Load

Table 3.7: Historical Ammonia

Year	Annual Average Conc.	Annual Average Load	Max Month Peaking Factor	Max 30-Day Peaking Factor	Max Day Peaking Factor
Feb. to Dec. 2014	43 mg-N/L	704 lbs/day	1.34	1.49	1.68
2015	41 mg-N/L	636 lbs/day	1.24	1.30	1.89
2016	36 mg-N/L	632 lbs/day	1.15	1.25	1.64
2017	38 mg-N/L	667 lbs/day	1.55	1.70	2.43
AVERAGE	**39 mg-N/L**	**647 lbs/day**	**1.31**	**1.42**	**1.99**

[1]The average is for 2015 through 2017 which includes only full year data following the change in sampling in early 2014.

Figure 3.5 indicates the influent ammonia concentration decreased at the time the influent composite sampler was changed to a 24-hour flow-weighted composite. Influent ammonia concentration averaged 54 mg-N/L and loading averaged 610 lbs/day prior to February 2014; after February 2014 influent ammonia averaged 39 mg-N/L and 663 lbs/day. WRF staff noted that prior to February 2014 ammonia samples were influent grab samples collected at 7am; they also noted that these samples were analyzed by a different method.

3.3.1.5 Influent Total Phosphorus

Weekly influent (24-hour composite) sampling for total phosphorus (TP) began in October 2014 **Figure 3.6** presents weekly influent TP concentrations and loads and a 30-day moving average trendline. **Table 3.8** summarizes the average values for the data available.

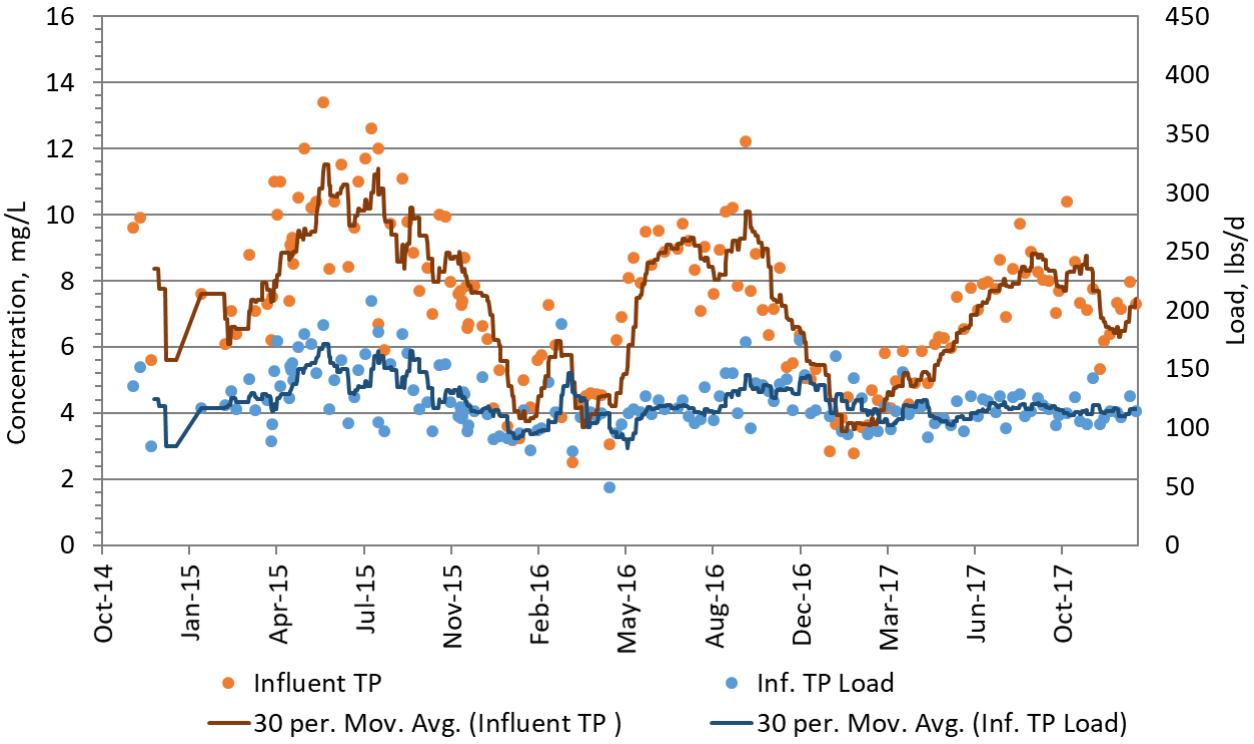


Figure 3.6: Influent Total Phosphorus and Load

Table 3.8: Historical Total Phosphorus

Year	Annual Average Conc.	Annual Average Load	BOD_5/TP Ratio	Max Month Peaking Factor	Max 30-Day Peaking Factor	Max Day Peaking Factor
Oct. to Dec. 2014	8.37 mg/L	124 lbs/day	45	- [1]	- [1]	1.22
2015	8.36 mg/L	129 lbs/day	28	1.20	1.33	1.61
2016	6.72 mg/L	117 lbs/day	32	1.16	1.25	1.60
2017	6.63 mg/L	114 lbs/day	32	1.09	1.20	1.41
AVERAGE[2]	**7.24 mg/L**	**120 lbs/day**	**31**	**1.15**	**1.26**	**1.54**

[1]Not enough data in 2014 for proper calculation of this parameter
[2]The average is for 2015 through 2017 which includes only full year data following the change in sampling in early 2014.

The average influent TP concentration was approximately 7.2 mg/L, and the influent TP load averaged 120 lbs/day. The BOD_5/TP ratio was 31, which is in the range typically observed for domestic wastewater. **Figure 3.6** shows that while there is seasonal variability of total phosphorus concentrations, the loadings remain fairly consistent.

3.4 Effluent Quality

UV effluent (EFF-001) quality is summarized below. UV effluent composite samples are taken weekly and analyzed for BOD_5 and TSS. Additionally, UV effluent grab samples are analyzed for total coliform daily. The Town also performs nutrient sampling for process control and monitoring. Effluent samples were 8-hr composite samples prior to February 2014. Effluent data reflect 24-hour flow-weighted samples after that time.

3.4.1 Effluent Biological Oxygen Demand

Figure 3.7 shows effluent BOD_5 concentrations reported to the NCRWQCB. These were collected weekly as a 24-hour flow-weighted composite sample. As described in **Section 4,** the WRF's BOD_5 average monthly and weekly effluent limits are 10 mg/L, and 15 mg/L, respectively. The limit of detection for the BOD_5 analysis at the WRF is 2 mg/L. Samples reported as "<2 mg/L" are shown as 2 mg/L in the graph below and in any further analysis. **Table 3.9** shows yearly average concentrations.

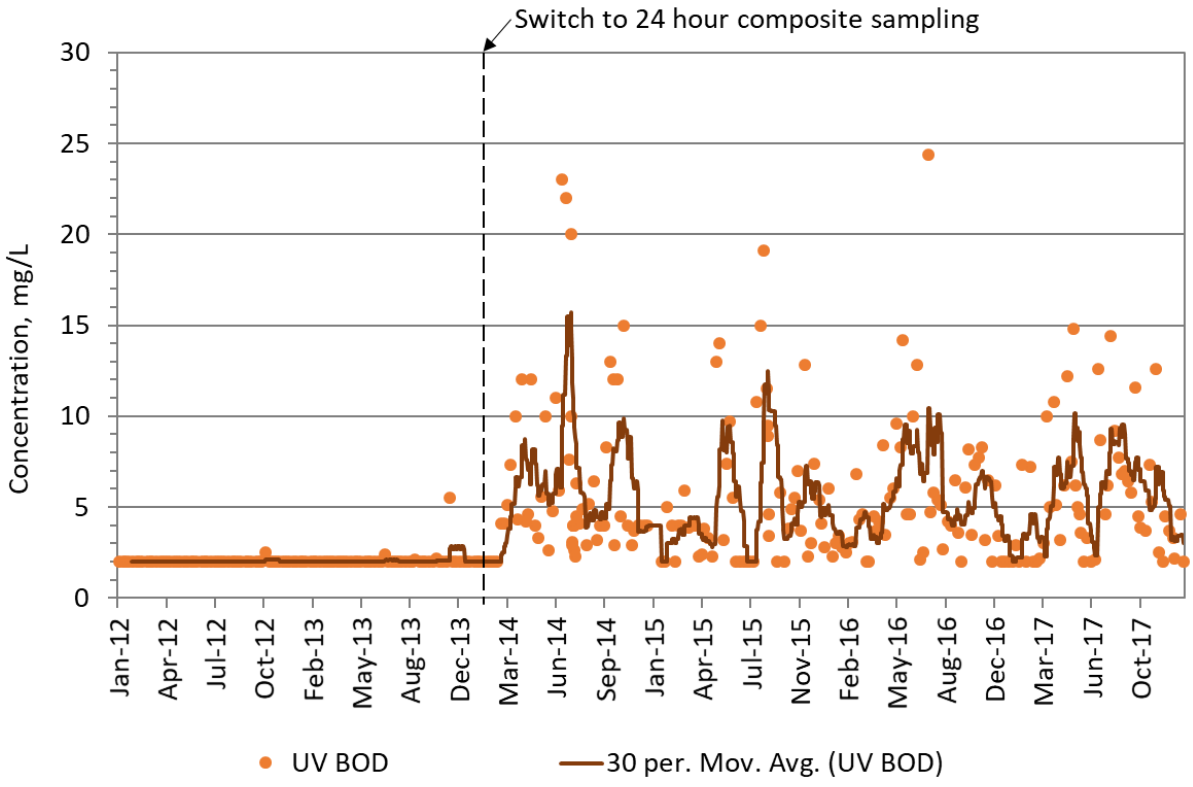


Figure 3.7: UV Effluent BOD_5

Table 3.9: Historical UV Effluent BOD_5

Year	Average UV Effluent BOD_5 (mg/L)
Feb. to Dec. 2014	6.1
2015	5.4
2016	5.4
2017	5.6
AVERAGE[1]	**5.5**

[1]The average is for 2015 through 2017 which includes only full year data following the change in sampling in early 2014.

Effluent BOD_5, though within the limits, is higher than expected given the lower effluent TSS concentrations. This indicates a higher than expected effluent concentration of soluble BOD_5. Soluble BOD_5 should be consumed by the aeration processes, so a higher than expected soluble BOD_5 indicates potential deficiencies in aeration. Insufficient dissolved oxygen in the Biolac® basins or hydraulic short-circuiting in the basins could result in this phenomenon. It should be noted that particulate forms of BOD_5 would be removed with TSS in the AWT System (flocculation, settling, and filtration).

It has recently come to light that reducing the standing water levels in the wet well, may contribute to lower effluent BOD_5 concentrations. It is believed that lowering the wet well pump level setpoints promotes more consistent flow velocities from the west trunk sewer and relieves BOD_5 surges from otherwise temporarily stagnated flow. The operations staff has implemented this solution with positive results thus far.

The WRF is usually able to meet its EFF-001 discharge limits for BOD_5. In accordance with the discharge permit, all exceedances are reported and addressed. It should be noted that over this study period most violations corresponded to warm weather periods when it is more difficult to maintain sufficient dissolved oxygen in the aeration process.

3.4.2 Effluent Total Suspended Solids

As the WRF has an advanced wastewater treatment system which includes flocculation, tertiary clarifiers and sand filters, effluent suspended solids at the EFF-001 discharge point are regularly maintained at a low level. Effluent samples are collected weekly as a 24-hour flow-weighted composite sample. **Figure 3.8** presents effluent TSS concentrations, and **Table 3.10** summarizes annual averages.

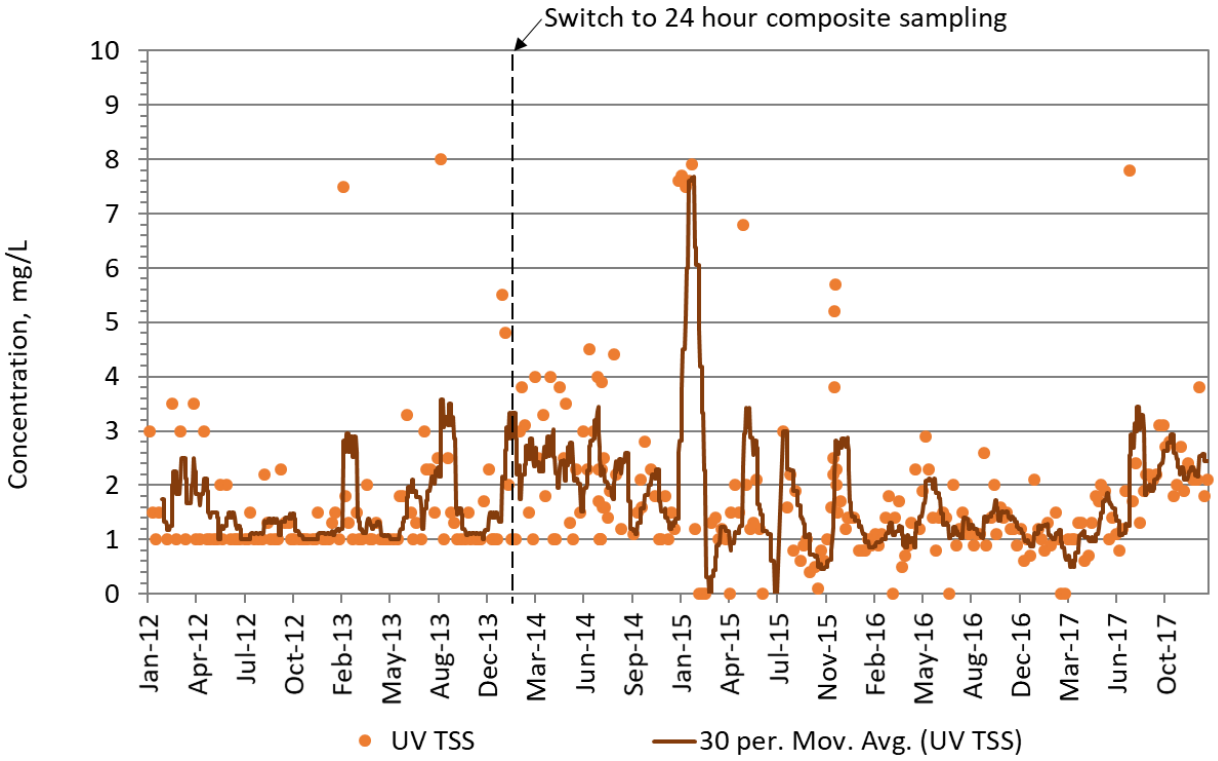


Figure 3.8: UV Effluent Total Suspended Solids

Table 3.10: Historical UV Effluent TSS

Year	Average UV Effluent TSS (mg/L)
Feb. to Dec. 2014	2.2
2015	2.4
2016	1.3
2017	1.9
AVERAGE[1]	**1.9**

[1]The average is for 2015 through 2017 which includes only full year data following the change in sampling in early 2014.

The permitted average monthly effluent TSS limit is 10 mg/L, and the average weekly limit is 15 mg/L (see **Section 4**). Compliance with effluent TSS limits was maintained throughout the study period. The average effluent TSS was 1.9 mg/L, although single samples measured as great as approximately 8 mg/L.

3.4.3 Effluent Ammonia and Nitrate

UV effluent ammonia was monitored weekly beginning in 2013 for process control. These samples are collected as a 24-hour flow-weighted sample. Nitrates are occasionally monitored, depending on lab availability. Effluent nitrogen data are presented in **Figure 3.9** and summarized in **Table 3.11**.

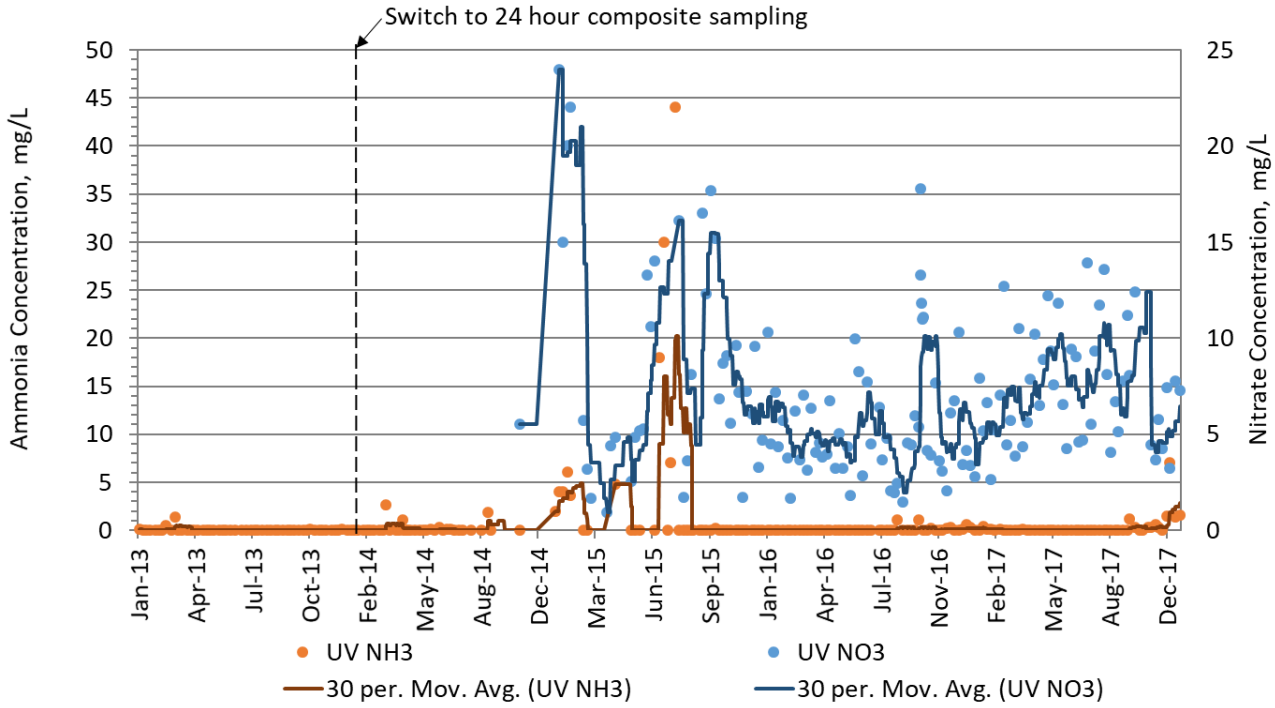


Figure 3.9: UV Effluent Nitrate and Ammonia

Table 3.11: Historical UV Effluent Nitrate and Ammonia

Year	Average UV Effluent NH_3-N (mg-N/L)	Average UV Effluent NO_3-N (mg-N/L)
Feb. to Dec. 2014	0.21	5.53
2015	3.17 [2]	8.87
2016	0.08	5.54
2017	0.29	7.33
AVERAGE[1]	**1.18 [2]**	**7.25**

[1]The average is for 2015 through 2017 which includes only full year data following the change in sampling in early 2014.
[2] The July 2015 data was abnormally high, and skews the 2015 average UV Effluent NH_3-N. This, in turn affects the 2015 through 2017 average.

The effluent ammonia data indicate that the WRF achieves full nitrification most of the time, with concentrations typically below 1 mg-N/L. It should be noted that the WRF lost nitrification in August 2015. Nitrite concentration data during that timeframe would be useful for assessing this and other similar periods of inhibited nitrification. The WRF's average effluent nitrate concentration is 7.25 mg/L. This indicates that the WRF performs substantial denitrification. Assuming an industry typical effluent dissolved organic nitrogen of 1 mg-N/L and minimal nitrite (the WRF samples for nitrite monthly and the results are usually non-detect), the data indicates that effluent total nitrogen (TN) concentrations are estimated at approximately 7-8 mg-N/L. This is supported by monthly total nitrogen sample results which generally show that WRF effluent is mainly in the form of nitrate.

3.4.4 Effluent Phosphorus

The Town began collecting UV effluent TP data in October 2014. These samples are collected as a 24-hour flow-weighted sample collected weekly. **Figure 3.10** and **Table 3.12** summarize the available UV effluent TP data.

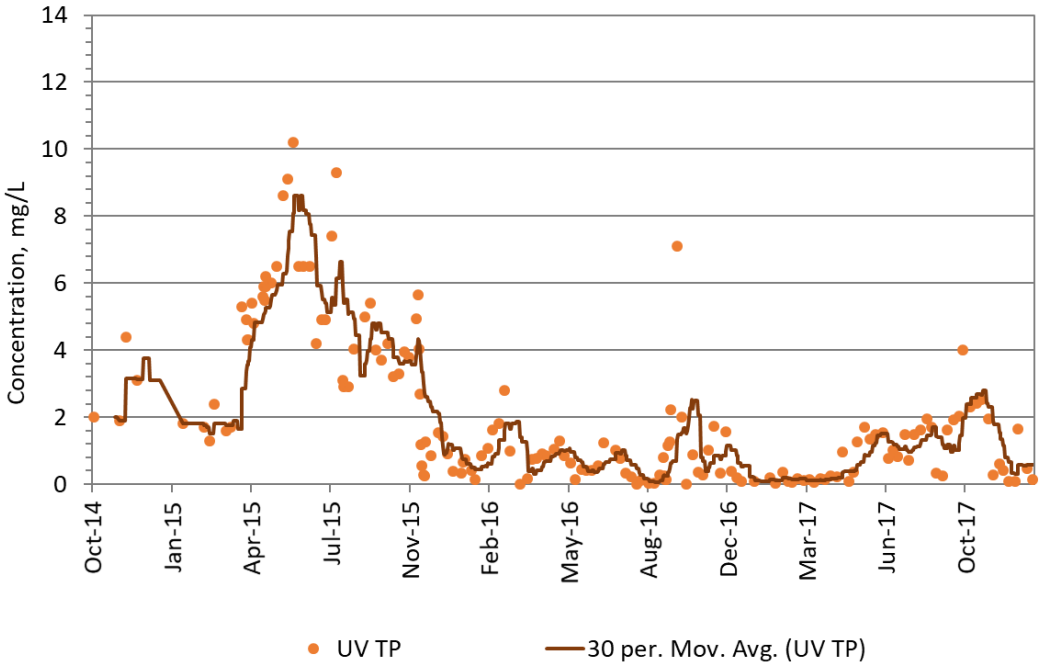


Figure 3.10: UV Effluent Total Phosphorus

Table 3.12: Historical UV Effluent Phosphorus

Year	Average UV Effluent TP (mg-P/L)
Oct. to Dec. 2014	2.85
2015	4.12
2016	0.88
2017	0.93
AVERAGE[1]	**1.97**

[1]The average is for 2015 through 2017 which includes only full year data following the change in sampling in early 2014.

UV effluent TP increased above 5 mg-P/L after April 2015, reaching 11 mg-P/L in June 2015. WRF staff noted that Alumer, a proprietary blend of alum and polymer supplied by Polydyne Inc. (SNF Polydyne), was not dosed to the flocculation tanks or filter influent during this time period to test the relative sensitivity of dosage to turbidity. Alumer addition to the AWT facilities resumed at the end of June 2015. Resuming alumer addition resulted in a decrease in UV effluent TP concentration.

UV effluent TP averaged 1.97 mg-P/L during the study period. The effluent TP decreased significantly between 2015 and 2016, likely due to changes in WRF operation. Prior to 2016, both Biolac® aeration basins were used. After 2016, the operation was changed to only one basin (Basin B) in order to get a better food to microorganism (F/M) ratio. This also likely resulted in the better selection of phosphate-accumulating organisms (PAOs) since the selector box in Basin B is larger and in better condition. The mixed liquor suspended solids concentration was also increased and resulted in greater settleability.

3.5 Future Projected Flows and Loads

The Water Reclamation Facility (WRF) serves approximately 27,000 people within its service area. The 1997 Growth Control Ordinance (amended in 2017) states that the Town will strive to control development to result in an annual average population growth rate of 1.5 percent. This is achieved by controlling the number of new residential units in the Town. The Town also proactively manages infrastructure capacity. Per the 2040 General Plan, every five years the Town must submit a report detailing remaining infrastructure capacity to ensure that the Town can provide necessary services to new residential developments. The most recent population projections in the 2040 General Plan estimate an average annual growth rate of 1.4 percent. Using this information, the population projections can be estimated for each year, as shown in **Table 3.13.**

Residential, commercial, and industrial wastewater flows were assumed to increase proportionally with the population. The baseline flows and loadings assumed in the projections were the average flows and loads for 2015 through 2017. These are reported previously in this section. The average population projections for 2015 through 2017 is approximately 27,300 residents. The 2040 General Plan projects that there would be 38,000 residents by 2040. The ratio of the 2040 population to 2015-2017 population is 1.39 or a 39 percent increase in population. 2015 through 2017 average flows and loadings for BOD_5, TSS, ammonia, and TP were multiplied by this factor (1.39) to estimate flows and loads in 2040. A

similar process was followed for each year between 2018 and 2040. Flow and loading projections are shown for BOD_5, TSS, ammonia, and TP in **Table 3.13**.

It is important to note that this analysis assumes that industrial growth also increases proportionally with population. It assumes that population growth will result in increasing industrial flows and loads from the same types of industries, or at least in similar ratios of effluent constituents. It does not account for unexpected growth in industry that may have abnormally high levels of a particular constituent, or significantly different ratios of constituents. It is assumed that industrial permits will regulate industrial effluent such that no particular constituent loading exceeds the WRF's treatment capability.

Table 3.13: Growth Projections and Future Projected Flows and Loads

Year	Population Estimate	ADWF (MGD)	Annual Average Flow (MGD)	BOD_5 Load (lbs/day)	TSS Load (lbs/day)	Ammonia (lbs/day)	TP Load (lbs/day)
Average 2015 - 2017	27,300	1.3	2.3	3,630	3180	650	120
2018	28,100	1.4	2.3	3730	3270	670	120
2019	28,500	1.4	2.4	3780	3310	670	130
2020	28,900	1.4	2.4	3840	3360	680	130
2021	29,300	1.4	2.4	3890	3410	690	130
2022	29,700	1.5	2.5	3940	3450	700	130
2023	30,100	1.5	2.5	4000	3500	710	130
2024	30,500	1.5	2.5	4050	3550	720	130
2025	30,900	1.5	2.6	4110	3600	730	140
2026	31,400	1.5	2.6	4170	3650	740	140
2027	31,800	1.6	2.6	4220	3700	750	140
2028	32,200	1.6	2.7	4280	3750	760	140
2029	32,700	1.6	2.7	4340	3800	770	140
2030	33,100	1.6	2.7	4400	3860	780	150
2031	33,600	1.6	2.8	4460	3910	800	150
2032	34,100	1.7	2.8	4520	3960	810	150
2033	34,500	1.7	2.9	4590	4020	820	150
2034	35,000	1.7	2.9	4650	4070	830	150
2035	35,500	1.7	2.9	4710	4130	840	160
2036	36,000	1.8	3.0	4780	4190	850	160
2037	36,500	1.8	3.0	4850	4240	860	160
2038	37,000	1.8	3.1	4910	4300	880	160
2039	37,500	1.8	3.1	4980	4360	890	160
2040	38,000	1.9	3.1	5050	4420	900	170

Table 3.13 provides the anticipated flows and loads for the WRF through 2040. For the projected population of 38,000 people, an ADWF of approximately 1.9 MGD is anticipated with an annual average flow of 3.1 MGD. The BOD load is anticipated to be 5,000 lbs/day and the TSS load would be approximately 4,400 lbs/day. The ammonia and TP loads are anticipated to be 900 lbs/day and 170 lbs/day, respectively. These projections are important, as they provide an estimate of the hydraulic and treatment capacities that will be required by 2040. These capacities are used for the WRF capacity evaluations in **Section 6**.

4. Regulatory Requirements

The purpose of this section is to summarize the existing discharge permit requirements and consider potential future regulatory requirements that may affect the Windsor WRF.

4.1 Summary of the Existing NPDES Permit

Under the Clean Water Act (CWA), the Environmental Protection Agency (EPA) regulates the wastewater effluent discharges through the issuance of National Pollutant Discharge Elimination System (NPDES) permits. The NPDES permitting program for the State of California and the Russian River Basin has been delegated by EPA to the North Coast Regional Water Quality Control Board (NCRWQCB). The WRF's current permit – NPDES Permit No. CA0023345, Order No. R2 2013-0042 – was adopted on February 1, 2014 and expires on January 31, 2019. The Windsor Water District owns and operates the WRF and is the discharger subject to the waste requirements set forth in the permit. The NPDES permit is included in **Appendix A**.

The WRF is designed for an average dry weather flow (ADWF) capacity of 2.25 million gallons per day (MGD), but the permit currently sets the limit to 1.9 MGD due to storage and disposal capacity limitations within the reclamation system. The WRF has a peak weekly wet weather flow (PWWWF) design capacity of 7.2 MGD, and a peak monthly wet weather flow of 3.75 MGD.

The WRF is permitted to discharge to the Mark West Creek seasonally (during the period of October 1 through May 14) and delivers recycled water to various customers during the year that include the following:

- Windsor High School and Windsor Fire Protection District Station No. 2 for landscape irrigation and toilet flushing
- The Geysers Recharge Project
- Santa Rosa Jr. College Shone Farm
- Various residential landscaping areas including Vintage Greens
- Various commercial and industrial landscaping areas including Conde Business Park
- Various Town parks including: Town Green, Keiser Park, Old Vineyard Park, Vintage Oaks Park, Star Creek Park, and the Wilson Ranch Soccer Park
- Windsor Golf Course
- Various other vineyards and agricultural customers.

4.2 NPDES Permit Effluent Limits

The WRF is permitted to discharge at two locations and distribute recycled water at five locations. These are summarized in **Table 4.1.** The effluent discharge locations are shown in **Figure 4.1**. The permitted

effluent discharge limits are summarized in **Table 4.2.** Reclamation limits can be found in the discharge permit Order No. R1-2013-0042 (see **Appendix A**) and are not listed here.

Table 4.1: WRF Monitored Discharge and Distribution Points

Point	Receiving Water	Effluent Description
EFF – 001	Town-owned effluent ponds	Disinfected tertiary municipal effluent
EFF – 002	Mark West Creek	
REC – 003A	Various irrigation discharges	
REC – 003B	Reclamation at Windsor High School and Windsor Fire Protection District Station No. 2	
REC – 004	Geysers Recharge Project	
REC – 005 [1]	Airport – Larkfield-Wikiup WWTF Reservoir D (Oceanview Pond)	

[1] REC-005 monitoring location is designated in the NPDES permit Order No. R1-2013-0042, this site is not a routine distribution point and is only monitored when in use.

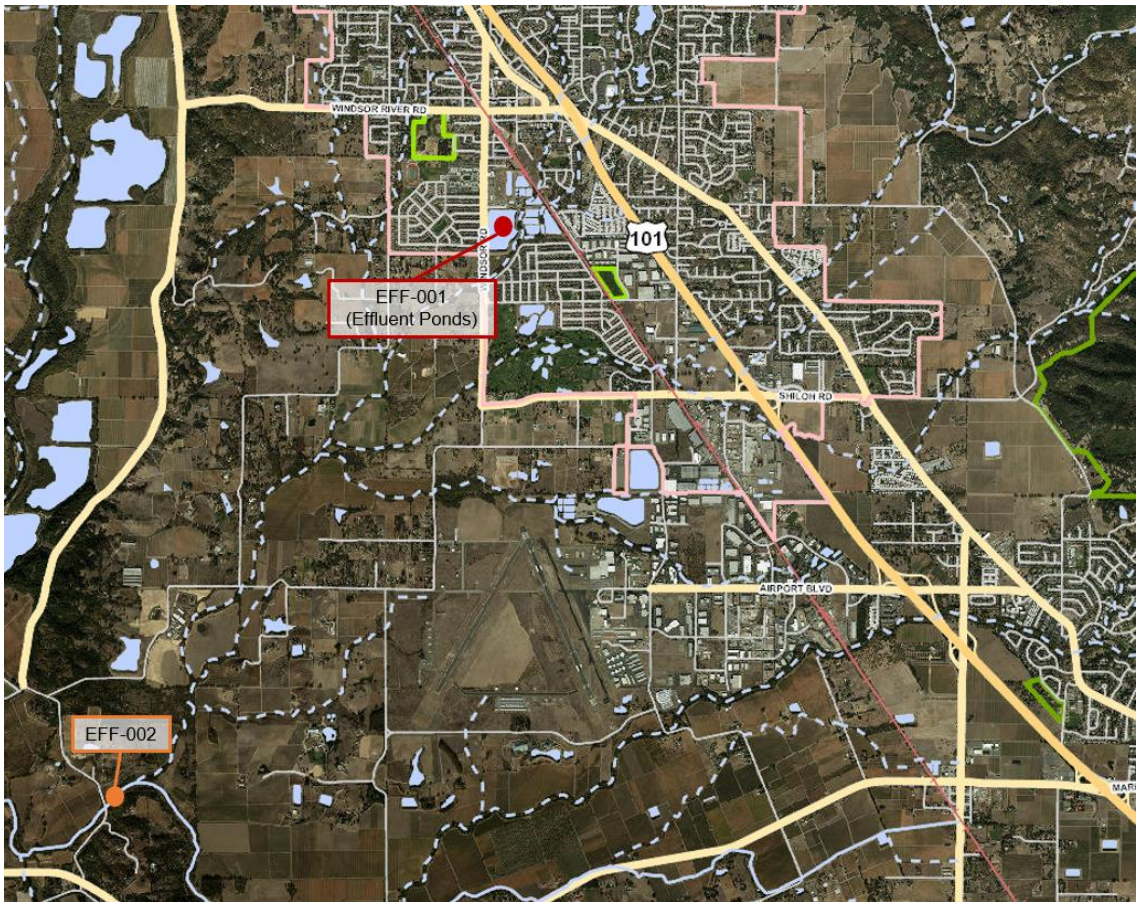


Figure 4.1: Location of Effluent Discharge Points

Table 4.2: WRF NPDES Permit Limitations Summary

Parameter	Units	Effluent Limitations
EFF – 001		
Five Day Biochemical Oxygen Demand (BOD_5)	mg/L	• Average Monthly: 10 • Average Weekly: 15 • Average Monthly Removal Minimum: 85%
Total Suspended Solids (TSS)	mg/L	• Average Monthly: 10 • Average Weekly: 15 • Average Monthly Removal Minimum: 85%
pH	-	• Instantaneous minimum; 6.0 • Instantaneous maximum; 9.0
Total Coliform	MPN / 100 mL	• 7 Sample Rolling Median: 2.2 • 30-day period Max: 23 • Single sample Max: 240
EFF – 002		
pH	-	• Instantaneous minimum; 6.5 • Instantaneous maximum; 8.5
96-Hour Bioassay of Undiluted Waste	% survival	• Single Bioassay Minimum: 70 • 3 Bioassay Rolling Median: 90
Total Nitrogen	mg/L	• Average Monthly: 10.5
Total Phosphorus	mg/L	• Average Monthly: 7.8 (Interim)

4.2.1 Total Phosphorus Loading

The NCRWQCB has imposed strict effluent phosphorus step down limits on the WRF. **Table 4.3** summarizes the schedule of compliance with final effluent limits. This table is reprinted from Table 9 of NCRWQCB Order No. R1-2013-0042.

Table 4.3: Schedule of Compliance with Final Effluent Limitation for Total Phosphorus[1]

Task	Task Description	Due Date
1	The Permittee shall submit a detailed scope of work for conducting a treatment plant modernization and optimization study to evaluate treatment plant improvements that could result in various levels of phosphorus removal from treated effluent discharged to Mark West Creek	February 1, 2015 (completed)
2	The Permittee shall complete the treatment plant modernization and optimization study identified in Task 1 and submit a written report that includes under Task 2, above and process toward completion of those activities and tasks.	February 1, 2016 (completed)
5	The Permittee shall have completed all activities, programs, and/or approved projects resulting in the reduction and/or offset of at least 10% of Permittee's estimated mass discharge of total phosphorus for the 2018/2019 discharge season.	October 1, 2018
6	The Permittee shall have completed all activities, programs, and/or approved projects resulting in the reduction and/or offset of at least 33% of Permittee's estimated mass discharge of total phosphorus for the 2019/20 discharge season.	October 1, 2019
7	The Permittee shall have completed all activities, programs, and/or approved projects resulting in the reduction and/or offset of at least 66% of Permittee's estimated mass discharge of total phosphorus for the 2020/2021 discharge season.	October 1, 2020
8	The Permittee shall have completed all activities, programs, and/or approved projects resulting in the reduction and/or offset of at least 100% of Permittee's estimated mass discharge of total phosphorus for the impending discharge season.	October 1, 2021 and annually thereafter

[1] Items reprinted from Table 9 of NCRWQCB Order No. R1-2013-0042. For full table please refer to permit.

The percent reduction or offset will be measured against a benchmark initial year phosphorus concentration of 2.75 mg-P/L. This benchmark was provided in email correspondence dated May 17, 2016. To meet the step-down phosphorus load reduction requirements listed in **Table 4.3**, the Town is employing a three-prong approach comprised of effluent pond optimization (to reduce discharge volume), process changes at the WRF (to reduce phosphorus concentration) and purchasing nutrient offsets (since 0 mg P/L or 0 lbs/day cannot be technically achieved). This is discussed in detail in **Section 7**.

4.3 Regulatory Considerations

This section provides insight into the existing and future regulatory considerations that may impact the WRF's discharges, biosolids production and disposal, over the 20- to 30-year planning horizon. Because continued regulatory compliance is an objective of the Modernization Study, identifying future regulatory trends is critical for:

- developing and evaluating treatment scenarios and alternatives;
- planning for space requirements for future regulatory compliance; and
- making budget considerations for major design/construction projects.

In identifying future pollutants of concern (POCs), such as metals, nutrients, and/or pathogens, this study can be further developed to include alternatives that are flexible and can be easily expanded or upgraded to treat future POCs.

4.3.1 Nutrient Removal

The NCRWQCB has been developing a nutrient Total Maximum Daily Loading (TMDL) for Mark West Creek that is expected to result in requirements for reduction of discharges of phosphorus and nitrogen in WRF effluent.

In the current NPDES permit, the WRF is given interim stepdown effluent limits for phosphorus, but not for total nitrogen. The WRF is currently implementing phosphorus reduction to be in compliance with the permit requirements. As the nitrogen criteria for the North Coast are developed, the WRF may need to implement nitrogen removal strategies.

4.3.2 Toxicity

The State Water Resources Control Board (SWRCB) is developing an updated approach to assess toxicity in wastewater that is expected to emphasize chronic toxicity testing relative to the current permit's emphasis on acute toxicity testing. The NCRWQCB has taken the initiative for early implementation of this proposed policy. It is anticipated that these revised requirements may be included in the Town's next permit.

4.3.3 Pathogen Total Maximum Daily Load

The NCRWQCB is developing an indicator bacteria total maximum daily load (TMDL) for the Russian River Watershed. Indicator bacteria are used to assess the likelihood of pathogens in water. An analysis of the Russian River and its tributary creeks reflects widespread contamination with bacteria and other indicators of human waste. This violates the narrative Bacteria Water Quality object set by the NCRWQCB which states that water "shall not be degraded beyond natural background levels." The NCRWQCB is considering adopting a TMDL for *E. coli* and *Bacteriodes* bacteria or other indicators of human contamination which will be based on the EPA's 2012 Recreational Water Quality Criteria (Russian River Watershed Pathogen TMDL Action Plan). The Draft TMDL concludes that irrigation with, and direct discharges to surface water of recycled water that meet effluent limitations for bacteria are not a significant source of bacteria. However, the Draft TMDL concludes that study of stored recycled water needs further study as a source of pathogens:

> "Based on these studies reviewed by Regional Water Board staff, discharges of treated wastewater from recycled water holding ponds may contain *E. coli* and in concentrations above the TMDL targets. However, the studies indicate that the sources of detected *E. coli* bacteria in recycled water storage ponds are not necessarily of human origin and therefore may not pose a more significant threat to public health or be relevant to protection of the REC-1 beneficial use.

> More site-specific information is necessary to determine the sources of *E. coli* or other fecal indicator bacteria in recycled water storage ponds and whether the discharge from a recycled water storage pond contains human pathogens before the holding pond can be eliminated as a pathogen source."

To the extent discharge from ponds is indeed a source of human pathogens, the Draft TMDL indicates that WDRs would be updated within five years of TMDL adoption to include water quality-based effluent limitations for fecal indicator bacteria that will ensure compliance with waste load allocations for *E. coli* bacteria such that compliance with waste load allocations is achieved within ten years. The Town is considering conducting a study of pathogens in its storage ponds to evaluate if it may be subject to such requirements.

4.3.4 Contaminants of Emerging Concern

The SWRCB convened two expert panels to address the need for monitoring and regulation of contaminants of emerging concern (CECs) in recycled and discharged water. The panels recommended risk-based screening framework is to identify an initial list of CECs to monitor. The panel also recommended implementing an adaptive monitoring approach to assess the presence of the CECs and their potential risk to humans and the environment. This adaptive approach would allow for future review of, and if required, the ability to modify the strategy to incorporate the latest science available to the water resources community.

CECs detected in surface waters and the environment that may potentially cause deleterious effects on aquatic life and the environment at relevant concentrations include:

- Persistent organic pollutants (POPs) such as polybrominated diphenyl ethers (PBDEs; used in flame retardants, furniture foam, plastics, etc.) and other organic contaminants.
- Pharmaceuticals and personal care products (PPCPs), including a wide suite of human prescribed drugs, over-the-counter medications, caffeine, bactericides, sunscreens, and other related products.
- Veterinary medicines such as antimicrobials, antibiotics, anti-fungals, growth promoters and hormones.
- Endocrine-disrupting chemicals (EDCs), including synthetic estrogens and androgens, naturally occurring estrogens, as well as many other compounds capable of modulating normal hormonal functions and steroidal synthesis in aquatic organisms.
- Nanomaterials such as carbon nanotubes or nano-scale particulate titanium dioxide.

Monitoring requirements for these trace pollutants are expected to increase per the panels' recommendations. Water quality criteria could eventually be developed followed by new effluent limits in permits.

The WRF is considering options and alternatives that minimize sources of these pollutants and remove them from the influent wastewater through increased source control and pollution prevention programs, where practicable. However, many of these compounds will be difficult to control at the source.

4.3.5 Land Application and Beneficial Use/Disposal of Biosolids

Solids generated at a wastewater treatment plant are typically comprised of screenings, grit, primary or raw sludge (PS), secondary or waste activated sludge (WAS), and solids from the advanced wastewater treatment system (AWT). The screenings and grit are typically dewatered and disposed to a landfill. The PS and WAS are categorized as sewage sludge or wastewater solids prior to stabilization.

Sludge generated by a wastewater treatment plant is defined as biosolids once beneficial use criteria (determined by compliance with the EPA's Title 40 Code 503 regulations) have been achieved through stabilization processes. Stabilization processes are described as those that help reduce pathogens and reduce vector attraction. Biosolids are defined as treated organic solid residuals resulting from the treatment of municipal sewage at a wastewater treatment facility. Biosolids are a product with a high carbon content and other beneficial use properties.

Biosolids are classified by the EPA's 40 CFR 503 regulations as Class B or Class A, according to the level of pathogen reduction. The Windsor WRF typically generates Class B biosolids. Biosolids must also meet vector attraction and metal concentration limits. All biosolids must meet the Ceiling Concentration Limits for pollutants. Land applied biosolids must also meet either the pollutant concentration limits, cumulative pollutant loading rate limits, or annual pollutant loading rate limits. The Town typically land applies their biosolids. A contractor determines the location of land application. In 2017, the biosolids were transported to Solano County.

Several federal, state, and local regulations are in place that influence whether biosolids from municipal wastewater treatment plants can be beneficially used or disposed. Increased concern and debate over biosolids use/disposal and its associated environmental impacts have led to more stringent regulations. Changes in regulations affecting biosolids management are expected in the future and make a flexible management program essential.

The California Department of Resources Recycling and Recovery (CalRecycle) oversees and regulates California's solid waste disposal including co-disposal issues and biosolids used as a daily covering material. Recently enacted regulations, including Senate Bill 1383 (Sep. 2016), Assembly Bill 1594 (Sep. 2014), and Assembly Bill 341 (Oct. 2011), will make it more difficult for the Town to continue sending biosolids to landfill for use as alternative daily cover. This will place price and capacity pressures on existing biosolids markets, such as compost and land application, increasing competition among utilities for available biosolids outlets.

In addition, many counties in California have developed, or are developing, ordinances for biosolids land application. The stringency of these county regulations ranges from requirements for higher standards of permit review to the banning of biosolids land application. Land application restrictions for biosolids are shown by county in **Figure 4.1**. Four counties ban all land application and an additional six counties ban Class B land application. Of the remaining counties, twenty-seven require a conditional use permit. Additionally, some of these counties limit biosolids application to only biosolids produced in their county

or place limits on where biosolids can be spread (e.g., 100 feet from surface waters, etc.) and require a corporate surety bond. Only five counties specifically allow Class B biosolids land application: Sonoma, Solano, Sacramento, Glen, and Sierra Counties. Additional discussion of the Town's sludge and solids management can be found in **Section 11**.



[1] This map is based on a search of online available Codes and may have missed County or other requirements, such as non-codified requirements or requirements of City or regional agencies. This map presents findings for individual County requirements and deals only with the unincorporated areas of the Counties. All areas in California are subject to California and Federal biosolids regulations.

Figure 4.2: Biosolids Landfill Ordinances for Counties in California (2016)[1]

4.3.6 Biosolids Air Pollution Control

Biosolids processes can generate odors which may disperse naturally into the surrounding environment. The odors can be a nuisance to local residents. These odors may detrimentally affect property values and the quality of life in affected communities. They often result in complaints or even public opposition. Minimizing odors is in the best interest of both the Town and the community. Proper planning and facility design can minimize odors and reduce odor complaints. The Town's current biosolids stabilization process, sludge ponds, includes maintaining a water cover over the solids. This helps to abate odors.

The California Ambient Air Quality Standards (CAAQS) dictate a maximum 1-hour average for hydrogen sulfide. This is likely the most relevant CAAQS regulated air quality standard that would be applicable to biosolids processing. The hydrogen sulfide standard is listed in the **Table 4.4**, along with other CAAQSs.

Table 4.4: California Ambient Air Quality Standards

Pollutant	Concentration	Description
Carbon Monoxide	20 ppm	1-Hour Average
	9.0 ppm	8-Hour Average
Hydrogen Sulfide (H_2S)	0.03 ppm	1-Hour Average
Lead	1.5 µg/m^3	30-Day Average
Nitrogen Dioxide	0.18 ppm	1-Hour Average
	0.030 ppm	Annual Average
Ozone	0.09 ppm	1-Hour Average
	0.070 ppm	Annual Average
Particulate Matter – $PM_{2.5}$	12 µg/m^3	Annual Average
Particulate Matter – PM_{10}	50 µg/m^3	24-Hour Average
	20 µg/m^3	Annual Average
Sulfate	25 µg/m^3	24-Hour Average
Sulfur Dioxide	0.25 ppm	1-Hour Average
	0.04 ppm	24-Hour Average
Vinyl Chloride	0.01 ppm	24-Hour Average

4.3.7 Climate Change

Concern for climate change has been growing as a result of more frequent extreme weather events over the last few years. Climate change is due, in part, to the human use of fossil fuels, which releases carbon dioxide and other greenhouse gases into the air. As a result, these gases trap heat in the atmosphere which may have a range of effects on the ecosystem such as severe weather events and drought. From extreme drought to flooding, fires and landslides, California has seen firsthand the effects of climate change.

As the WRF considers future facilities and improvements, it will be considering alternatives that improve the sustainability and resiliency of the Town through continued efforts to adapt to climate change (2040 General Plan, Goal PHS-7). The Town strives to minimize greenhouse gas emissions and achieve a sustainable, resilient community. This can be accomplished by: promoting energy efficiency and

greenhouse gas reduction strategies; protecting and adapting public infrastructure, services, natural systems and resources from climate change impacts; and continuing to coordinate and communicate locally and regionally to monitor and address the changing needs and conditions of the community. The Town also strives for a climate resilient community through continued efforts to prepare for flooding and drought conditions.

The Town is planning to install floating photovoltaic panels to generate clean power for the WRF, Geysers Pump Station, and the Town Corporation Yard. The system is estimated to generate 1.7 MW of power which is approximately 95 percent of the annual electrical demands. The solar panel system will provide sustainable energy for these facilities and will assist the Town in becoming more sustainable.

5. Existing Facilities

The purpose of this section is to describe the key unit processes at the WRF and to summarize the condition of each process. The key unit processes are described in **Section 5.2** and general condition assessments are provided in **Section 5.3**. The capacity assessments of key unit processes are described in **Section 6.**

5.1 Background

The key unit processes at the WRF include:

- Influent Pump Station
- High Flow Equalization Ponds
- Headworks - influent screening, grit removal, and Parshall flume
- Biolac® Extended Aeration Basin System and Blower Building
- Secondary Clarifiers
- Return Activated Sludge (RAS)
- Waste Activated Sludge (WAS)
- Advanced Wastewater Treatment (AWT) System
    - Chemical Addition and Flocculation Tanks
    - Tertiary clarification
    - Sand Filtration
- Ultraviolet (UV) Disinfection
- Effluent Pump Station
- Effluent Storage Ponds (Ponds S2 and Ponds 5, and 7 to 10)
- Geysers Pump Station

Figure 5.1 and Figure 5.2 shows the site plan and notes the various ponds located on the site. **Figure 5.3** provides a schematic of the flow plan. The High Flow Equalization Ponds 1 and 2 are utilized when flows to the WRF exceed 4 to 5 MGD. Diverted flow is pumped from Pond 2 to the Biolac® basins when flows to the facility recede and there is capacity at the WRF to treat diverted flow. Ponds S2, 5 and 7, 8, 9 and 10 provide storage for treated effluent that may be used as recycled water or discharged per the limitations in the NPDES permit. Pond S2 was originally built to hold sludge but has been repurposed as an effluent storage pond. It may be converted back to a sludge pond in the future.



Figure 5.1: Windsor WRF Site Plan

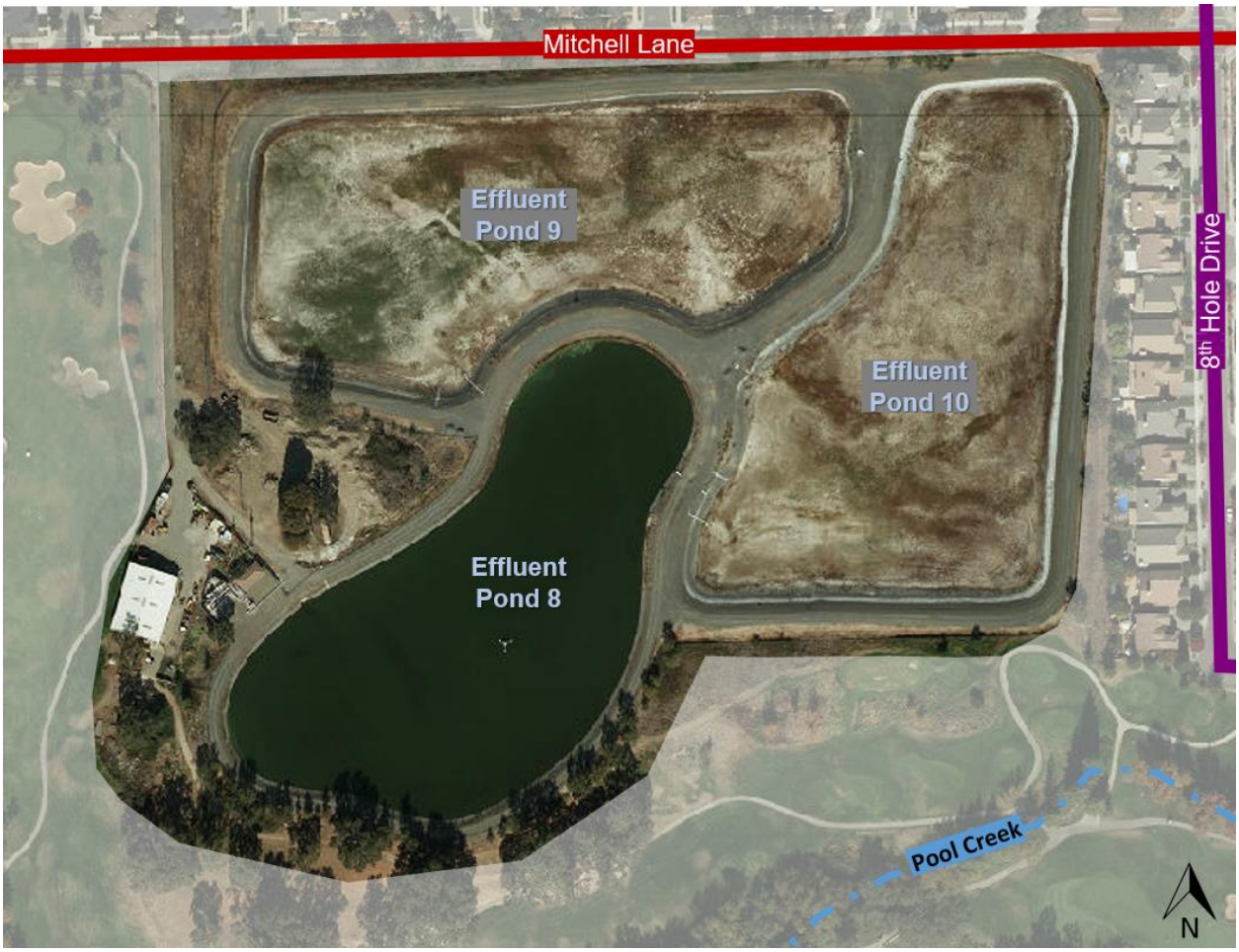


Figure 5.2: Location of Additional Effluent Ponds

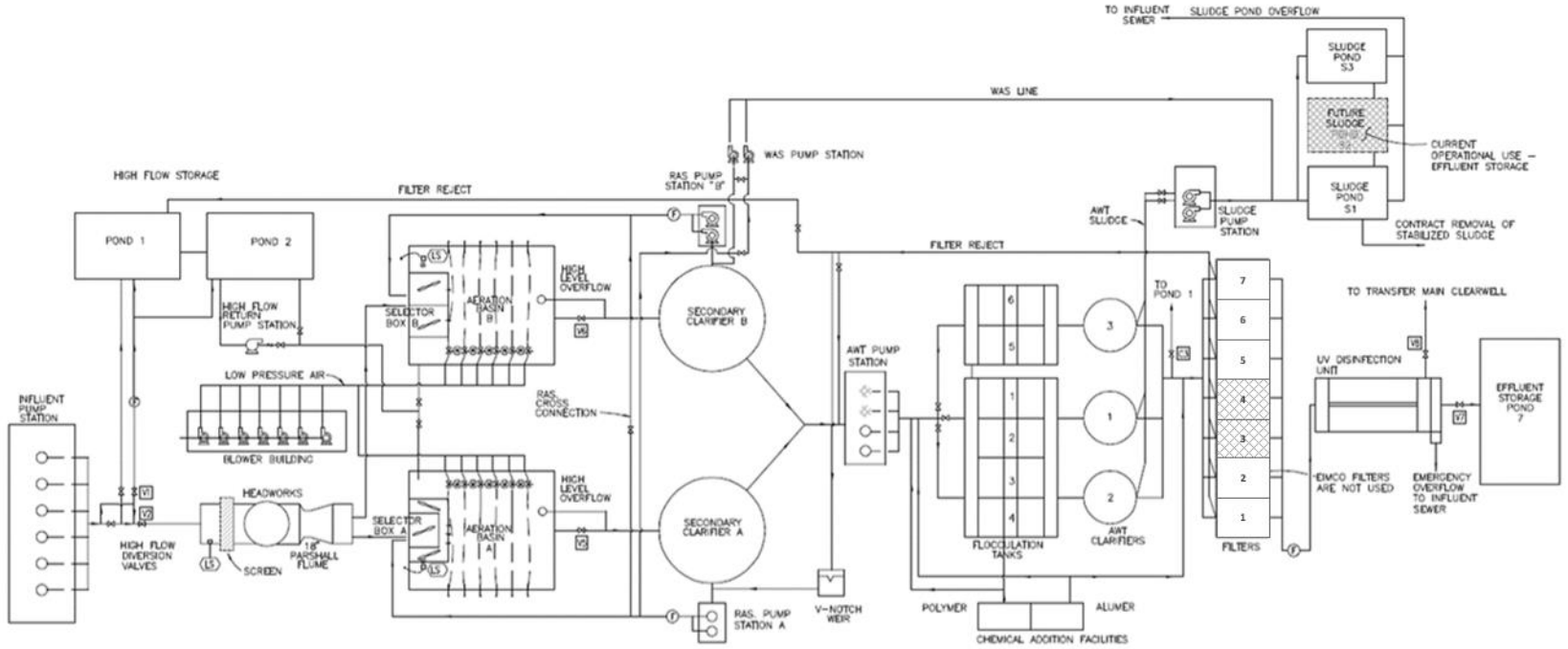


Figure 5.3: Windsor WRF Process Flow Diagram[1]

[1]From Figure 2 of the March 2012 Engineering Report (Brelje & Race)

5.2 Existing Unit Processes

The first sewage treatment facilities developed by the Town were constructed in 1964. The WRF has undergone numerous improvements since its first construction. The construction year of the key unit processes are provided in **Table 5.1.**

Table 5.1: Construction Dates of WRF Facilities

Process	Year Constructed or Installed
Influent Pump Station	1999
High Flow Equalization Ponds	Pond 1 – 1970 Pond 2 - 1964
Headworks	1996
Biolac® Aeration Basins	Basin A – 1996 Basin B - 2000
Blower Building / Blowers	1996 (modified piping in 2011)
Secondary Clarifiers	Clarifier A – 1996 Clarifier B - 2000
RAS Pump Stations	Station A – 1996 Station B - 2000
WAS Pump Station	2000
AWT Pump Station	1991 (2 pumps upgraded in 1993)
AWT Flocculation Tanks	Tanks 1 & 2 – 1991 Tanks 3 & 4 - 1994
AWT Clarifiers	Clarifier 1 – 1991 Clarifier 2 – 1994 Clarifier 3 - 2001
AWT Sludge Pumps	1991
AWT Filters	Filters 1 & 2 – 1991 Filters 3 & 4 – 1994 Filters 5, 6 & 7 - 2000
UV Disinfection	2001
Effluent Pump Station	1992
Effluent Storage Ponds	Pond S2 – 1971 Pond 5 - 1971 Pond 7 - 1971
Geysers Pump Station	2011

The permitted ADWF of the WRF is 1.9 MGD, the ADWF hydraulic capacity is 2.25 MGD, and the permitted peak flow is 7.2 MGD. The following sections detail the unit processes and design criteria of the current major unit processes at the WRF. The capacities stated in the various unit processes are design criteria and may not reflect the current performance and operation of the unit processes. The performance of each treatment process may affect overall WRF performance.

5.2.1 Influent Pump Station

The influent pump station is configured in a wet well/dry well configuration with six high solids self-priming Gorman-Rupp® Model T10A-B-3 centrifugal pumps that were installed in 1999. The interior

floor of the dry well rests approximately 8 feet above the bottom of the wet well. The 10-inch diameter pump intakes are set approximately 6-inches above the bottom of the dry well and are not protected by a screen. Overall, each pump intake rests approximately 10.75 feet below the centerline of the pump volume intake. Each pump has an approximate capacity of 2,600 gallons per minute (gpm). The pumps are controlled by variable frequency drives. The influent pump station is capable of pumping 14.8 MGD of wet weather flow, while retaining a redundant pump. A summary of the influent pump station design criteria is shown in **Table 5.2**.

Table 5.2: Influent Pump Station Design Criteria

Parameter	Design Criteria
Number of Pumps	6
Design Capacity (each)	2,600 gpm
Wet Well Operational Volume[1]	11,000 gal
Total Dynamic Head	35 ft
Total Dynamic Suction Lift	10 ft
Maximum Re-priming Lift	10 ft
Pump Speed	1,050 rpm
Motor Horsepower	40 HP
Motor Voltage	480 Volt, 3 Phase, 60 Hz
Motor Speed	1,800 rpm
Influent Pump Station Firm Capacity (1 pump offline)	14.8 MGD

[1] *Based on wet well length and width and the typical operational depths, which range from 6 ft (High High) to 1.5 ft (Low Low).*

5.2.2 High Flow Equalization Ponds

When the influent flow is above the capacity of the treatment units or in the case of a power failure, raw wastewater flow is diverted to the high flow equalization ponds 1 and 2 via the diversion valve V1. Currently, high flow diversion is used for influent flows greater than 5.0 MGD. Water is stored in the high flow equalization ponds 1 and 2 until it can be fed back into the treatment system. Pond 1 was installed in 1970 and Pond 2 was installed in 1964. The capacities of the high flow equalization ponds are shown in **Table 5.3**.

Table 5.3: High Flow Equalization Ponds Design Criteria

Parameter	Pond 1	Pond 2
Max. Operating Water Depth (ft)	8.0	10.0
Minimum Freeboard (ft)	2.0	2.0
Normal Storage Capacity (MG)	8.0	9.9

5.2.3 Headworks

The headworks complex includes a multi-rake mechanical bar screen, a grit chamber, grit classifier and grit conveyer, a Parshall flume, and a vertical flow splitter. These facilities were installed in 1996. The

influent mechanical bar screen is a Parkson™ Corporation Aqua Guard® Screen with a 6 millimeter (mm) spacing and a peak flow capacity of 9.6 MGD. Immediately downstream of the mechanical bar screens is the Smith & Loveless® grit cyclone unit (refurbished in 2015), classifier and screw conveyor. The WRF influent flow is measured downstream of the grit removal system via Parshall flume. Downstream of the flume, a vertical flow splitter is designed to equally split flow to aeration basins A and B; however, the true proportions of flow allocation are not measured. A summary of key design criteria is provided in **Table 5.4**.

Table 5.4: Headworks Design Criteria

Parameter	Design Criteria
Filter Screen Channel Width	2.5 ft
Filter Screen Openings	6 mm
Grit Chamber Main and Lower Chamber Diameter	11 and 5 ft, respectively
Grit Chamber Main and Lower Chamber Depth	5.67 and 6.67 ft, respectively
Parshall Flume Width	1.5 ft
Parshall Flume Water Depth	0.5 – 2 ft
Headworks Flow Capacity	9.6 MGD

5.2.4 Biolac® Extended Aeration Basin System and Blower Building

The WRF has two aeration basins that are designed to remove BOD and nitrogen in the influent wastewater. Aeration Basin A was installed in 1996 and Basin B was installed in 2000. Each basin is 3.0 MG in volume and was designed to treat 1.5 MGD of wastewater containing approximately 260 mg/L BOD_5, 140 mg/L TSS and 40 mg/L Total Kjeldahl Nitrogen (TKN). The design peak flow of each basin is 4.8 MGD.

Although Basin A is serviceable, the WRF is currently operating using only one basin (Basin B) due to dissolved oxygen (DO) control issues and because the selector box in Basin B is newer and in better condition. A mixed liquor suspended solids concentration in the range of 2,600-3,500 mg/L is typically maintained in the basins.

The Biolac® system uses aerobic, anoxic, and anaerobic microbial processes utilizing a unique wave-ox system. Each basin has 10 floating air headers placed perpendicular to the flow direction. Each air header has 17 diffuser heads (swings) with 5 diffusers (individual membrane units) per swing. The programming associated with the Biolac® system cycles the air to the basins on and off at different times. This process is intended to create temporary oxic and anoxic zones to enhance nitrification and denitrification. There is an anoxic selector at the inlet of each aeration basin to encourage growth of desirable microorganisms in the basin. **Table 5.5** shows the design criteria for the Biolac® process.

Table 5.5: Biolac® Process Design Criteria

Parameter	Design Criteria
Selector Box A Volume	0.12 MG
Selector Box B Volume	0.09 MG
Design ADWF (combined)	3.0 MGD
Peak Flow (combined)	9.6 MGD
BOD	260 mg/L [2]
TSS	140 mg/L
NH_3-N	30 mg/L
TKN	40 mg/L
Number of Basins	2
Volume per Basin	3 MG
Length, average	220 ft
Width, average	172 ft
Basin Depth	17.75 ft
Operating Depth	14.25 – 16.75 ft
Number of Chains per Basin	10
Diffuser Float Assemblies per Chain	17
Diffusers per Assembly	5
Total Number of Diffusers per Basin	850

[1] The Selector Box A Volume is the volume at its initial design and construction. The selector box has since degraded, and the curtain is not fully intact.
[2] The BOD design criteria taken from the Phase IIA Stage 2 Expansion drawings.

5.2.4.1 Blower Building

The extended air process is supported by seven positive displacement blowers, air distribution piping, and air valving coupled with submerged fine bubble diffusers. There are low flow and high flow blowers. The original blower building was constructed in 1996 with improvements to the air piping in 2011. The Town recently added the ability to cycle blowers on individually. The design criteria for the blowers is summarized in **Table 5.6.**

Table 5.6: Blower Design Criteria

Quantity	Type	Horsepower (HP)	Design Discharge Pressure (psig)	Design Airflow (scfm)
4	Positive Displacement	50	7.9	968
3	Positive Displacement	75	7.9	1,291
			Total Firm Capacity[1]	**6,454**
			Total Capacity	**7,745**

[1]Firm capacity assumes one large blower out of service

Currently the blowers feed both aeration basins through a common manifold; therefore, there is no specific control of air flow to either individual basin.

5.2.5 Secondary Clarifiers

The activated sludge flows by gravity from the aeration basins to the two circular secondary clarifiers where the microorganisms settle out. Clarifier A was constructed in 1996 and Clarifier B was constructed in 2000. Each aeration basin was designed with a dedicated clarifier; however, either clarifier can be used under operation with a single aeration basin. The two circular clarifiers are constructed of concrete with a water depth of 16 feet, a diameter of 87 feet and a volume of 710,000 gallons. Influent enters the center flocculation well of the clarifier, where energy dissipation baffles slow the flow out to the weir. RAS is collected at the bottom rotating skimmer by vertical PVC draft tubes that bring water up via differential fluid levels between the RAS outflow box and the general (slightly higher) water level in the clarifier. Each clarifier is designed for a peak overflow rate of 800 gpd/ft^2 at 4.8 MGD loading. The secondary clarifier design criteria are presented in **Table 5.7.**

Table 5.7: Secondary Clarifiers Design Criteria

Parameter	Design Criteria
Number of Clarifiers	2
Max. Design Flow (combined)	9.6 MGD
Diameter	87 ft
Surface Area (each)	5,945 ft^2
Side Wall Water Depth	16 ft
Volume (each)	0.71 MG
Max. Overflow Rate	800 gpd/ft^2

Though the maximum design flow is 9.6 MGD, flow is kept below 3.0 MGD per clarifier due to sludge settleability issues. The sludge settleability issues are further discussed in **Section 6.**

5.2.6 Return Activated Sludge (RAS)

In the secondary clarifier, solids that have settled to the bottom are either returned to the aeration basin (RAS) or wasted to sludge ponds (WAS). Solids are returned to the aeration basin via two RAS pump stations located next to each of the clarifiers. RAS Pump Station A was constructed in 1996 and RAS Pump Station B was constructed in 2000. Each RAS pump station has 2 pumps. The total capacity of each pump station is approximately 2,400 gpm. RAS flow is metered using conduction meters; however, operations staff have noted that air entrainment is a frequently encountered problem when measuring and controlling the RAS flow rates. RAS pump station design criteria is shown in **Table 5.8.**

Table 5.8: RAS Design Criteria

Parameter	Pump Station A	Pump Station B
Number of Pumps per Station	2	2
Total Dynamic Head (ft)	17	29
Pump Speed (rpm)	850 [1]	850 [1]
Motor Horsepower (HP)	15	15
Motor Voltage	230/460 Volts, 3 Phase, 60 Hz	230/460 Volts, 3 Phase, 60 Hz
Motor Speed (rpm)	1775	1775/1180
RAS Capacity (MGD)	2.6	3.5

[1] Pumps A1 and B1 are variable speed. The other two pumps (A2 and B2) are constant speed.

5.2.7 Waste Activated Sludge (WAS)

Periodic wasting of solids from the secondary clarifiers to the sludge ponds is performed by the waste activated sludge (WAS) pump station. The current WAS pump station, which was constructed in 2000, is located adjacent to the RAS pump station at Secondary Clarifier B. The WAS pump station consists of two self-priming centrifugal wastewater pumps, each with a capacity of 200 gpm. The WAS pumps are controlled by a PLC local to the pump station. The WAS pumps are single speed. WAS pump station design criteria are shown in **Table 5.9.**

Table 5.9: WAS Design Criteria

Parameter	Design Criteria
Number of Pumps	2
Capacity per Pump	200 gpm
Total Dynamic Head	31 ft
Pump Speed	410 rpm
Motor Horsepower	20 HP
Motor Voltage	230/460 Volts, 3 Phase, 60 Hz
Motor Speed	1750 rpm
WAS Pump Station Firm Capacity	200 gpm
WAS Pump Station Capacity	0.58 MGD

5.2.8 Advanced Wastewater Treatment (AWT) System

The Advanced Wastewater Treatment (AWT) system includes flocculation, settling, and filtration processes for further solids removal from the wastewater. The AWT system includes a pump station with four pumps, six 2-stage flocculation tanks, three circular clarifiers, and five operational continuous backwash filters. In total the AWT system has seven continuous backwash filters, however the two Eimco Rotoco filters are non-operational and non-serviceable. Chemical flocculants and coagulants (“Alumer” and polymer) may be added prior to the flocculation tanks. “Alumer” can also be added at the inlet of the flocculation tanks and to the effluent of the AWT clarifiers. Polymer can also be added prior to the second stage of the flocculation tanks. Typically, approximately 60 ppm of “Alumer” is added in the pipeline prior to the flocculation tanks and 1.3 part per million (ppm) of polymer is added to the second stage of the flocculation tanks.

5.2.8.1 AWT Pump Station

The AWT pump station consists of two vertical turbine pumps and two centrifugal pumps. The centrifugal pumps have a capacity of 1,800 gpm and the vertical turbine pumps have capacities of 1,300 and 1,800 gpm. The pumps lift the effluent from the secondary clarifiers to the elevation required to flow by gravity through the AWT and disinfection units to the storage ponds. The original pump station was constructed in 1991 with improvements in 1993 when two centrifugal pumps were added. The pumping capacity is stated in **Table 5.10.**

Table 5.10: AWT Pumping Design Criteria

Parameter	Design Criteria
Pumps 1 and 2 (centrifugal) Capacity (each)	1,800 gpm
Pump 3 (vertical turbine) Capacity	1,800 gpm
Pump 4 (vertical turbine) Capacity	1,300 gpm
Total Dynamic Head	40 ft
Pump 1 and 2 Speed	1050 rpm
Pump 3 and 4 Speed	1760 rpm
Motor Horsepower	30 HP
Motor Voltage	480 Volts, 3 Phase, 60 Hz
Motor Speed	1760 rpm
AWT Firm Capacity (1 Pump Out of Service)	7.1 MGD
AWT Pumping Capacity	9.6 MGD

5.2.8.2 AWT Flocculation Tanks

Six 25,000-gallon concrete flocculation tanks operate in parallel to provide a minimum of 30 minutes of contact time for the wastewater to react with the chemical flocculant and polymer. Each tank consists of two stages that are separated by a redwood baffle wall. The first stage has a 1-horsepower mixer with a pitched blade turbine impeller. The second stage has a 1-horsepower mixer with a hydrofoil impeller. Dosing of alumer and polymer is adjusted depending on the quality of the effluent coming from the secondary clarifiers. Each of the six flocculation tanks has a treatment capacity of 1.2 MGD. Flocculation tanks 1 and 2 were constructed in 1991, flocculation tanks 3 and 4 were constructed in 1994, and flocculation tanks 5 and 6 were constructed in 2001. Design criteria for the AWT flocculation facility are summarized in **Table 5.11.**

Table 5.11: AWT Flocculation Design Criteria

Parameter	Design Criteria
Flocculation Tanks	6
Flocculation Stages per Tank	2
Flocculation Volume per Stage	12,500 gal
Flocculation Total Volume	0.15 MG
Flocculation Current Average HRT	51 min
Flocculation HRT @ maximum flow	22 min

5.2.8.3 AWT Clarifiers

After the flocculation tanks, flow enters three above-ground circular, steel AWT clarifiers. AWT Clarifiers 1, 2 and 3 were constructed in 1991, 1994 and 2001, respectively. Each clarifier is 55-foot diameter with an operating water depth of 13.5 feet and a volume of 240,000 gallons. Each clarifier has a peak capacity of 2.4 MGD and was designed for a peak overflow rate of approximately 1,000 gpd/ft^2. The solids that settle out are removed from the bottom of the clarifier and pumped to sludge ponds by the AWT sludge pump station. AWT Clarifier Design Criteria is provided in **Table 5.12.**

Table 5.12: AWT Clarifiers Design Criteria

Parameter	Design Criteria
Number	3
Total Clarifier Design Flow (combined)	7.2 MGD
AWT Clarifier Diameter	55 ft
AWT Clarifier Surface Area (each)	2,376 ft^2
AWT Clarifier Volume (each)	240,000 gal
Max. Design Surface Settling Rate	1,000 gpd/ft^2
Min. Detention Time	2.4 hrs.

5.2.8.4 AWT Sand Filters

Effluent from the AWT clarifiers flows to five Parkson™ Dynasand® continuous backwash sand filters. Each unit has 200 square feet of surface area and is operated at a maximum loading rate of 5 gpm/ft^2. Two Eimco Rotoco filters were installed in 1994, however they are no longer operable or serviceable. The Parkson™ Dynasand® total filter treatment capacity is approximately 7.2 MGD. Key filter design criteria are listed in **Table 5.13.**

Table 5.13: AWT Sand Filters Design Criteria

Parameter	Design Criteria	
	Parkson™ Dynasand®	Eimco Rotoco[1]
Number	5	2
Surface Area (ft^2)	200	200
Maximum Loading Rate (gpm/ft^2)	5	3
Maximum Design Flow per Filter (MGD)	1.44	0.86
Total Capacity (MGD)	7.2	1.72

[1]*Eimco Rotoco filters are not operable or serviceable.*

5.2.9 UV Disinfection

The WRF uses an Ultraviolet (UV) light system as the primary means of disinfection. The UV equipment is an Aquaray® 40 system manufactured by Ondeo Degremont (now distributed by SUEZ) and was installed in 2001. Its primary hydraulic elements are an influent control box, two parallel channels with UV lamps, and two effluent control boxes. Each channel contains ten UV banks (each consisting of two co-operated UV modules). Each bank is comprised of five rows of sixteen vertically-oriented low-

pressure low-output lamps. Thus, there are a total of 1,600 lamps in the system. The interior surfaces of the UV channels were recently resurfaced to reduce potential for coliform violations and each channel is currently cleaned on a bi-monthly basis (during downtime). The UV lamps and modules are cleaned monthly. The SCADA /PLC was upgraded during a recent SCADA/PLC project. **Table 5.15** presents the design criteria for the UV system.

Table 5.14: UV Channel Design Criteria

Parameter	Design Criteria
Number of Channels	2
Flow Capacity (each)	5.1 MGD [1]
Total Flow Capacity (both)	10.1 MGD
UV Influent Transmittance Design	70%

[1] Note that this number is rounded.

5.2.10 Effluent Pump Station

The effluent pump station discharges treated WRF effluent, primarily from Pond 7 and the clear well, to the irrigation/discharge transmission main. Disinfected effluent is pumped for irrigation use or it is pumped to Mark West Creek during the permitted discharge season. There are four electric motor driven vertical turbine effluent pumps that were installed in 1992: two 50 HP, one 100 HP and one 200 HP. The pumps are generally controlled by the level in a 64,250 gallon gravity surge tank located at a high point in the distribution network off-site. In addition to the pumps, there is a 5,000 gallon horizontal surge arrestor tank at the pump station on site. **Table 5.15** presents the design criteria for the effluent pump station.

Table 5.15: Effluent Pumping Design Criteria

Parameter	Design Criteria
Capacity of Pumps 1 and 2	500 gpm
Capacity of Pump 3	1,100 gpm
Capacity of Pump 4	2,500 gpm
Total Dynamic Head	225 ft
Pumps 1 and 2 Motor Horsepower	50 HP
Pump 3 Motor Horsepower	100 HP
Pump 4 Motor Horsepower	200 HP
Motor Voltage	480 Volts, 3 Phase, 60 Hz
Motor Speed	1760 rpm
Total Effluent Pumping Capacity	7.98 MGD

5.2.11 Effluent Storage Ponds

The Town currently stores disinfected tertiary effluent in open ponds constructed of earthen material. The ponds are used solely for effluent storage. The disinfected tertiary effluent is held in storage ponds S2, 5, 7, 8, 9, and 10 prior to reclamation and/or disposal. Ponds S2, 5 and 7 are on-site (**Figure 5.1**). Ponds 8, 9, and 10 are off site near Mitchell Lane (**Figure 5.2**) and are primarily used to irrigate the golf course. Ponds S2, 5, and 7 are connected to the transfer pump station and can be equalized via gravity through the

connecting pipes. Ponds 8, 9, and 10 are also interconnected to each other; however, Pond 8 serves as the main control and feed pond from the WRF. The capacities of the effluent storage ponds are shown in **Table 5.16**.

Table 5.16: Storage Ponds Design Criteria

Parameter	Pond S2	Pond 5	Pond 7	Pond 8	Pond 9	Pond 10
Max. Operating Water Depth (ft)	12.0	12.0	14.0	10.0	10.3	10.3
Normal Storage Capacity (MG)	6.9	14.4	78.5	12.0	15.1	16.8
Minimum Freeboard (ft)	2.0	2.0	3.0	2.0	2.0	2.0

5.2.12 Geysers Pump Station

A portion of the Town's treated effluent is pumped to the Geysers Steamfield via the Geysers Pump Station and then conveyed via the Geysers Pipeline. The pump station is located on the WRF site. The Geysers Pump Station is owned by the Town but is exclusively operated by the City of Santa Rosa for the Geysers Steam Generation Project.

Table 5.17: Geysers Pump Station Capacity

Parameter	Design Criteria	Current Operation
Number of Pumps	4 [1]	-
Maximum Design Flow (per pump)	0.56 MGD [2]	-
Pump Station Firm Capacity	1.67 MGD	Max. 0.7 MGD

[1] The design documents list 3 duty pumps and 1 standby pump.
[2] This number is rounded.

The WRF currently delivers 0.2 MGD to the Geysers via the Geysers pump station and pipeline during non-discharge months and 0.7 MGD during the discharge season, for an annual average flow of 0.53 MGD.

5.3 Equipment Condition Assessment

A visual assessment of the physical condition and remaining useful life of the existing mechanical equipment was performed as part of this Modernization Study and is provided in as an **Appendix B**. This mechanical assessment also provided the opportunity to identify operating deficiencies and potential modifications or enhancements necessary to optimize operations. The results of the assessment are used to identify equipment and processes that may need to be modified or rehabilitated. Equipment assessment forms are included in the **Appendix B**. Additionally, in 2013 Brelje and Race developed a report titled "Capital Replacement and Maintenance Program" which provided a relatively current assessment of certain processes. This is included as an **Appendix C**. It should be noted that since the 2013 report, the grit chamber has been refurbished and several other recommendations from the report are planned or in design.

The general findings of the condition assessment are that while much of the WRF unit processes and equipment are nearing the end of their useful life and will be considered for replacement, they have been well maintained and operated. A summary of the major findings by process area are shown in **Table 5.18**.

The following is a list of major deficiencies that were determined as a result of the evaluation of process needs and observations during site visits. Discussion with WRF staff also helped identify deficiencies and operational difficulties.

Primary Treatment Processes

- The influent pump station does not yet have a bar screen. A Duperon® mechanical screen is to be installed in the spring or early summer of 2019.
- There is no built-in way to bypass flow around the existing headworks.
- The headworks grit classifier is heavily corroded and needs immediate replacement. This recommendation remains unresolved from the 2013 Brelje & Race report. A replacement grit classifier is planned for installation during the summer of 2019.
- The Parkson™ screen should have a washer installed. The screen itself has surpassed its service life; however, a replacement screen is not planned until the performance of the influent Duperon® screen (currently under design) are known. In addition, in its current position, the screen cannot be removed easily without also removing the level sensor. This makes servicing the screen more difficult.

Secondary Treatment Processes

- Influent flow to the aeration basins from the headworks is uneven and requires a flow splitting structure to better balance the flows if both aeration basins are used with the existing blower manifold piping configuration. The intent under the current configuration is to operate both basins as close as possible as one biological unit. This is difficult without balanced flow.
- The selector box curtain in Aeration Basin A needs refurbishing and should be resized at that time to optimize both nitrogen and phosphorus removal.
- Selector box B, while in better condition and better sized for nitrogen removal, is not optimally-sized for both nitrogen and phosphorus removal.
- Voltage imbalances and an aged MCC have caused electrical feed issues which have resulted in regular blower motor failures. The blower capacity is limited to 5 blowers running to avoid electrical service issues.
- Currently there is no way to monitor blower air flow rates or electrical usage. Thus, it is difficult to ascertain how the factory blower efficiencies relate to the actual blower efficiencies. The Town is in process of installing air meters on each aeration basin header and has plans to install permanent electrical metering of the blower building.

- There is no way to automatically control the amount of air to each basin. Controlling the amount of air to the individual basins is key if influent flow balancing between the two basins is not achieved. Otherwise, the over-loaded (higher flow) basin is frequently starved of air and the under-loaded basin is in aerobic excess.

- It is difficult to access or remove the blowers and blower motors from the blower building.

- The diffusers in Basin A are old and should be replaced before the basin is placed back in service (Basin B diffusers were replaced in 2017).

- The diffusers at the edges of Basin B are slightly raised causing uneven air distribution within the basin.

- Despite a greater design capacity, the secondary clarifiers seem to be operationally limited to 3 MGD capacity based on sludge settling characteristics (see **Section 6** for more detail).

Tertiary Treatment Processes

- Alum, added via the chemical feed pumps, is not thoroughly mixed before entering the flocculation tanks. A static mixer should be added between the chemical addition and the inlet to the flocculation tanks.

- The flocculation tank-mixer gear boxes are aged and failing. The Town is aware of this and is currently replacing them at a rate of about 2 per year.

Table 5.18: Summary of Equipment Condition Assessment Findings

Process Area	Installation Year	Remaining Useful Life[1]	Condition[2]	Comments	Recommended Action
Influent Pump Station	1999	10	1	• Daily clogging is an O&M problem, but pumps operate efficiently. • No upstream screening is present.	• Replacement is needed in the next 10 - 20 years. In the short term add screening system to increase longevity of pumps. • Inspect and survey MCC for hot spots.
Headworks	1996	1-5	3	• Screens and flume operate well, but the grit classifier system is severely corroded and no longer efficient. The grit vortex unit was replaced in December 2015.	• Replace the grit classifier in the next 1 - 5 years. • Rehab or replace the screen mechanical equipment in the next 5 years. • Inspect and survey MCC for hot spots.
High Flow Storage Pond 1	1970	10	2	• Some concrete berm erosion and cracks were noted.	• Repair the concrete berm in 10 -20 years.
High Flow Storage Pond 2	1964	10	2	• Some berm erosion and pipe corrosion were noted. Northern berm was repaired in 2013. • There are no permanent facilities to pump all the raw water from High Flow Storage Pond 2 to High Flow Storage Pond 1.	• Repair the berm and replace corroded pipes in 10 - 20 years. • Add a permanent pump station to High Flow Storage Pond 1.
Aeration Basin A	1996	10	2	• Air piping valve corrosion • Selector curtain wall has collapsed. Diffusers were last changed in 2000. The selector box is not properly sized for nutrient reduction (the curtains were added to increase the size after the original construction).	• Repair corroded valves as they fail. • Replace the diffusers in the next 1 – 5 years. • Rebuild the selector box and resize for nitrogen and phosphorus reduction in 1 – 5 years.
Aeration Basin B	2000	10	2	• Air piping valve corrosion. Diffusers were replaced in 2017.	• Repair corroded valves as they fail.

Process Area	Installation Year	Remaining Useful Life[1]	Condition[2]	Comments	Recommended Action
Blower Building/ Blowers	1996 (modified piping in 2011)	5	Blower condition varies from 2 - 4	• Blowers are in reasonable condition and operate well. Building has limited access and poor ventilation. • No flow or electrical metering is available for the blowers. • Electrical limitations due to imbalance in power feed that results in overheating. MCC and electrical breakers are outdated.	• Add flow meters to the exiting blowers in 1 – 5 years. • Upgrade the power feed system in 1 – 5 years. • Replace or rehab Blower Building and blowers in 5 – 10 years. • Include fine tune control and VFDs on future blowers with more sophisticated DO control strategy. • Inspect and survey MCC for hot spots.
Secondary Clarifier A	1996	5	3 - 4	• Extensive corrosion in center well, top rim, and major effluent weir, mechanisms and gear, walkways and handrails. Structural integrity of center well to be evaluated.	• Replace mechanical equipment in 1 – 5 years and rehab structure.
Secondary Clarifier B	2000	10	2	• Corrosion in center well, top rim, effluent weir, mechanisms and gear,	• Replace mechanical equipment in 5 – 10 years and rehab structure.
RAS Pump Station A	1996	5	2	• Minor corrosion, internal wear, and meter inaccuracy due to and air entrainment in the pipelines.	• Replace in 5 – 10 years. • Replace air release valves.
RAS Pump Station B	2000	10	2	• Minor corrosion, internal wear, and meter inaccuracy due to and air entrainment in the pipelines.	• Replace in 10 – 15 years. • Replace air release valves.
WAS Pump Station	2000	10	2	• Minor corrosion. Batch wasting as opposed to continuous wasting.	• Consider replacing with a different type pump (diaphragm) in 10 – 20 years.
AWT Pump Station	1991 (modified in 1993)	5	2	• Some corrosion on the pump casings. Currently replacing two pumps.	• Replace in 1 – 5 years. New vertical turbine pumps already being planned for installation in CIP.
Flocculation Tanks 1&2	1991	1 - 5	3	• Concrete defects and gear boxes need replacement.	• Rehab structure and replace mechanical equipment in 1- 5 years.

Process Area	Installation Year	Remaining Useful Life[1]	Condition[2]	Comments	Recommended Action
Flocculation Tanks 3&4	1994	5	2	• Concrete defects and gear boxes are in need of repair/replacement.	• Rehab structure and replace mechanical equipment in 5 – 10 years.
Flocculation Tanks 5&6	2001	10	2	• Concrete defects and gear boxes are in need of repair/replacement.	• Rehab structure and replace mechanical equipment in 5 – 10 years.
AWT Clarifier 1	1991	1 -5	3	• Major corrosion and defects were noted.	• Replace equipment in 1 – 5 years and rehab structure.
AWT Clarifier 2	1994	5	3	• Major corrosion and defects were noted.	• Replace equipment in 5 – 10 years and rehab structure.
AWT Clarifier 3	2001	10	2	• Minor corrosion and defects were noted.	• Replace equipment in 5 – 10 years and rehab structure.
AWT Sludge Pumps	1994	1 – 5	2	• Only one pump works.	• Rehab the entire structure and replace or repair the pneumatic valves in 5 – 10 years.
Filters 1 & 2	1991	1 – 5	3	• Structural defects and corrosion to mechanical equipment were noted.	• Rehab or replace equipment in 1 – 5 years and rehab structure.
Filters 3 & 4	1994	0	2	• Permanently out of service.	• Remove and replace these filters in 1 – 5 years as needed.
Filters 5,6 & 7	2000	10	2	• Minor defects and corrosion were noted.	• Rehab or replace equipment in 5 – 10 years and rehab structure.
UV System	2001	5	2	• Lamps replaced each year. • Manufacturer proprietary PLC system is outdated and needs replacement.	• Replace equipment in 5 – 10 years.
Alumer System	1991 (refurbished in 2018)	1-5	2	• Pumps are new. System needs a static mixer for more efficient chemical mixing. • Some tank corrosion.	• Replace tank and add a static mixer in 1 – 5 years. • Replace pumps in 10 – 15 years.
Polymer System	1991	1-5	2	• Minor defects and corrosion were noted.	• Replace pumps in 1 – 5 years.
Hypochlorite System	2001 (Replaced in 2017)	1-5	3	• This equipment is new.	• Replace pumps in 10 – 15 years.
Sludge Ponds	1971 (modified in 2001)	15	2	• Some slope erosion was noted. • Some aerators are in poor conditions.	• Replace the mixer equipment in 5 – 10 years. • Rehab ponds slopes and grading in 10 – 15 years.

Process Area	Installation Year	Remaining Useful Life[1]	Condition[2]	Comments	Recommended Action
Effluent Pump Station	1992	5 - 10	2	• Minor defects and corrosion were noted. • Motors have no soft starts or VFDs	• Replace flow meter and add soft starts or VFDs in 5 – 10 years.
Geysers Pump Station	2011	20	2	• Minor defects and corrosion were noted.	• Replace the equipment in 20 – 30 years.
No. 3 Water System	1991 (modified in 2011)	5 - 10	2	• Minor defects and corrosion were noted. Filters corroding. • There are currently no meters to measure usage.	• Add metering equipment in strategic locations as needed in 1 – 5 years. Replace filters in 5 – 10 years.
Emergency Generator	2000	15	2	• Generator is in relatively good condition with low hours of operations. • Existing generator is grandfathered in with respect to California Air Resources Board regulations. Any upgrades would result in more stringent regulations • Leaking radiator and warped oil pan gaskets were replaced in 2018. • Current generator does not run the entire WRF. It is underpowered to run blowers and effluent pump station.	• Replace the generator in 10 years with a larger unit capable of providing standby power to the entire WRF or at least the activated sludge system. • Coordinate with the Air Board on the permit.
Main Feed Electrical Switch Board	1991	10	2	• Equipment may be close to the end of its useful life. There is unbalanced load to the WRF.	• It is recommended that the MSB be inspected and surveyed. Include thermographic inspection for hot spots. Replace within the next 10 – 20 years.

[1] Based on expected equipment life values. It is assumed that mechanical equipment has a useful life that typically ranges from approximately 15 to 30 years and structures have a useful life that ranges from approximately 30 to 50 years.
[2] Condition rating ranges from 1 (very good) to 5 (unserviceable). The condition shown is the lowest condition of the items in each process area. When the condition is not available, an N/A is inserted. See Condition Assessment forms in Appendix B for more detail.

6. Water Reclamation Facility Capacity Analysis

The purpose of this section is to evaluate how current flows and loadings at the WRF compare to the design capacities of key treatment processes. Hazen updated a report "Small Winery Classification and Investigation Regarding Wastewater Capacity Fees" (2014 Report) completed in April 2014 by Brelje and Race Consulting Engineers. The 2014 Report indicated that there was excess treatment capacity remaining for the secondary treatment facilities. However, a recent change in sampling protocol has changed the analysis (see **Section 3**). The sampling protocol was changed in February 2014 to a method that better reflects influent loads to the WRF. Data collected after the change in sampling protocol show substantially higher BOD_5 and TSS loads. The Town asked Hazen to provide an updated version of the 2014 Report to assess any impacts to the previously reported excess capacity of the treatment processes.

This section focuses on the hydraulic capacities of key unit processes and on secondary treatment capacity. Process flow capacities related to current operations and projected Town growth through 2040 were examined. An updated comparison of design capacity to current operation of the WRF is illustrated in this section. Hydraulic capacities for key unit processes and design loadings for the Biolac® aeration basins were evaluated. The possible consequences of projected population growth were evaluated through 2040 based on the population projections from the adopted Town of Windsor 2040 General Plan. Remaining capacity is defined in terms of equivalent single-family dwellings (ESDs) for each parameter.

6.1 Future Flow and Loading Projections

Population growth projections through 2040 are provided in the Town's 2040 General Plan. These are used to estimate influent flows, BOD_5, TSS, and TKN loadings anticipated through 2040, as discussed in **Section 3**. The average future projected flows and loads are provided in **Table 6.1.**

Table 6.1: Summary of Future 2040 Anticipated Flows and Loads

Parameter	2040 Projection
ADWF	1.9 MGD
Annual Average Flow	3.1 MGD
BOD_5 Load	5,050 lbs/day
TSS Load	4,420 lbs/day
Ammonia Load	900 lbs/day
TKN Load[1]	1,350 lbs/day
TP Load	170 lbs/day

[1]TKN was estimated as a ratio to projected 2040 Ammonia loading. Based on special sampling, a NH_3-N:TKN ratio of 0.67 was used.

WRF standard operating procedure is to equalize peak flows prior to headworks treatment. Current operating protocol is to divert influent flows above 5 MGD except in emergency situations. The annual average flow anticipated in 2040 is 3.1 MGD, which suggests that this procedure could be continued for future flows through 2040. It was assumed that this standard operating procedure would be continued. Thus, the maximum future flow that would be treated in the WRF processes after the headworks was assumed to be 5 MGD.

6.2 WRF Hydraulic Capacity and Current Operation

This section summarizes the design hydraulic capacity of key processes within the WRF; an evaluation of the hydraulic capacity of each unit process is necessary to establish the overall hydraulic capacity of the WRF. A process flow diagram is provided in **Section 5.** When applicable, the current operation or flow of the process is provided using data from 2015 through 2017 so that it can be compared to the design hydraulic capacity. The data used for the current operation of the WRF is derived from **Section 3** unless noted otherwise.

6.2.1 Influent Permitted Flows and Average Annual Flow

The WRF influent flow is regulated based on treatment capacity in addition to available effluent storage and irrigation area. The current permitted average dry weather flow is 1.9 MGD, however this flow may be increased with future increases in effluent storage capacity and irrigation area. The maximum permitted average dry weather flow capacity if effluent storage and irrigation are maximized is 2.25 MGD; this corresponds to the treatment design flow. A summary of permitted flows and current operation is provided in **Table 6.2.** Further discussion of the assumptions for parameters listed in the "Current Operation" column can be found in **Section 3**.

Table 6.2: Influent Flow Capacity

Parameter	Permit Criteria (MGD)	Current Operation (MGD)
Permitted Capacity		
Current ADWF	1.9	1.34 [1]
WRF Design ADWF Capacity	2.25	
Design Peak Weekly Wet Weather Flow	7.2	4.72 [2]
Design Peak Monthly Wet Weather Flow	3.75	3.52 [2]
Annual Average Flow		
Annual Average Flow	-	2.26 [2]

[1]The ADWF calculation uses data from 2015-2017. ADWF was calculated using Influent Adjusted Flow, which is equivalent to the influent flow (INF-001) less the plant recycle flow.
[2]Calculated using flow measured at INF-001 which includes plant recycle flows.

The WRF is approaching its permitted design peak monthly wet weather flow; however, peak flows can be equalized prior to headworks treatment. Current operating protocol is to divert influent flows above 5.0 MGD except in emergency situations. More detailed influent flow data can be found in **Section 3.** Peak monthly wet weather flows can be dampened by following operating protocols to utilize influent equalization.

The average annual flow to the WRF is 2.26 MGD. This flow is measured at the INF-001 location at the headworks, and includes all recycle flows, which are introduced to the influent stream prior to the influent pump station. These capacity evaluations use flows measured at INF-001, because recycle flows contribute to loading and should be included to quantify the remaining capacity at the WRF.

The projected ADWF in 2040 is approximately 1.9 MGD, with an annual average flow of 3.1 MGD (see **Section 3**). The Town will likely be at its current permitted ADWF capacity of 1.9 MGD in 2040,

however future increases in effluent storage and agricultural irrigation would allow the Town to increase its permitted ADWF capacity up to 2.25 MGD.

6.2.2 Influent Pump Station

Influent to the WRF arrives at the influent pump station prior to headworks treatment. Design criteria for the influent pump station are provided in **Table 6.3**.

Table 6.3: Influent Pump Station

Parameter	Design Criteria	Current Operation
Number of Pumps	6 Pumps	Varies
Capacity per Pump	2,600 gpm	2,600 gpm
Firm Influent Pumping Capacity (5 Pumps)	14.8 MGD	Max. 12.6 MGD[1]

[1]*See discussion. Influent pump station flow was analyzed for 12/11/2014-12/18/2014 and 12/7/2015-1/26/2016.*

Peak flow through the influent pump station was analyzed for two flow periods, 12/11/14-12/18/14 and 12/7/15-1/26/16. The flow through the pump station was calculated as the sum of the flow through the headworks, and through the bypass line to influent equalization. The maximum flow measured during this period was 12.6 MGD. WRF staff indicated that this was the highest flow they had observed for at least five years. Since this exceptionally high peak flow is below the firm influent pumping capacity of 14.8 MGD, capacity to treat flows through 2040 is expected to remain. It should be noted that these maximum flows do not correspond to what might occur during larger 10-year or 100-year storms. Potential flows caused by these less frequent storm events are being evaluated in the Town's pending Collection System Master Plan.

6.2.3 Headworks

The headworks consists of a Parkson™ Aquaguard® mechanical screen, a Smith & Loveless® PISTA® Grit Chamber, and a Parshall flume. The design headworks capacity is shown in **Table 6.4.**

Table 6.4: Headworks Flow Capacity

Parameter	Design Criteria	Current Operation
Headworks Capacity	9.6 MGD	Avg. 2.26 MGD Max. 5 MGD

Influent equalization is utilized such that flows to the headworks treatment facilities do not exceed 5 MGD. Future flows through 2040 are also not anticipated to exceed 5 MGD if influent equalization continues to be utilized. Therefore, the headworks should have sufficient capacity to treat flows through 2040.

6.2.4 Aeration Basins

The WRF has two Parkson™ Biolac® aeration basins (Basins A and B). The design hydraulic criteria and current operation of the aeration basins are shown in **Table 6.5**. This includes the design average and

peak flows, and the minimum hydraulic residence time (HRT). This section focuses only on flow capacity; however, the aeration basins are also reviewed with respect to design load (i.e. BOD_5, TSS, TKN) capacity limitations. The design loadings and current loadings are evaluated separately, in **Section 6.3**.

Table 6.5: Aeration Basins Capacity

Parameter	Design Criteria	Current Operation
Number of Basins	2	Varies
Basin Volume (each)	3.0 MG	3.0 MG
ADWF (combined)	3.0 MGD[1]	1.34 MGD
Peak Flow Capacity (combined)	9.6 MGD[1]	Max. 5 MGD
Min. HRT	15 hrs.	1.2 days[2]

[1]These flows are the design flows of influent to the aeration basins. RAS flows are not incorporated in this number.
[2]Assuming a peak flow of 5 MGD.

The current ADWF is 1.34 MGD which is approximately 44 percent of the design ADWF for the aeration basins. The ADWF is anticipated to increase to 1.9 MGD in 2040, which is 63 percent of the design ADWF. It is anticipated that influent equalization can be used to maintain flows within the Biolac® design capacity. Based on the design peak hydraulic capacity of the Biolac® basins (9.6 MGD), and the capability to utilize influent equalization, there appears to be sufficient hydraulic capacity for future average and peak flow conditions.

6.2.5 Blower Capacity Evaluation

The original WRF design utilized four blowers to deliver process air to the Parkson™ Biolac® process. Three additional blowers were installed to account for increased loads during WRF upgrades in 1999, prior to bringing Basin B on-line. **Table 6.6** summarizes the operating characteristics of the existing blowers. Based on discussions with WRF staff, the facility is currently limited to using four small blowers and one large blower due to electrical system constraints, which the Town is currently investigating.

Table 6.6: Existing Blower Characteristics

Quantity	Type	Horsepower (each)	Design Discharge Pressure	Design Airflow/Blower
4	Positive Displacement	50 hp	7.9 psig	968 scfm
3	Positive Displacement	75 hp	7.9 psig	1,291 scfm
			Total Current Limit[1]	**5,163 scfm**
			Total Firm Design Capacity[2]	**6,454 scfm**
			Total Design Capacity[2]	**7,745 scfm**

[1]Current limit based on current electrical limitation.
[2]Firm design capacity assumes one large blower out of service. Total design capacity assumes all units in service.

Figure 6.1 shows the current average, current max month, current max day, and 1999 design average airflow requirements based on the calculations in **Appendix D**. These airflow requirements were calculated using current average, maximum month, and maximum day flows and loads (full-year, not dry-

weather), the design flows and loads stipulated in the 1999 design documentation, and diffuser quantity and operation information (Wave-Ox) from the Biolac® design documentation. Assumptions about Wave-Ox operation (i.e. period of time aerated versus unaerated) were confirmed with Parkson™. All the calculated airflow requirements assume two basins are in service.

Based on this information the analysis yields the following conclusions:

- The design firm capacity of the blowers can meet the current maximum month conditions with about 55 percent excess capacity.
- Based on the current electrical blower limitation (four small blowers and one large blower in service) the blowers can meet current maximum month conditions with approximately 44 percent excess capacity.
- Using the electrical limitations, the blowers can meet current maximum day conditions with approximately 12 percent excess capacity.

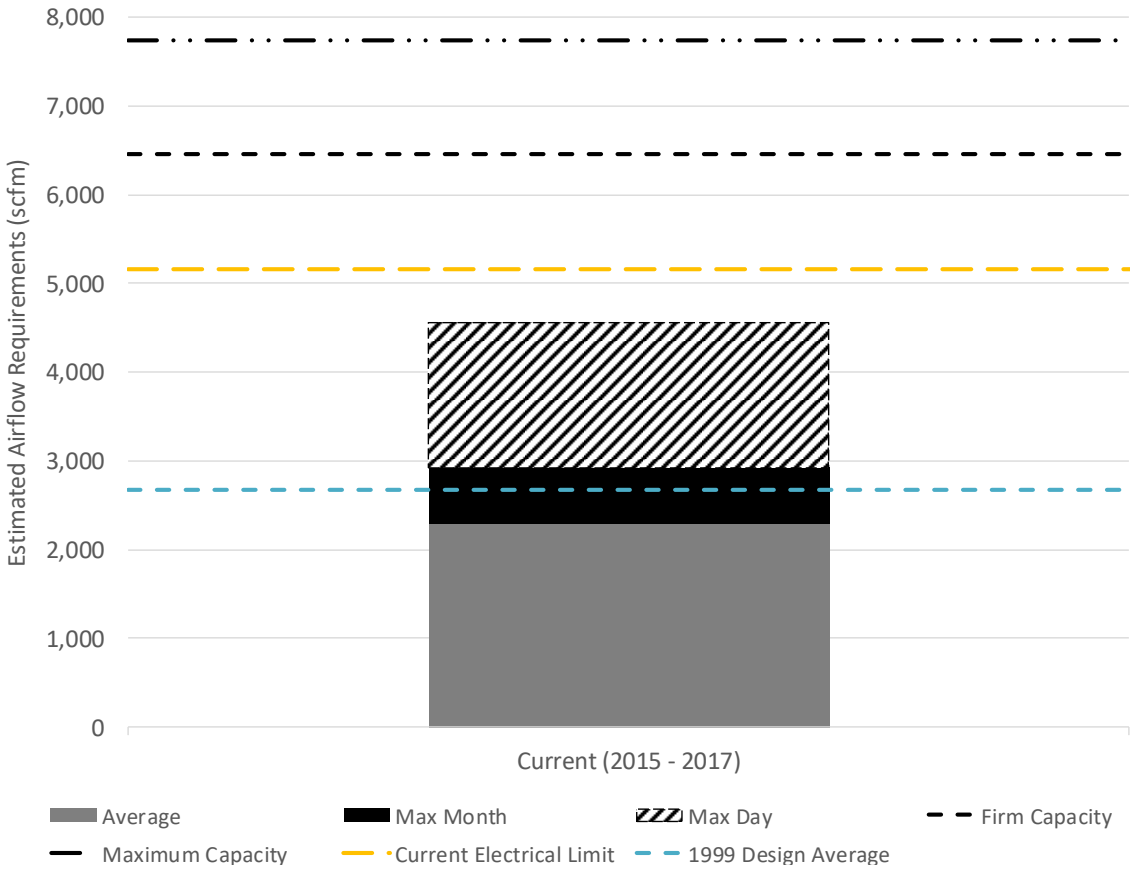


Figure 6.1: Current (2015 – 2017) Loading Airflow Requirements, Two Basins in Service

Based on anticipated loading increases due to population growth, Hazen developed the process air requirements for maximum month and maximum day loading projections to 2040. **Figure 6.2** depicts the projected aeration requirements with current and design airflow capacities shown for reference. Based on

the calculated airflow requirements, the existing firm blower capacity (using one of the two large 75 HP blowers and all four smaller 50 HP blowers) at the WRF should be sufficient for all future loading conditions assuming that the WRF makes appropriate electrical upgrades. Without electrical upgrades, the maximum day blower capacity will be reached or exceeded in approximately 7 years (year 2025) based on the population growth projections in the 2040 General Plan.

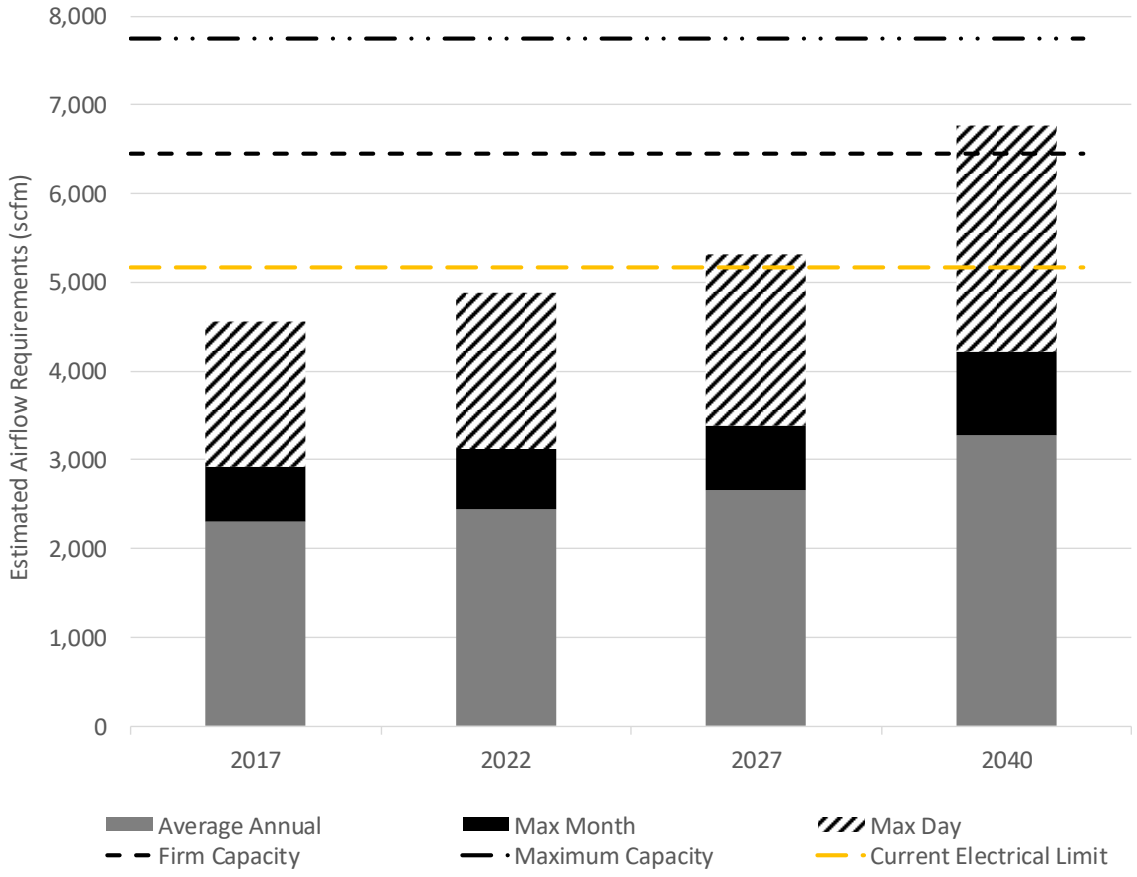


Figure 6.2: Anticipated Process Air Requirements for Growth through 2040

Recent correspondence with operations staff has indicated that diffusers directly adjacent to the sloping sides of the Biolac® basins (ends of the diffuser chains) were set at a higher elevation (a total of 40 of the 340 diffuser sets per basin). This was done to avoid a situation where the diffusers would scrape the sides of the basin leading to premature wear and failure. Details on these findings are available in the Brelje & Race Memorandum titled "WRF Aeration Analysis and Improvements", file number 4248.01 Phase 17.

This difference in diffuser submergence causes air to preferentially flow through the shallower diffusers, resulting in an overall decrease in oxygen transfer efficiency. The Brelje & Race memorandum estimated that an additional 8.7 percent of air was consumed by this loss of efficiency. For the purposes of this study, the capacity analysis assumes that this situation can be remedied and that all diffusers are operating at the same height, as this was the original design intent. A more thorough analysis would be required to quantify the impact of the height differential on overall oxygen transfer efficiency and process air requirements.

Additional details on the airflow requirement calculations, assumptions for diffuser oxygen transfer efficiencies, and physical mass transfer coefficients are described in **Appendix D**. It should be noted that the Town is in the process of installing airflow meters on the main air header to each Biolac® basin. Air transfer efficiencies, blower output capacity, and other assumptions can be validated once this information becomes available.

6.2.6 Secondary Clarifiers

The WRF has two secondary clarifiers, and the characteristics are summarized in **Table 6.7**.

Table 6.7: Secondary Clarifiers

Parameter	Design Criteria	Current Operation[2]
Flow Capacity	9.6 MGD	Avg. 2.26 MGD Max. 5 MGD
Diameter (each)	87 ft	87 ft
Area (each)	5,940 ft^2	5,940 ft^2
Side Water Depth	16 ft	16 ft
Volume (each)	0.71 MG	0.71 MG
Mixed Liquor Suspended Solids	3,500 mg/L	2,600 – 3,500 mg/L [3]
Overflow Rate (gpd/sf)	300 (Average[1]) 700 (Peak[1])	190 (Average) 720 (Peak)
Solids Loading Rate (lbd/sf)	14.4 (Average[1]), 35 (Peak[1])	8.8 (Average) 24 (Peak)

[1]Average and Peak Ranges from Metcalf and Eddy, 4th Edition
[2]Based on 2015 – 2017 data set.
[3]MLSS of approximately 3,500 mg/L have been achieved in more recent operations, with only one aeration basin in service.

Solids loading rate (SLR) and surface overflow rate (SOR) are two of the most commonly used parameters for preliminary secondary clarifier sizing. **Figure 6.3** and **Figure 6.4** depict the SLR and SOR for the WRF clarifiers from 2015 through 2017, and for anticipated 2040 future flows and loads. The green shading on each graph represents the typical design range for average SOR and SLR. The red dotted line on each graph represents the typical design peak values for SOR and SLR.

A procedure similar to that used in **Section 3** to estimate future flows was also used in the analysis of future projected SOR and SLR. Flows were assumed to increase proportionally with projected population growth through 2040. Individual daily data was increased to reflect population growth through 2040. The 2040 projected flows were used to calculate the SOR. The calculations assumed a RAS flow rate equal to the influent flow rate, as these were the average flow proportions during the period analyzed.

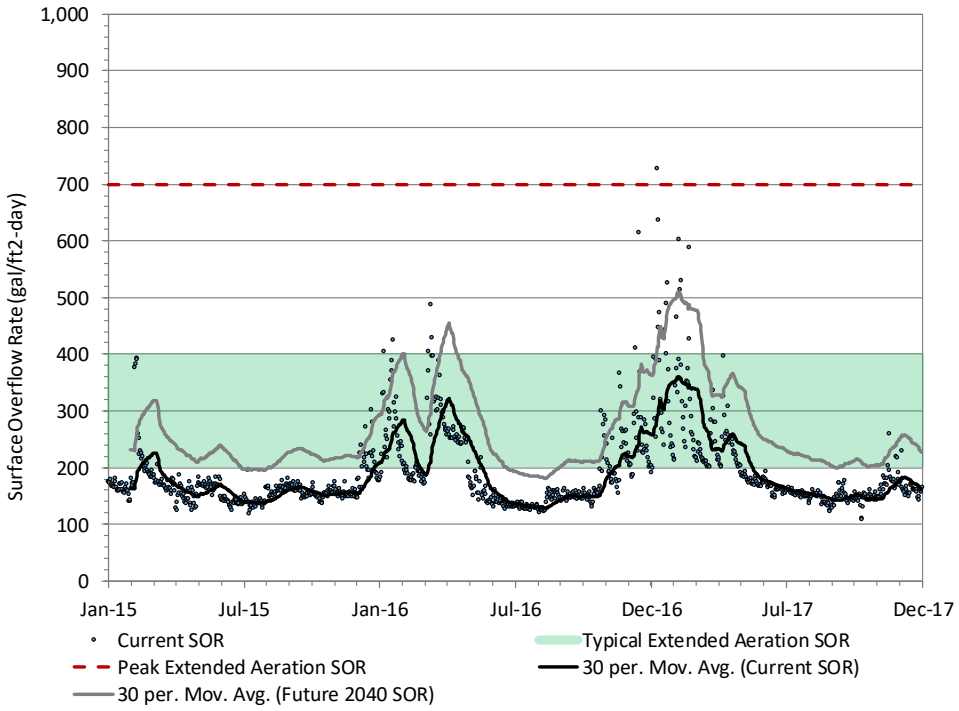


Figure 6.3: Surface Overflow Rate (SOR) and Anticipated Future SOR with Two Clarifiers in Service

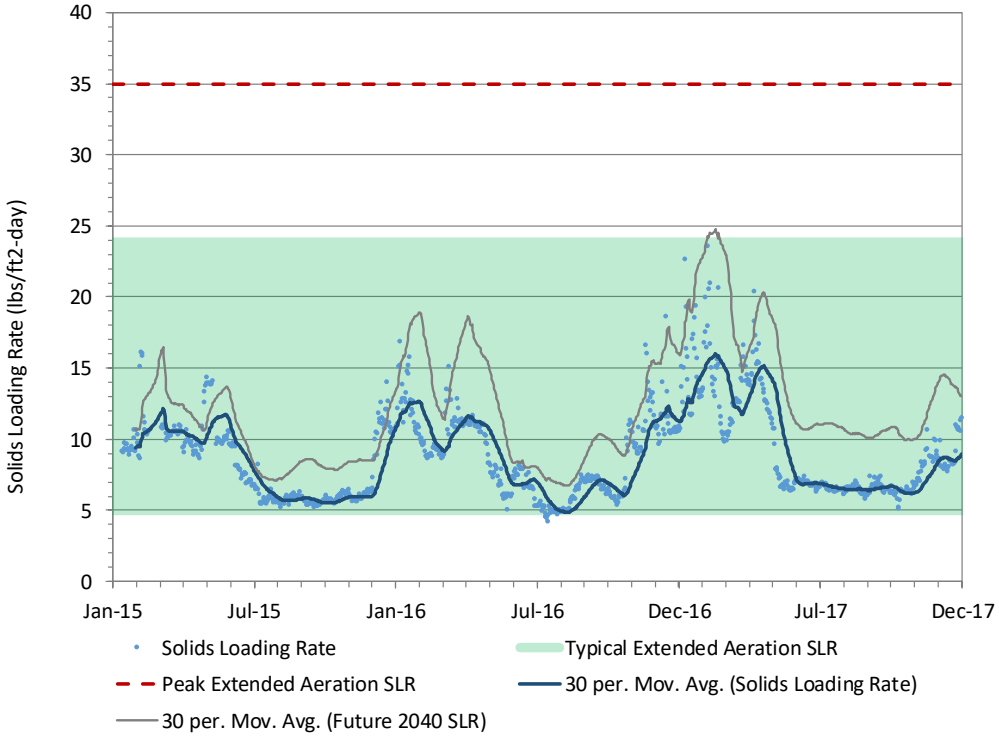


Figure 6.4: Solids Loading Rate (SLR) and Anticipated Future SLR with Two Clarifiers in Service

The analysis shows that the SOR and SLR for current and future conditions are generally within the normal average and peak ranges. For future conditions, peak SOR exceeds the recommended range for a period of a few months. It should be noted that the secondary clarifier design assumed an MLSS concentration 25 percent lower than the current system operation. However, even at the design MLSS the Future 2040 SLR would still remain within the acceptable range.

The SOR and SLR ranges shown above are general, and do not consider the specific characteristics of the WRF sludge. To better understand clarifier capacity, Hazen reviewed sludge volume index (SVI) data as a measure of sludge settleability. The following ranges for SVI are widely used as guidelines for assessing sludge settleability (Metcalf & Eddy, 2003):

1. SVI < 100 mL/g – indicative of good-settling sludge
2. SVI 100 – 150 mL/g – indicative of moderately well-settling sludge
3. SVI > 150 mL/g – indicative of poor-settling sludge.

Figure 6.5 shows the SVI for the period of 2015 through 2017.

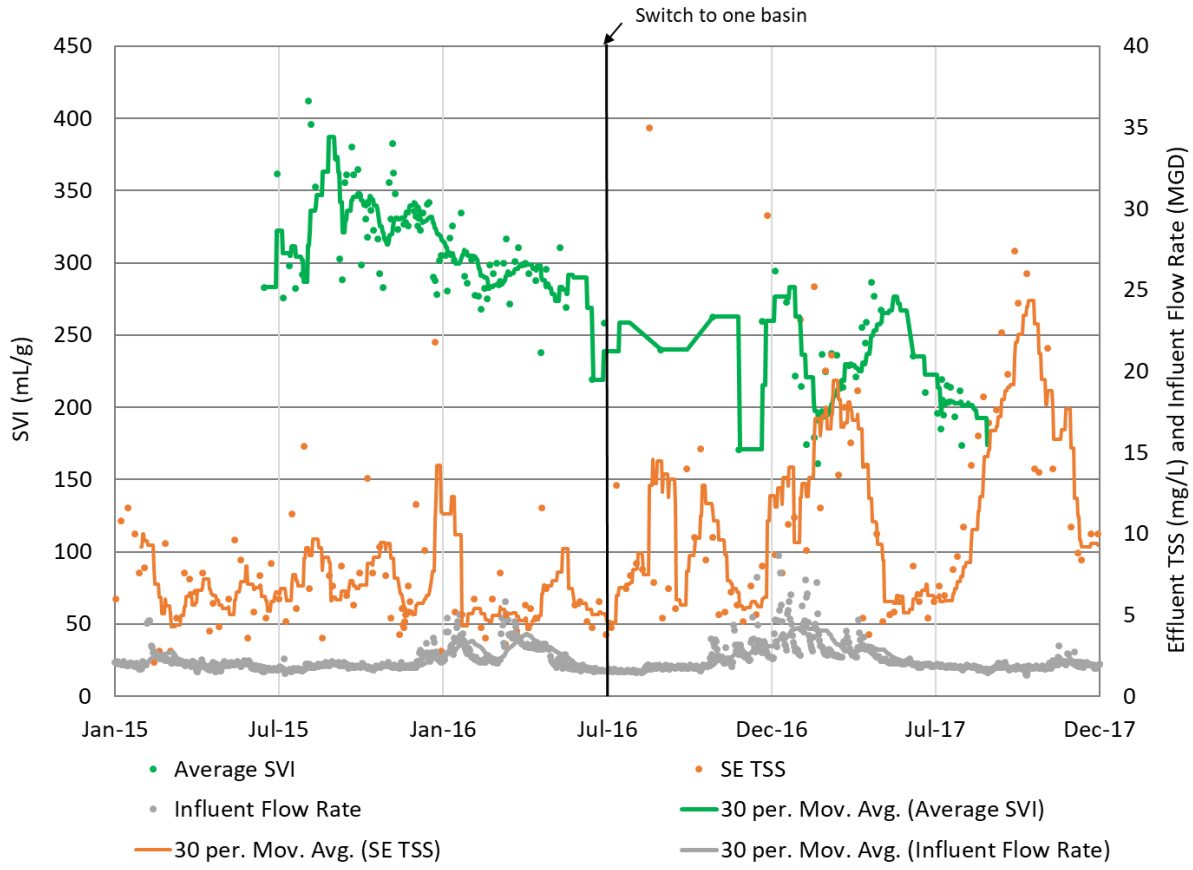


Figure 6.5: Average Sludge Volume Index and Secondary Effluent TSS

The WRF SVI data, depicted in **Figure 6.5**, indicates poor settleability over the 2015 through 2017 period, with an average value of 307 mL/g. Although settleability has improved over the past year, values

between 200 and 250 mL/g are still indicative of a poor settling sludge. This poor settleability reduces the capacity of the secondary clarifiers.

The secondary effluent TSS, also depicted in **Figure 6.5**, indicates that despite the high SVI, the secondary clarifiers perform adequately. The only sustained period of elevated secondary effluent TSS occurs during the high WRF flow rates in February and October 2017. The decrease in SVI after switching to one basin operation in July 2016 is also visible.

The secondary clarifier capacity depends on influent flow rate, RAS flow rate, SVI and MLSS. Below is a brief sensitivity analysis of secondary clarifier capacity with respect to influent flow rate and SVI. To better understand the effect of influent flow rate on secondary clarifier capacity, Hazen has made the following assumptions which are based on operational targets for two basins:

1. SVI: 225 mL/g
2. MLSS: 2,500 mg/L
3. Maximum RAS Flow Rate: 6.1 MGD (see **Table 6.8**)

Under these conditions, the secondary clarifiers can reliably remove suspended solids up to an influent flow rate between 5 and 5.5 MGD. Above that flow rate, the clarifiers approach their critical solids loading rate and may lose solids in the effluent. At these operating conditions, a third secondary clarifier would provide the required capacity to meet influent flows up to approximately 7.5 MGD.

To better understand the sensitivity of secondary clarifier capacity to changes in SVI, Hazen has made the following assumptions:

1. Influent Flow Rate: 5.0 MGD (maximum operation WRF treatment flowrate)
2. Maximum RAS Flow Rate: 6.1 MGD
3. MLSS: 2,500 mg/L

At these conditions, the secondary clarifier capacity is sufficient to meet 2040 flows up to an SVI of between 235 - 250 mL/g. Over that SVI, the clarifiers exceed the critical solids loading rate and will begin losing solids in the effluent. As demonstrated here and in **Appendix E**, the secondary clarifier capacity and the necessity for a third clarifier is strongly related to SVI.

The WRF has sufficient hydraulic capacity for projected flows through 2040. Loading with regards to TSS, TKN and BOD_5 capacities are addressed in **Section 6.3**.

6.2.7 Return Activated Sludge

The return activated sludge (RAS) pumps are divided between two pump stations, with one pump station dedicated to each secondary clarifier. Pumping capacity is summarized in **Table 6.8.**

Table 6.8: Return Activated Sludge Pump Capacity

Parameter	Design Criteria		Current Operation[1] (Average)
	Pump Station A	Pump Station B	
Pump Station Capacity	2.6 MGD [2]	3.5 MGD [2]	2.3 MGD (both basins)

[1] *The calculation for the current operation of the combined RAS pump stations uses data from 2015-2017.*
[2] *The capacities were taken from the Water Reclamation Facilities O&M Manual (2002).*

The current combined RAS flow rate is 38 percent of the design capacity. It appears that under the current operation of the WRF, RAS pumping capacity should be sufficient through 2040.

6.2.8 Waste Activated Sludge

The waste activated sludge (WAS) pump station sits near secondary clarifier B and consists of two pumps. WAS pump capacity is provided in **Table 6.9.**

Table 6.9: Waste Activated Sludge Pump Capacity

Parameter	Design Criteria	Current Operation
Number of Pumps	2	Varies
Capacity per Pump	200 gpm	200 gpm
Firm WAS Capacity (one pump)	0.29 MGD	0.11 MGD[1]
Total WAS Capacity (two pumps)	0.58 MGD	

[1] *The calculation for the current operation of the WAS pump stations uses data from 2015-2017.*

The WAS flow rate is 38 percent of the firm design capacity (one pump out of service). It appears that WAS capacity should be sufficient through 2040.

6.2.9 Advanced Wastewater Treatment System

The advanced wastewater treatment (AWT) system consists of an AWT pump station, six (6) flocculation tanks, three AWT clarifiers, and 5 AWT sand filters. Key design criteria of the AWT pump station, flocculation tanks, and clarifiers are compared to current operating flows in **Table 6.10.** The AWT sand filters are discussed in **Table 6.11.**

Table 6.10: Advanced Wastewater Treatment System Capacity

Parameter	Design Criteria	Current Operation
AWT Pumping		
AWT Pumping	9.6 MGD	Avg. 2.26 MGD Max 5.0 MGD
AWT Flocculation		
Flocculation Tanks	6	Varies
Stages per Flocculation Tanks	2	2
Stage Volume	0.0125 MG	0.0125 MG
Total Volume	0.15 MG	0.15 MG
Total Design Capacity	7.2 MGD	Avg. 2.26 MGD Max. 5 MGD
Current Average HRT[1]	-	96 min[1]
Minimum HRT (HRT @ Maximum Flow)	30 min	43 min[2]
AWT Clarifiers		
Number of Clarifiers	3	Varies
Volume per Clarifier	0.24 MG	0.24 MG
Total Volume	0.72 MG	0.72 MG
Flow Capacity	7.2 MGD	Avg. 2.26 MGD Max. 5 MGD
Current Average Detention Time[1]	-	7.6 hrs[1]
Min. Detention Time	2.4 hrs.	3.5 hrs[2]

[1]Based on INF-001 Flows from 2015 through 2017. See Section 3.
[2]Based on current standard operating procedure – diversion of flows greater than 5 MGD

These tertiary treatment processes were reviewed only in terms of hydraulic capacity. The AWT pumping station has a 9.6 MGD design capacity, and the AWT flocculation tanks and clarifiers have a 7.2 MGD design capacity. Influent flows are not anticipated to exceed 5 MGD through 2040 if standard operating procedures are followed. This suggests that the AWT facilities have sufficient capacity through 2040. Key AWT sand filter design criteria are listed in **Table 6.11.**

Table 6.11: AWT Sand Filters Capacity

Parameter	Design Criteria		Current Operation
	Parkson™ Dynasand®	Eimco Rotoco[1]	
Number	5	2	Varies
Surface Area	200 ft^2	200 ft^2	200 ft^2
Maximum Loading Rate	5 gpm/ft^2	3 gpm/ft^2	5 gpm/ft^2
Maximum Design Flow per Filter	1.44 MGD	0.86 MGD	1.44 MGD
Firm Capacity	5.8 MGD	-	Avg. 2.26 MGD Max. 5 MGD
Total Capacity	7.2 MGD	1.72 MGD	

[1]Eimco Rotoco filters are not operational and non-serviceable

The total capacity of the five operating Parkson™ Dynasand® filters is 7.2 MGD with all filters in service. The firm capacity with one Parkson™ Dynasand® filter out of service is 5.8 MGD. Influent flows are not anticipated to exceed 5 MGD through 2040 if standard operating procedure is followed. This suggests that the filters have sufficient capacity through 2040.

6.2.10 UV Channels

The design criteria for the UV channels is summarized in **Table 6.12.**

Table 6.12: UV Channel Capacity

Parameter	Design Criteria	Current Operation
Number of Channels	2	Varies
Flow Capacity (each)	5.1 MGD	5.1 MGD
Total Flow Capacity (both)	10.1 MGD[1]	Avg. 2.26 MGD Max. 5 MGD

[1]Total flow not a multiple of flow capacity due to rounding.

The annual average influent flow was 2.26 MGD, which is 22 percent of the UV channel flow capacity. The annual average flow in 2040 is anticipated to be 3.1 MGD. Peak flows are expected to be limited to 5.0 MGD per influent diversion protocols previously stated. Based on these anticipated influent flows, the UV channels are expected to remain within their hydraulic capacities through 2040.

6.2.11 Geysers Pump Station

The design criteria for the Geysers Pump Station is provided in **Table 6.13**.

Table 6.13: Geysers Pump Station Capacity

Parameter	Design Criteria	Current Operation
Number of Pumps	4 [1]	Varies
Maximum Design Flow (per pump)	0.56 MGD [2]	0.56 MGD [2]
Pump Station Firm Capacity	1.67 MGD	Max. 0.7 MGD

[1] The design documents list 3 duty pumps and 1 standby pump.
[2] This number is rounded.

The WRF currently delivers 0.2 MGD to the Geysers via the Geysers pump station and pipeline during non-discharge months and 0.7 MGD during the discharge season, for an annual average flow of 0.53 MGD. In the future, if the flow to the Geysers is increased to an annual average flow of 0.75 MGD, the future flow during the non-discharge season will be 0.42 MGD and the flow during the discharge season will be 0.92 MGD, as listed in Table 1 of the Geysers Pipeline Agreement. Thus, the capacity is sufficient for future flows through 2040.

6.2.12 Effluent Pump Station

The design criteria for the effluent pump stations is summarized in **Table 6.14.**

Table 6.14: Effluent Pump Station Capacity

Parameter	Design Criteria	Current Operation
Number of Pumps	4	Varies
Pump Capacity	2,500 gpm, 1,100 gpm, Two Pumps @ 500 gpm	2,500 gpm, 1,100 gpm, Two Pumps @ 500 gpm
Effluent Pumping Capacity	7.98 MGD	Avg. 1.09 MGD Max. 4.20 MGD

The annual average flow in 2040 is anticipated to be 3.1 MGD, but the peak flow is expected to be limited to 5.0 MGD per influent diversion protocols previously stated. Effluent pump station flows would not typically exceed influent flows, unless the effluent storage ponds were being drained. Even if the WRF was experiencing a peak flow of 5.0 MGD, the pump station would still have 2.98 MGD excess capacity to drain the effluent storage ponds. Based on these anticipated influent flows, the effluent pump station is expected to remain within its hydraulic capacity through 2040.

6.2.13 Hydraulic Capacity Summary

Table 6.15 summarizes the current permitted ADWF and the maximum permitted ADWF with the current ADWF and the projected ADWF in 2040. It should be noted that in order to re-rate the WRF for the maximum permitted ADWF, additional storage and irrigation area would be required. This is further discussed in **Section 8.**

Table 6.15: Overall WRF Hydraulic Capacities

Parameter	Total Hydraulic Capacity (MGD)	Current Flow (MGD)	Projected 2040 Flow (MGD)
Permitted ADWF	1.9	1.34	1.9
Design ADWF	2.25	1.34	1.9

Table 6.15 indicates that the current permitted ADWF could be insufficient for future projected 2040 ADWF. The future projected 2040 flow is 1.9 MGD, which is exactly the permitted ADWF capacity. Because the future flow projections are inherently uncertain, steps may need to be taken to re-rate the WRF. This will provide a margin of safety to ensure the Town remains within its permitted limits. This is discussed further in **Section 6.5.**

Table 6.16 summarizes the flow capacities of the major unit processes, including the current flows, the projected 2040 annual average flow, and the peak flows anticipated. Current and future peak flows are not expected to exceed 5 MGD if the current standard operating procedure for influent equalization is continued.

Table 6.16: Summary of Maximum Flow Capacities of Major Unit Processes

Parameter	Total Hydraulic Capacity (MGD)	Firm Capacity[1] (MGD)	Current Flow	Projected 2040 Flow
Influent Pump Station	NA[2]	14.8	Avg. 2.26 Max. 12.6	NA[3]
Headworks	9.6	NA	Avg. 2.26 Max. 5	Avg. 3.1 Max. 5
Aeration Basins	ADWF 3.0 Max. 9.6	ADWF 1.5 Max. 4.8	Avg. 2.26 Max. 5	Avg. 3.1 Max. 5
Secondary Clarifiers	9.6	4.8	Avg. 2.26 Max. 5	Avg. 3.1 Max. 5
AWT Pumping	9.6	7.1	Avg. 2.26 Max. 5	Avg. 3.1 Max. 5
AWT Flocculation	7.2	4.8	Avg. 2.26 Max. 5	Avg. 3.1 Max. 5
AWT Clarifiers	7.2	4.8	Avg. 2.26 Max. 5	Avg. 3.1 Max. 5
AWT Sand Filters	7.2	5.8	Avg. 2.26 Max. 5	Avg. 3.1 Max. 5
UV Channels	10.1	5.1	Avg. 2.26 Max. 5	Avg. 3.1 Max. 5

[1] Reflects the capacity with 1 unit out of service (1 pump, filter, etc.).
[2] The total hydraulic capacity of the Influent Pump Station is not available in the Water Reclamation Facilities O&M Manual (2002)
[3] As previously mentioned, potential peak flows for future storm events are being evaluated by in the Collection System Master Plan

With peak 2040 flows of 5.0 MGD, the main processes have sufficient capacity through 2040. Based on the findings in this section, the secondary clarifiers, AWT flocculation, and AWT clarifiers would be limited to 4.8 MGD with one unit out of service. These systems would have sufficient capacity if all units were in service. Having one unit out of service would be required for maintenance activities. The UV channels would be limited to 5.1 MGD with one unit out of service. UV maintenance activities would require that one channel be out of service. However, for both current and future conditions equalization would continue to be utilized to keep WRF influent flows less than 5.0 MGD. Therefore, it is unlikely that the UV system would be a bottleneck at this flow.

6.3 WRF Secondary Treatment Design Loading Analysis

The purpose of this section is to provide additional analysis for the comparison of the current and future flows and loads to the design loads of the secondary treatment system. The WRF has two Biolac® aeration basins (Basins A and B). The design criteria and current operation of the aeration basins are shown in **Table 6.17**. These design loads were calculated based on the design criteria in the 1994 and 1999 Parkson™ Design Documentation.

Table 6.17: Aeration Basins Design Flows and Loads

Parameter	Design Values		Current Operation			Future 2040 Operating Percent of Design[1]
	Criteria	Design Load	Current Operation		Current Operation Percent of Design	
ADWF	3.0 MGD	-	1.34 MGD		44%	63%
Peak Flow Capacity	9.6 MGD[2]	-	5 MGD		52%	52%
Average BOD_5	260[3] mg/L	6,500 lbs/day	212 mg/L	3,630 lbs/day	56%	77%
Average TSS	140 mg/L	3,500 lbs/day	186 mg/L	3,180 lbs/day	91%	126%
Average NH_3-N	30 mg-N/L	750 lbs N/day	39 mg-N/L	654 lbs N/day	87%	120%
Average TKN	40 mg-N/L	1,000 lbs N/day	59 mg-N/L[4]	971 lbs N/day[4]	97%	135%
Average TP	-	-	7.24 mg-P/L	120 lbs P/day	-	-

[1]Future 2040 anticipated loads contained in Section 3.
[2]Peak Flow Capacity represents secondary influent flow capacity. It does not include RAS flows.
[3]The BOD_5 design criteria taken from the Phase IIA Stage 2 Expansion drawings.
[4]TKN was estimated from influent ammonia data and special sampling results. Based on this, a NH_3-N:TKN ratio of 0.67 was used.

From a loading standpoint, the current annual average and anticipated 2040 BOD_5 loads are below the design values; however, current NH_3-N and TKN loads are approaching design values. NH_3-N and TKN loads are anticipated to exceed ADWF design loads by approximately 20 and 35 percent, respectively, in 2040. Similarly, current TSS loads are also approaching the design load and are anticipated to exceed the design load by approximately 26 percent in 2040.

The following key assumptions should be noted when comparing current and future loads to the design flows and loads:

1. The Parkson™ Biolac® design assumed a mixed liquor suspended solids (MLSS) of 3,500 mg/L.
    - Operations between February 2014 and July 2017 maintained an MLSS of approximately 2,600 mg/L. Operations have shifted to an MLSS concentration closer 3,500 mg/L after July of 2017.
    - The reduced MLSS concentration has a negative impact on secondary treatment capacity.
2. The Parkson™ Biolac® design assumes two (2) basins in operation.
    - Since June 2016, the WRF has been operating using one basin only.
    - One basin operation was commenced to increase the food-to-mass (F/M) ratio. The increase in F/M due to one basin operation has also correlated with lower sludge volume index (SVI) and improved sludge settleability.

- Operating using one basin increases process air requirements due to more restricted oxygen transfer efficiency. Higher air flows to a single basin results in air bubbles that are larger, on average. This results in a lower overall surface area in which oxygen transfer can occur, and thus a lower oxygen transfer efficiency.
- The WRF has been able to treat current loads with only one basin in operation.

A summary of the secondary effluent quality is provided in **Table 6.18.** The secondary effluent values are greater than the final effluent measurements shown in **Section 3** because further treatment occurs in the Advanced Wastewater Treatment process (see **Section 6.2.9**). The final effluent measurements are regulated under the WRF's NPDES permit, as described in **Section 4.**

Table 6.18: Secondary Effluent (SE) Quality

Parameter	Two Basins Average (Aug. 2014 – Jun. 2016)	One Basin Average (Jun. 2016 – Dec. 2017)	Maximum (Aug. 2014 – Dec. 2017)
SE BOD_5 (mg/L)	7.2	14.5	45
SE TSS (mg/L)	8.2	9.5	35
SE NH_3-N (mg-N/L)	<1	<1	3.2
SE NO_3 (mg-N/L)	4.8	6.3	23

The secondary effluent long-term historical data review shows significant variations in effluent quality. **Table 6.18 s**hows that one basin operation resulted in higher secondary effluent BOD_5 compared to two basin operations. Because BOD_5 should be consumed quickly in the aeration basin, this points to the possibility of hydraulic short-circuiting and/or inadequate dissolved oxygen for soluble BOD_5 removal using one basin.

As the WRF has been able to treat current loads with only one basin in service, it is likely that the projected future loads, which are 39 percent greater than the current loads, can be treated with two basins in service. The data suggest that maintaining a proper F/M does have positive impacts on sludge settleability. Therefore, while one basin operation for the current flows and loadings improves secondary clarifier operation, conversely, it slightly reduces the aeration system efficiency. Further analysis of whether a good F/M ratio can be maintained if operation is changed to two basins should be conducted.

An additional analysis was conducted for the comparison of the current and future loading to the design loads of the aeration basins. As shown in **Table 6.17**, current flows and loads to the Biolac® aeration basins are approaching their design loadings and should be studied in greater detail. This section provides additional analysis for ADWF, BOD_5, TSS, and TKN loads for current and future scenarios. The approach followed the analysis in the 2014 Report conducted by Brelje and Race. Monthly average loadings were projected based on current BOD, TSS, and TKN loading rates and the population growth projections in the 2040 General Plan. 2040 flows and loads are stated in **Table 6.1**. Because annual average loadings do not provide detail of the fluctuations in loading, the following sections analyze the variation of these parameters in more detail.

6.3.1 Average Dry Weather Flow

Figure 6.6 presents current flows (2/1/2014-12/31/2017) with the effects of population growth (to 2022, 2027, and 2040). Future flows were projected to determine the effect of population growth. The population in 2016 (the midpoint of the data analyzed) was 27,336 residents, and it is projected that there will be 38,028 residents by 2040 based on the 2040 General Plan. The ratio of 2040 to 2015 population is 1.39 or a 39 percent increase in population. Thus, each average daily flow was multiplied by a factor of 1.39. A similar analysis was repeated with shorter time steps – 5 years (to 2022) and 10 years (to 2027). The population projections estimated an approximately 8.5 percent increase in population to 2022 (29,700 residents) and a 12 percent increase in population in 10 years (31,800 residents).

The influent adjusted flow is used for this analysis, because the ADWF should not include recycle flows. Because the average dry weather flow is defined as the minimum 30-day flow, a 30-day rolling average is shown. The current ADWF capacity set by the NPDES permit is 1.9 MGD, however this is anticipated to increase to 2.25 MGD prior to 2040. The Town will have to demonstrate that it has sufficient reclamation and storage capacity for the increase. The design capacity of the aeration basins is 3.0 MGD for ADWF, however this flow is not allowed by the current NPDES discharge permit.

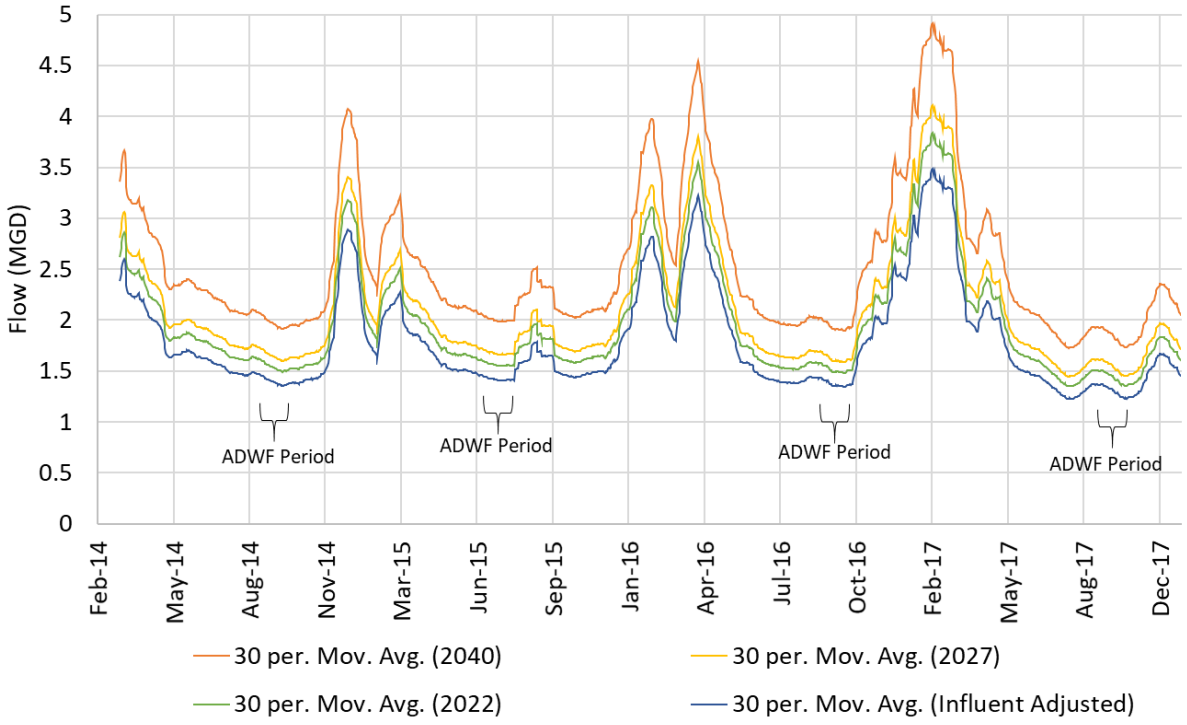


Figure 6.6 Current and Projected Influent Adjusted Flows

It should be noted that the current 1.9 MGD permitted flow and the future 2.25 MGD permitted flow applies only to the ADWF. It is acceptable for peaks to exceed the ADWF limits. A summary of current and projected ADWFs through 2022, 2027, and 2040 is provided in **Table 6.19.**

Table 6.19: Current and Estimated Future Flows

	Flow
Permitted ADWF	1.9 (current) – 2.25 (2040) MGD
Average Recent ADWF (2015-17)	1.34 MGD
Range of ADWF	1.23 (2017) – 1.41 (2015) MGD
Projected 2022 ADWF	1.5 MGD
Projected 2027 ADWF	1.6 MGD
Projected 2040 ADWF	1.9 MGD

The above projections are only estimates. Population growth through 2040 may cause the WRF to exceed the current maximum permitted ADWF. Because the design average flow of the aeration basins is 3.0 MGD with both basins in operation, the true process capacity of the WRF is likely sufficient to treat flows through 2040.

6.3.2 Biological Oxygen Demand

The BOD_5 design load of the secondary treatment process is 6,500 lbs/day, as calculated from the design flow and load in **Table 6.5**. Any loadings that are 90 percent of the design load would indicate that the Town is approaching its design loading and should consider upgrades. For BOD_5, this is 5,850 lbs/day. **Figure 6.7** shows the current (2/1/2014-12/31/2017) average monthly BOD_5 loading in pounds per day entering the WRF's secondary treatment basins (labeled Current). Future average monthly loadings were projected to determine the effect of anticipated population growth. BOD_5 projections are shown in **Figure 6.7.** By 2040, the BOD_5 mass loading is expected to increase by a factor of 1.39.

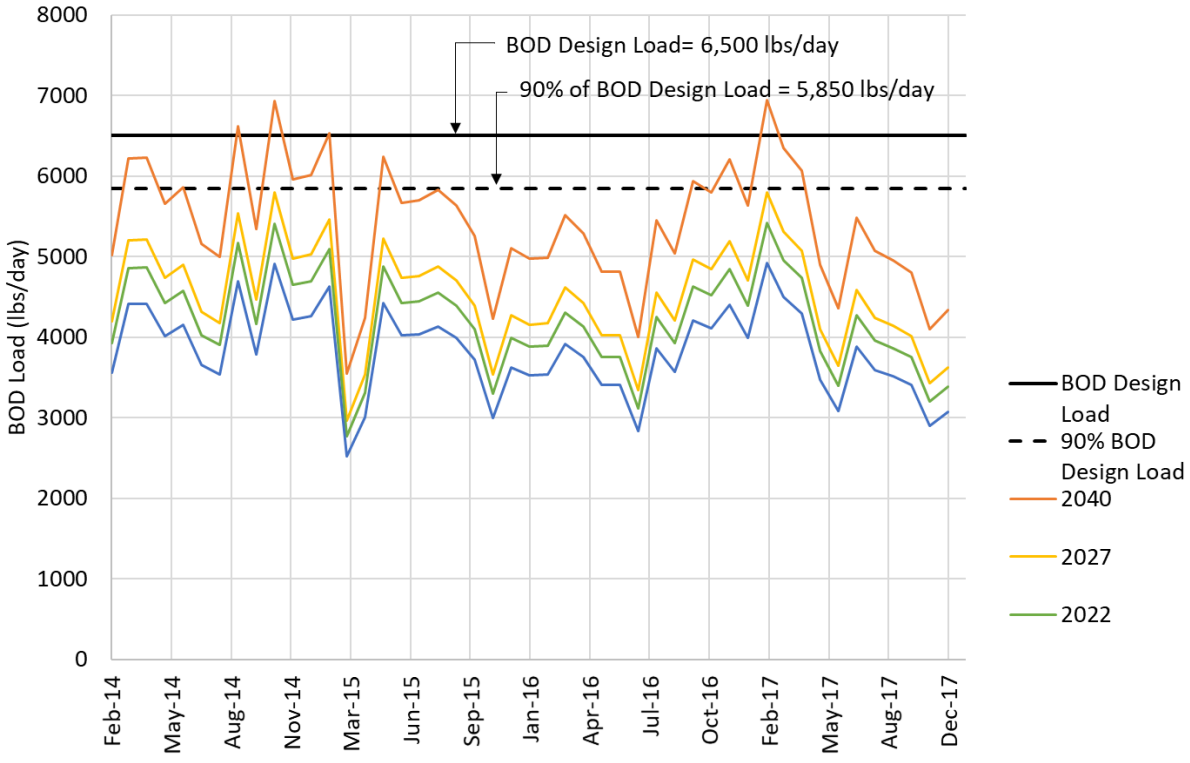


Figure 6.7: Current and Projected Average Monthly BOD_5 Loading

Currently, BOD_5 design loading has not been exceeded. However, by 2027 projected loadings have approached 90 percent of the BOD_5 design load. Recall that the graph shows average monthly loadings, and thus each exceedance of the design load represents a potential month in which the load exceeds the design load. The analysis suggests that population growth through 2040 may cause the WRF to exceed its design loading. Based on the assumptions that are made in this analysis, it is estimated that the WRF would exceed its design BOD_5 load around 2036.

As discussed previously, the Town is typically able to treat current flows, even when operating only on one Biolac® basin. This suggests that capacity likely remains if two basins are operated in the future. The Town should work with Parkson™ to ensure consistently high-quality secondary effluent, and to evaluate what ultimate treatment capacity is available if two basins are utilized. The Town could then potentially re-rate the facility and determine what additional capacity is available for future population growth.

6.3.3 Total Suspended Solids

Figure 6.8 shows the current and projected TSS loading based on current (2/1/2014-12/31/2017) data. The projected TSS loading increases resulting from population growth are shown for 2022, 2027, and 2040. The TSS projections are extrapolated using the same method used for ADWF and BOD_5 in **Sections 6.3.1** and **6.3.2**.

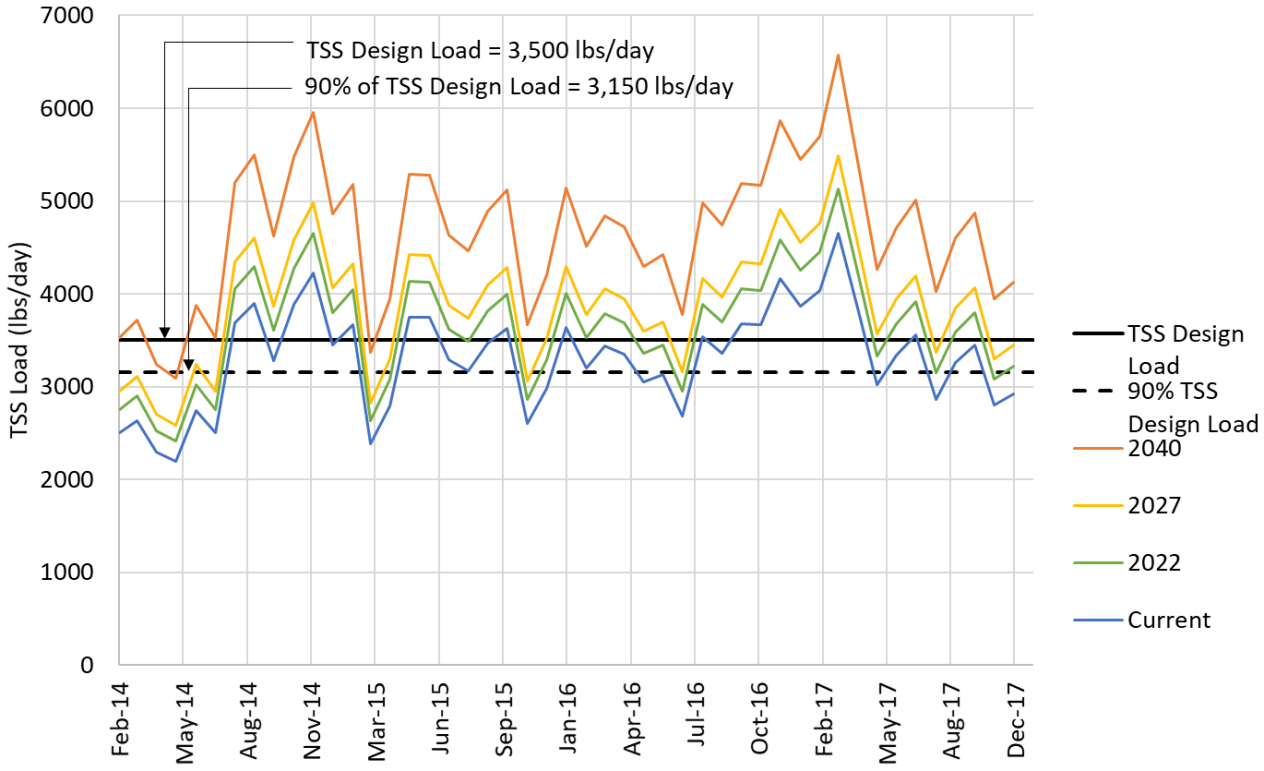


Figure 6.8: Current and Projected Average Monthly TSS Loading

As shown **Figure 6.8** there are times when the TSS exceeds secondary design loading of 3,500 lbs/day, however the WRF has been able to meet its effluent TSS limits. Because population growth could cause

the WRF to further exceed its design loading and potentially to exceed its capacity to treat TSS, recommendations are provided in **Section 6.5** to help the Town evaluate and improve TSS treatment.

6.3.4 Total Kjeldahl Nitrogen

Total Kjeldahl Nitrogen (TKN) is the sum of organic nitrogen and ammonia species (reduced nitrogen). TKN represents the nitrogen that is available for biological growth. The TKN loading increase estimated due to population growth is shown in **Figure 6.9** for current operation, 2022, 2027, and 2040. The projections are extrapolated using the method used for ADWF, BOD_5, and TSS.

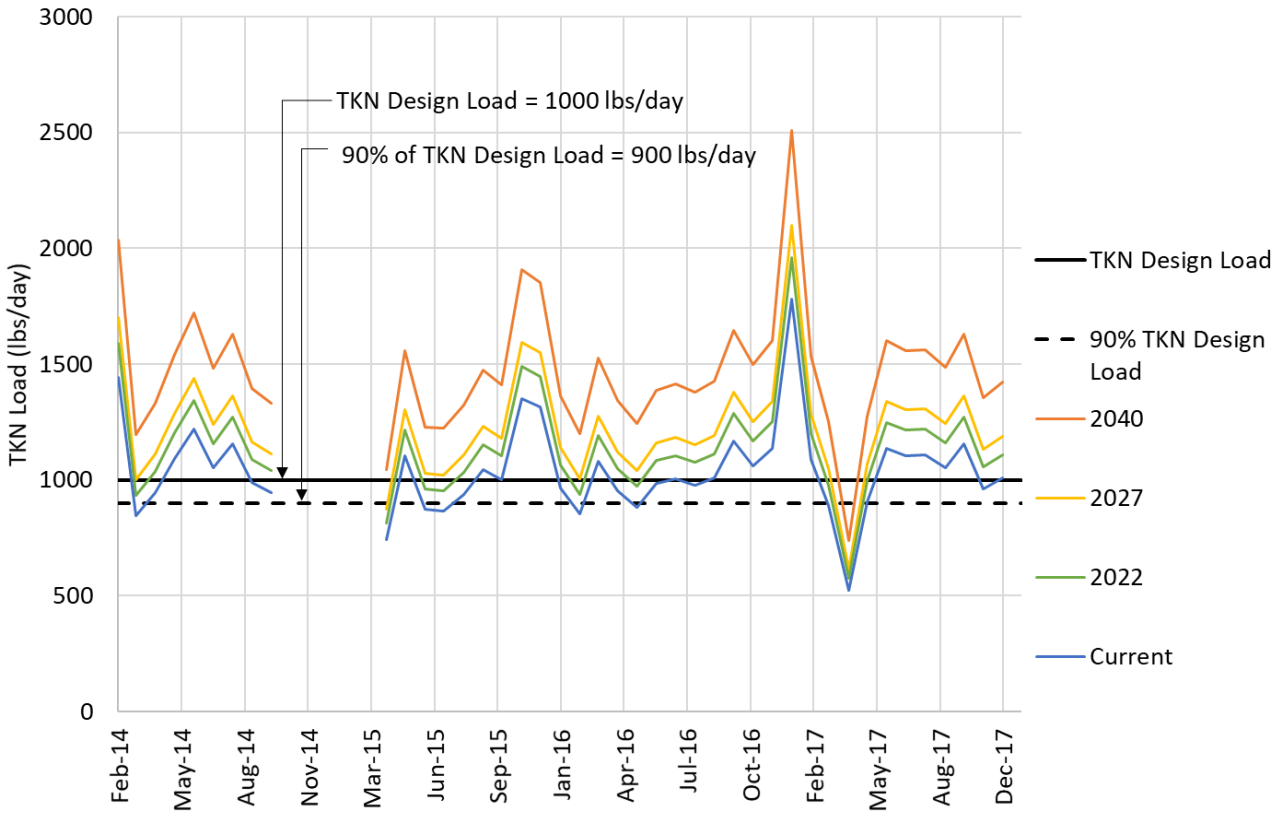


Figure 6.9: Current and Projected Average Monthly TKN Loading

The WRF is currently operating at its design TKN loading, however the WRF is currently removing nitrogen more efficiently than was anticipated in the design. Typically, the WRF is able to provide fully nitrified effluent, though operations should be optimized to ensure this occurs consistently. As shown in **Section 3**, effluent ammonia measured at the UV Effluent has been variable.

It should be noted that the NPDES effluent limitations apply to Discharge Point 002 (after the effluent storage ponds) and additional denitrification typically occurs in the effluent storage ponds. This is also assisting the WRF with meeting its permit limits. However, because population growth is likely to cause the WRF to see influent TKN concentrations above its original design loading, recommendations are provided in **Section 6.5** to optimize nitrogen removal through denitrification.

6.3.5 Summary of Secondary Treatment Flow and Design Loading Analysis

Table 6.20 shows the 90 percent design load for BOD_5, TSS, and TKN, and shows the current loadings and future projected loadings through 2040.

Table 6.20: Secondary Treatment Loading Capacity for BOD, TSS and TKN

Parameter	Design Load (lbs/day)	90% Design Load (lbs/day)	Current [1] Load (lbs/day)	Current [1] Peak Monthly Load (lbs/day)	Projected 2040 Load (lbs/day)	Projected 2040 Peak Monthly Load (lbs/day)
BOD_5	6,500	5,850	3,630	4,680	5,050	6,510
TSS	3,500	3,150	3,180	3,880	4,420	5,390
TKN [2]	1,000	900	971	1,280	1,350	1,770

[1] 2015-2017 average load. For additional information see Section 3.
[2] In lbs N/day

Table 6.20 indicates that current average loadings for TSS and TKN are above the 90 percent design load for these parameters. Future loading through 2040 would likely exceed the design load.

The projected 2040 peak monthly loads for BOD_5, TSS, and TKN exceed the design load. In fact, even the current TKN peak monthly loads exceed the design load. As mentioned previously, the WRF is typically able to meet its effluent limits; however, additional steps should be taken to ensure the WRF has sufficient capacity through 2040. This is further discussed in **Section 6.5**.

Table 6.21 updates the ADWF chart shown in **Section 6.2.13** to include the design flow of the aeration basins.

Table 6.21: WRF ADWF Capacities

Parameter	Total Hydraulic Capacity (MGD)	Current Flow (MGD)	Projected 2040 Flow (MGD)
Permitted ADWF	1.9	1.34	1.9
Design ADWF	2.25	1.34	1.9
Aeration Basins Design ADWF	3.0	1.34	1.9

Table 6.21 illustrates that the aerations basin design ADWF is likely sufficient for anticipated flows through 2040. As stated previously, the projected 2040 ADWF is at the limit of the current permitted ADWF (both are 1.9 MGD). Because the future flow projections are inherently uncertain, steps may need to be taken to re-rate the WRF. This will provide a margin of safety to ensure the Town remains within its permitted limits. This is discussed further in **Section 6.5.**

6.4 Equivalent Single-Family Dwellings

Moving forward, the Town would like to evaluate excess secondary treatment loading capacity in terms of equivalent single-family dwellings (ESDs) for consistency with their fee calculation. Currently the WRF has remaining capacity for BOD_5. It has reached its design loading capacity for TSS and TKN. The

BOD_5 loading associated with one ESD is calculated below. The calculation uses the Town's assumed typical flow (200 gpd) and BOD_5 concentration (200 mg/L) for one ESD.

$$BOD\ Loading\ for\ 1\ ESD = 200\ gpd \div 10^6\ {}^{MGD}/_{gpd} \times 200\ {}^{mg}/_{L} \times 8.34^{*} = 0.334\ lbs/day$$

**The factor 8.34 is the conversion factor from MGD*mg/L to lbs/day.*

The average remaining BOD_5 loading is equivalent to the 90 percent design load minus the average BOD_5 loading. The remaining loading capacity is 2,220 lbs/day which is approximately 6,650 ESDs. A similar calculation could be conducted for TSS and TKN, however no capacity remains for these parameters, based on the design loadings. The results are summarized in **Table 6.22**.

Table 6.22: Calculation of Remaining ESDs for Current Loading

Loading Parameter	90% Design Load (lbs/day)	90% Design Load (ESDs)	Current Loading[1] (lbs/day)	Current Loading[1] (ESDs)	Remaining Capacity (lbs/day)	Remaining Capacity (ESDs)
BOD_5	5,850	17,520	3,630	10,870	2,220	6,650
TSS	3,150	10,790 [2]	3,180	10,890 [2]	No Capacity	No Capacity
TKN	900	NA [3]	971	NA [3]	No Capacity	No Capacity

[1] As described in Table 6.14 and Section 3.
[2] Based on the Town's nominal concentration of 175 mg/L.
[3] No defined concentration for TKN (or other nitrogen species). Unable to calculate ESDs.

A similar procedure was conducted for future loadings projected through 2040. This is shown in **Table 6.23.**

Table 6.23: Calculation of Remaining ESDs for 2040 Future Loadings

Loading Parameter	90% Design Load (lbs/day)	90% Design Load (ESDs)	Future Projected Loading[1] (lbs/day)	Future Projected Loading[1] (ESDs)	Remaining Capacity (lbs/day)	Remaining Capacity (ESDs)
BOD_5	5,850	17,520	5,050	15,120	800	2,400
TSS	3,150	10,790 [2]	4,420	15,140 [2]	No Capacity	No Capacity
TKN	900	NA [3]	1,350	NA [3]	No Capacity	No Capacity

[1]As described in Table 6.1 and Section 3.
[2] Based on the Town's nominal concentration of 175 mg/L.
[3] No defined concentration for TKN (or other nitrogen species). Unable to calculate ESDs.

These calculations use the average BOD_5 load, which does not account for peaking that occurs at smaller time scales (months, weeks, or days). The design load could be exceeded on these smaller timescales.

This exercise could be conducted for TSS and TKN if the WRF capacity is re-evaluated and re-rated and if additional TSS and TKN loading capacity then remains. TSS loading for 1 ESD would be based on a similar calculation but using the Towns defined concentration for TSS of 175 mg/L (used for ESD calculation purposes). No average or nominal ESD load is currently defined by the Town for TKN.

6.5 Conclusions and Next Steps

This section provides conclusions based on the comparison of the current loads to the design capacity of the secondary treatment system. To address areas where BOD_5, TSS, TKN, and flow approach the design loads, the following key elements should be considered:

1. Aeration System: improve oxygen transfer efficiency.
2. Secondary Clarifier: continue to improve sludge settling or construct third clarifier. Secondary Treatment: using the above data, re-evaluate the WRF secondary treatment capacity.

6.5.1 Aeration System

Based on population growth projections, the air demand calculations show that the existing blower design capacity can meet all anticipated future process air demands using two-basin operations. To improve the life, capacity, and efficiency of the aeration system, the followings concepts should be considered:

1. Bring Aeration Basin A on-line and in proper working order. Several design and construction considerations are desired for this to occur.
2. Design and construct a new selector box in Basin A to replace the current selector box which has structurally failed. The selector box is necessary to ensure good sludge settleability and proper conditions for nutrient reduction (nitrogen and phosphorus). The original selector box has been modified to improve nitrogen removal; however, phosphorus removal was not considered at that time. Selector box design and sizing should take into consideration both phosphorus and nitrogen removal under current and future flows/loads.
3. Provide monitoring and control equipment (e.g. modulating valves, or flow meters) to ensure that the flow rate of influent wastewater can be measured and controlled. The existing flow splitter does not perform optimally and leads to dissimilar flow between the two basins.
4. Provide monitoring and control equipment to balance or separately control RAS flows to each aeration basin. This is especially needed if the influent flow is not balanced between the basins. The current RAS systems contain flow meters; however, RAS flow control has been challenging.
5. Provide for measurement and control of air flow to each aeration basin. The current blower configuration does not allow for measurement of air flow rates, nor does the system allow for any control of air flow to either of the basins. Air flow to both basins is fed through the same manifold and it is assumed that equal air flow is provided to each basin. However, influent flow is not equally balanced between the two basins (as mentioned in Item 3). This leads to operational challenges in balancing the biological needs of the two separate basins. The Town is planning to install separate air flow meters for each basin during the summer of 2018.
6. Upgrade electrical feed service to be able to run all current and any future additional blowers.
7. Conduct a condition assessment and capacity evaluation of the aeration blowers with a manufacturer's technical representative to determine if blower rehabilitation or replacement is

required in the short term. Such a blower efficiency study would require electrical monitoring and air flow metering; the Town is currently planning to implement air flow metering during the summer of 2018 which will provide needed information.

8. Upgrade automation and controls associated with Biolac® process. Newer Biolac® processes utilize air flowmeters, automated dissolved oxygen and ammonia-based airflow control schemes. Nitrate monitoring should also be added to better understand nitrification and denitrification throughout the process. Also, if an external carbon source is ever needed for enhanced denitrification in the future, nitrate monitoring would provide feedback for efficient carbon dosing control.

9. Increase overall oxygen transfer efficiency by operating using two basins when influent loads dictate the need to do so and when F/M ratio allows. This may require adjustments to MLSS concentrations and raising or lowering aeration basin water surface elevations within the design parameters. Oxygen transfer efficiency and F/M goals will need to be balanced to ensure efficiency and good sludge settleability can be maintained.

10. Investigate with Parkson™ the possibility of using curtains to longitudinally compartmentalize the basins into two smaller cells. This would be in conjunction with the monitoring and control improvements highlighted in recommendations numbers 3, 4, and 5. Dividing the basins longitudinally will improve operational flexibility and allow operators to utilize a smaller basin for low flows.

11. If additional aeration capacity is required in the future, Parkson™ recommended that the 4-foot Biofuser assemblies can be replaced with 6-foot assemblies, for an increase of approximately 6 percent in aeration capacity. A more detailed analysis of this option is recommended to determine feasibility.

12. Another option for increased air flow, if needed, is to replace at least one of the smaller blowers with larger units. A detailed electrical analysis would be required prior to moving forward with this option.

13. Optimization of the anoxic zones using baffling to create distinct and more efficient anoxic zones within the Biolac® basins. Any potential design should consider localized trapping of foam that may be encountered.

6.5.2 Secondary Clarifier

Based on the review of historical TSS data, the existing clarifiers perform well despite poor sludge settleability. The current SOR and SLR are within normal ranges for extended aeration processes, and the SOR and SLR projections (Future 2040) fall in the middle to upper range of the typical thresholds.

Secondary clarifier capacity was estimated using operating data and state point analysis. Assuming the current SVI of 225 mL/g and an MLSS of 2,500 mg/L, the secondary clarifiers at the WRF should be sufficient for influent flow rates up to 5.5 MGD. The construction of a third clarifier would provide capacity for influent flow rates up to 7.5 MGD at these conditions. As shown in **Appendix E**, the maximum clarifier capacity is very sensitive to SVI and the WRF can realize a significant increase in

secondary clarifier capacity by reducing SVI of the MLSS. Therefore, prior to the design and construction of a third clarifier, Hazen recommends the following:

1. Consult with Parkson™ regarding modifications to the operation of the Biolac® process which could improve SVI.
    - Parkson™ typically sees SVI in the 75 – 125 mL/g range for the Biolac® process.
    - Continue maintaining SRT in the 30 – 35-day window.
2. Consult with Parkson™ regarding operation at decreased MLSS (e.g. 2,000 – 2,500 mg/L). **Appendix E** shows that this has a positive impact on maximum secondary clarifier capacity. Parkson™ has indicated that nutrient removal targets can be met at this reduced MLSS and the anticipated operating temperatures. Decreased MLSS can also help maintain a favorable future F/M ratio if and when two aeration basins are brought on-line. Reductions in MLSS should be done incrementally to ensure adequate treatment (e.g. nitrogen removal) can be maintained.
3. Conduct settling tests to determine site-specific sludge settling parameters. Sludge settling parameters vary significantly and the state-point analysis conducted for this evaluation uses typical values. The site-specific sludge settling parameters would enable a more accurate estimation of existing clarifier capacity and the effect of adding an additional clarifier. Parameters to be tested would include the settling properties of the sludge (zone and compression rates), flocculation parameters, flocculated suspended solids (FSS) and dispersed suspended solids (DSS), discrete settling parameters, and sludge volume index (SVI).
4. Develop a computation fluid dynamic model of the clarifiers to better quantify clarifier capacity and identify modifications to improve existing clarifier performance. Hazen has used this method in the past to improve existing clarifiers and defer or eliminate new clarifier construction.

If the WRF determines that an additional clarifier is required, consideration of head losses and flow balancing among the three clarifiers must be considered during preliminary design.

6.5.3 Continue to Evaluate Denitrification Efficiency

Parkson™ suggests that the Biolac® system operates best at a carbon to nitrogen to phosphorus ratio of 500:2:1. The main way that this ratio is adjusted is by adding a carbon source. Under the current operation of the Biolac® system, the WRF typically nitrifies fully and denitrifies such that the total nitrogen (TN) is below the effluent limits. Population growth is expected to result in additional influent TKN. Thus, in the future it may be necessary to add a carbon source (e.g. molasses) to achieve this ratio. The Town receives wastewater from local wineries. It is possible that winery wastewater, which is high in BOD_5 and low in TKN, can help the Town achieve better denitrification by acting as a carbon source, and increasing the carbon to nitrogen ratio. Though this source of BOD_5 is seasonal, it could help offset the amount of additional carbon needed for nutrient reduction when winery wastewater is available. It is recommended that the carbon to nitrogen to phosphorus ratio be evaluated periodically to determine whether additional BOD_5 would improve denitrification efficiency.

6.5.4 Re-Evaluate WRF Treatment Capacity with the NCRWQCB

The WRF is permitted to treat an ADWF of 1.9 MGD, though this could be increased to 2.25 MGD if the WRF is re-rated. Growth projections in the Town's 2040 General Plan suggests that the ADWF capacity may be reached by 2040. Growth through 2040 is expected to result in an increase in average dry weather flow to 1.9 MGD. The secondary treatment process was designed to treat an ADWF of 3.0 MGD. The Town should prepare to increase its permitted ADWF treatment capacity to 2.25 MGD to provide flexibility for the projected population growth. The Town will be required to show that it has sufficient reclamation and storage capacity to offset increased flow per the requirements in the NPDES permit. This is further discussed in **Section 8.**

6.5.5 Summary of Key Findings

The analysis determined that sufficient hydraulic capacity remains for projected flows through 2040 for the major unit processes. The aeration basins (together) have sufficient capacity to treat BOD_5 through 2040; however, based on the design loads the aeration basins have already exceeded 90 percent of the design loading for TSS and TKN. Although the WRF consistently meets effluent permit limits for Total Nitrogen and TSS while operating with one aeration basin, increased loads in the future could bring the WRF above the design capacity with respect to these constituents.

The Town should make changes to the aeration system that will improve aeration efficiency. The Town should continue to improve sludge settleability to improve secondary clarifier efficiency prior to engaging in construction of a new secondary clarifier. Denitrification efficiency should be evaluated periodically, considering possible sources of BOD_5 to maintain proper carbon to nitrogen to phosphorus ratios. Finally, steps should be taken to increase the WRF's permitted ADWF capacity to the WRF's design ADWF capacity to prepare for future flows. This is likely to include increasing recycled water storage capacity and irrigation areas to offset the increase in flow as discussed in **Section 8**.

7. Phosphorus Removal Strategy

The Town's current NPDES permit includes interim effluent limitations for total phosphorus (TP). The limitations are based on objectives that require there be no net loading of total phosphorus to the waterbodies of the greater Laguna de Santa Rosa watershed which includes Mark West Creek (MWC). The interim limits proposed are in the form of step-down limits to a zero-net phosphorus loading by October 1, 2021. The deadline for the first step of the reduction of limits is October 1, 2018. Additional discussion of interim step-down limits for phosphorus loading can be found in **Section 4** and **Table 4.3.**

The permit also requires that the Town complete a modernization and optimization study report that includes the following:

- The results, findings and recommendations of the treatment plant modernization and optimization study;
- An evaluation of other options for complying with the final effluent limitation for total phosphorus, if necessary including but not limited to: feasible source control and treatment improvements, pretreatment limitations, development and implementation of a nutrient offset program, water recycling efforts, storm water programs, nonpoint source assistance programs, and other water diversion programs such as the Geysers Recharge Project; and
- A proposed time schedule for completing any tasks not already identified in this Compliance Schedule.

The Town submitted the required report on February 1, 2016. The report was acknowledged and accepted by the NCRWQCB in May 2016.

To complete the study, a BioWin™ process model was developed for the WRF for the purpose evaluating phosphorus removal strategies. The process model was developed based on historical data and special sampling data (**see Appendix F**). Technologies for phosphorus removal were evaluated, and the results are discussed in this section. The evaluation determined that to meet these effluent limits the Town could employ a three-prong approach comprising effluent pond optimization, process changes at the WRF and purchasing nutrient offset credits.

This section provides a detailed summary of the various options contained in the February 1, 2016 report to the NCRWQCB and how these technologies can be used at the WRF to meet effluent phosphorus limits. The complete report that was submitted to the NCRWQCB is included as **Appendix F** to this Modernization Study Report. It should be noted that since the phosphorus report was completed, a capacity analysis review of WRF data from 2015 to 2017 was completed as part of the Modernization Study. As a result, the historical flows and loads and projections used in this section are similar but not identical to data in the rest of this report; however, the conclusions from the evaluation of the phosphorus reduction alternatives remain consistent.

7.1 Phosphorus Removal Alternatives

The impending permit limits require that the WRF incrementally reduce or offset the phosphorus load discharged to MWC, with 100 percent of the load to be either removed or offset by 2021 and thereafter. This section summarizes the evaluations of three strategies:

1. Optimization of effluent pond operation to reduce discharges to Mark West Creek,
2. Process changes that can enhance the WRF phosphorus removal rates, and
3. Purchase phosphorus credits or implement projects that obtain credits to offset phosphorus discharges.

The three strategies, when combined, create a phosphorus removal plan that can meet the effluent phosphorus step down limits. Potential strategies and combinations of strategies are discussed; overall seven potential alternatives were identified. Process changes were evaluated using the BioWin™ model. Details of the model development are provided in **Appendix F.** An initial evaluation of the alternatives based on economic factors (capital and operational and maintenance costs) is discussed in **Section 7.4**. The results of the evaluation are also summarized in this section.

7.1.1 Effluent Pond Optimization

One strategy to limit phosphorus loading is to optimize pond management to reduce effluent discharged to MWC, thereby reducing total phosphorus load discharged.

A spreadsheet-based water balance model developed by Brelje & Race (B&R) and reviewed by Merritt Smith Consulting was used to estimate recycled water production and disposition (and discharge to MWC in particular). The result of the model was the modified storage curve. This is further discussed in **Section 8**.

The analysis determined that discharge management is an important component of a phosphorus reduction strategy. A combination of effluent storage alternatives would reduce or eliminate discharge to MWC. Strategies to achieve this include modifying storage management to reduce discharge by approximately 8 MG per year, on average. The Town could negotiate with other agencies to request to utilize storage or Geysers pipeline capacity. The Town may also consider increasing effluent storage pond capacities. A detailed evaluation of these alternatives is provided in **Section 8**. Discharge management is typically limited by the ability to discharge to MWC. The Town is allowed to discharge to MWC between October 1 to May 14. The Town's discharge is limited to 10 percent of the creek flow for most of this period, and 1 percent of the creek flow during shoulder periods (October 1 – 30 and May 1 – 14). Windows of opportunity for discharging are limited by these requirements and the often-fleeting nature of MWC flowrates in the spring and fall.

The analysis assumes two ADWF conditions: the flow expected in 2018 and the flow expected in 2021. These represent the ADWFs projected when 10 percent and 100 percent reduction in phosphorus loading are required, respectively. The recycled water demand was not changed in the water balance model to match the ADWF increase from 2018 to 2021. Therefore, a disproportionate amount of the modeled incremental flow due to the planned ADWF increase would be discharged to MWC (ADWF would

increase 2 percent, but the water balance model-estimated MWC discharge would increase an average of 12 percent). This illustrates that adding recycled water capacity is an important consideration as ADWF increases.

It should be noted that the use of recycled water for irrigation is seasonal. Demand for recycled water vastly diminishes in the winter; thus, increasing the number of irrigation customers does not directly correlate to reduced discharge to MWC in the winter unless recycled water storage is added.

7.1.1.1 Additional Effluent Storage

Phosphorus loading is reduced when discharges to MWC are reduced. Thus, one strategy to reduce phosphorus loading is to store and reuse effluent for reclamation. For the purposes of this analysis, it was assumed that 160 MG of storage would be required to eliminate discharges to MWC. This is the current annual average volume of discharge to MWC (see **Section 8**). The stored water would need recycled water and irrigation users. The City of Santa Rosa currently has a shortfall in recycled water. The Windsor WRF could supply recycled water to City of Santa Rosa to offset this shortfall in the summer. The Town could pursue an interagency agreement. Alternatively, the Town could negotiate the joint use of ponds owned by other agencies, such as ponds owned by the City of Santa Rosa or the Sonoma County Water Agency. This is further discussed in **Section 8.**

Increasing effluent pond storage is the topic of **Section 8** where several alternatives are explored, including increasing flow to the Geysers pipeline, increasing the existing ponds storage capacity, use of ponds owned by other agencies, or a combination of these alternatives. Benefits and limitations of these alternatives are discussed in **Section 8**.

7.1.2 Water Reclamation Facility Process Improvements

While the pond management modeling accounts for high (wet), low (dry) and average precipitation years, actual discharge reduction will remain highly dependent on actual wet weather conditions during the permitted discharge season, October 1 through May 14; a large storm at the end of the season can require significant discharge even with a modified storage curve implemented. In anticipation that pond management alone will not address the effluent phosphorus discharge reduction requirements during wet weather years, process improvements at the WRF should also be considered to augment pond management reduction of phosphorus discharges. Note that the process changes discussed in this section will reduce phosphorus load to the effluent ponds and does not account for naturally occurring processes that effect phosphorus concentrations in the ponds.

This section evaluates five alternative process strategies for the WRF. Process changes considered include the following:

- Chemical Addition (for precipitation or adsorption of phosphorus)
- Enhanced Biological Phosphorus Removal (EBPR)
- Ultrafiltration
- Alternative Sludge Management Methods
- Resource Recovery (Struvite precipitation in a dedicated reactor)

7.1.2.1 Chemical Addition

Aluminum and iron salts are commonly used for the removal of phosphorus from municipal wastewater. The most prevalent chemicals used for phosphorus removal are aluminum sulfate, commonly referred to as alum, and ferric chloride. Several other compounds can be used for phosphorus removal; ferrous sulfate and ferrous chloride (often referred to as "pickle liquor", a byproduct of steelmaking), sodium aluminate, and polyaluminum chloride (PACl). Proprietary alum blends are also available and show appreciable phosphorus removal. A number of treatment plants were originally designed for phosphorus removal through lime addition; however, this is no longer commonly practiced due to the substantial sludge production and operational and maintenance issues associated with lime handling.

Ferrous sulfate and ferric chloride can contain large quantities of hydrochloric or sulfuric acid, which can decrease alkalinity and depress pH. Alum addition will also consume alkalinity, resulting in pH depression. In contrast, sodium aluminate adds alkalinity and will increase pH. The impact of PACl on alkalinity is dependent on the product used but is generally less than that of alum or ferric chloride. Chemical costs vary depending on the supplier and the exact product used; generally, sodium aluminate and PACl are expected to have higher costs than alum.

For this study, the chemical phosphorus removal analysis focused on alum as a potential metal salt to be used at the WRF. The facility currently doses an alum–polymer blend product (a.k.a. alumer), C-2005, supplied by SNF Polydyne Inc. to the tertiary treatment facilities prior to flocculation tanks and AWT clarifiers to settle out remaining suspended solids. To a degree, alum addition for phosphorus removal should be able to be implemented with only small changes to the WRF infrastructure and operations. In addition, alum is widely used in the wastewater industry and is generally a cost-effective chemical in northern California.

The extent of phosphorus removal is dependent on many variables including the concentration of phosphate in solution, chemical dose, chemical age, pH and mixing intensity at the application point.

Chemical feed requirements and operational costs of implementing chemical phosphorus removal were developed based on the flows and secondary effluent phosphorus concentrations presented in **Table 7.1**. Secondary effluent TP concentrations are based on historical data available at the time of the February 2016 report to the NCRWQCB. Since reported secondary effluent TP and PO_4-P concentrations were essentially identical, it is assumed for the purposes of this evaluation that the evaluated secondary effluent TP is soluble and can be removed chemically. More detailed and more recent influent and effluent phosphorus data can be found in **Section 3**.

Table 7.1: Influent and Secondary Effluent Total Phosphorus

Parameter	Current Annual Average	Design	Build Out Condition (2045) Annual Average
Flow (MGD)	1.74	2.25	2.27
Influent TP (mg P/L)[1]	8	8	8
Secondary Effluent TP (mg P/L)[1]	6.5	6.5	6.5

[1] Estimated from October 2014-July 2015 historical data and special sampling results. Recent operational changes, mainly the operation of only one aeration basin, have resulted in lower secondary effluent TP concentrations (generally between 3 and 4 mg/L). For the purposes of this evaluation, the values in this table are used as a conservative estimate.

Table 7.2 summarizes the expected operational parameters and effluent quality assuming 48 percent (by weight) liquid alum feed at the advanced wastewater treatment system (AWT) influent (after secondary treatment) to attain the desired effluent ortho-phosphate concentrations.

Table 7.2: Design Criteria Used in Phosphorus Alternatives Analysis

	Current Conditions	Permit Step-down Reductions			
		10%	33%	66%	100%
Effluent TP Target Concentration (mg P/L)	2.75 [1]	2.48	1.84	0.91	~0
Current Annual Average Flow = 1.74 MGD					
Alumer Dose (mg/L)	54	58	67	83	127
Alumer (lbs/day)	148	158	184	229	347
Estimated Additional Sludge Production (lbs/day)	0	305	353	435	619
Build Out Condition (2045) Annual Average Flow = 2.27 MGD					
Alumer Dose (mg/L)	54	58	67	83	127
Alumer (lbs/day)	192	206	238	297	450
Estimated Additional Sludge Production (lbs/day)	0	395	458	565	803

[1]Based on NCRWQCB January 14, 2016 letter

The limit of technology for chemical phosphorus removal with the current tertiary solids removal facilities at the WRF is estimated at approximately 0.08 – 0.1 mg P/L based on experiences at facilities with a similar level of tertiary treatment. This will adequately meet the permit required effluent step-down limits until a net zero load requirement is in effect. Chemical sludge (sludge produced from chemical addition) will increase the total mass of sludge stabilized at the WRF. Optimal chemical removal of phosphorus may require upgrades to the chemical feed, additional chemical storage, additional mixing equipment, additional dose points/facilities (such as sludge pond supernatant), and potential changes to sludge handling operations (such as increased solids removal frequency).

Full scale testing was conducted in November 2015 at the WRF to demonstrate the ability of the WRF to reach effluent TP limits of 1 mg-P/L with the existing chemical dosing and solids removal facilities. Although it was demonstrated that the WRF was able to reach effluent TP limits of 1 mg-P/L, chemical doses used during full scale testing were higher than what was predicted due to system inefficiencies. There are opportunities to improve the system that would provide redundancy and chemical cost savings. Opportunities for improvement include providing better rapid mixing, redundant chemical feed pumps, and additional dose points (sludge pond supernatant and raw influent – these likely require new facilities). These are further detailed in **Section 7.3**.

7.1.2.2 Enhanced Biological Phosphorus Removal

Enhanced biological phosphorus removal (EBPR) systems typically include an anaerobic reactor prior to aerobic zones. The anaerobic reactor acts as a selector for phosphate accumulating organisms (PAOs). Phosphorus is taken up during cell growth of most microorganisms in wastewater treatment, but PAOs are classified by their propensity to accumulate significantly more phosphorus than the ordinary heterotrophic organisms (OHOs). This is typically referred to as "luxury uptake." As PAOs are removed from the system through sludge wasting, the phosphorus assimilated into their cells is also removed.

EBPR installations designed with adequate anaerobic zones and aerated zones that do not promote re-release of orthophosphate from PAOs routinely achieve secondary effluent orthophosphate concentrations of < 1 mg-P/L.

Implementing EBPR at the WRF will require construction of an upfront anaerobic zone prior to aeration. Note that while the WRF has an upfront anaerobic zone (selector box) in each aeration basin, they are not specifically sized for PAO growth and one of the two selector boxes (Basin A) is currently in poor condition. Mixing equipment will need to be installed in a PAO-optimized selector box to maintain solids in suspension. It is imperative to limit dissolved oxygen and nitrate introduction (via the RAS) to the anaerobic zone to maintain proper anaerobic conditions. Increasing the carbon available for EBPR is key to a reliable, stable EBPR process that can achieve low secondary effluent phosphorus concentrations which can help limit the amount of chemical coagulants needed downstream.

Other important criteria in establishing reliable EBPR include the following:

- Providing a sufficient aerobic hydraulic retention time (HRT) for phosphorus uptake (minimum of six hours)
- Providing sufficient dissolved oxygen within the aerobic zone to promote phosphorus uptake and limit phosphorus release
- Maintaining pH of 7 or greater in the aerobic zone
- Avoiding anaerobic environments after the initial anaerobic zone (for instance, in secondary clarifier solids blankets) where phosphate would be re-released – note this contradicts the on/off aerobic nature of the Biolac® process
- Limiting phosphorus return from solids storage recycle streams

Oxic zones are those with sufficient dissolved oxygen concentrations such that it is the primary terminal electron acceptor in microbial metabolic processes. An anoxic zone is one in which nitrite/nitrate is the primary electron acceptor. In an anaerobic zone, both oxygen and nitrite/nitrate are depleted, and alternative electron acceptors are used. Common EBPR configurations used at municipal wastewater treatment plants are listed below

- Anaerobic / Oxic Configuration (AO)
- Anaerobic / Anoxic / Oxic Configuration (A2O)
- University of Cape Town Configuration (UCT)

The anaerobic-oxic (AO) configuration (shown in **Figure 7.1**) is a plug flow system where influent and RAS enter an anaerobic zone prior to an aerobic (also called oxic) zone.

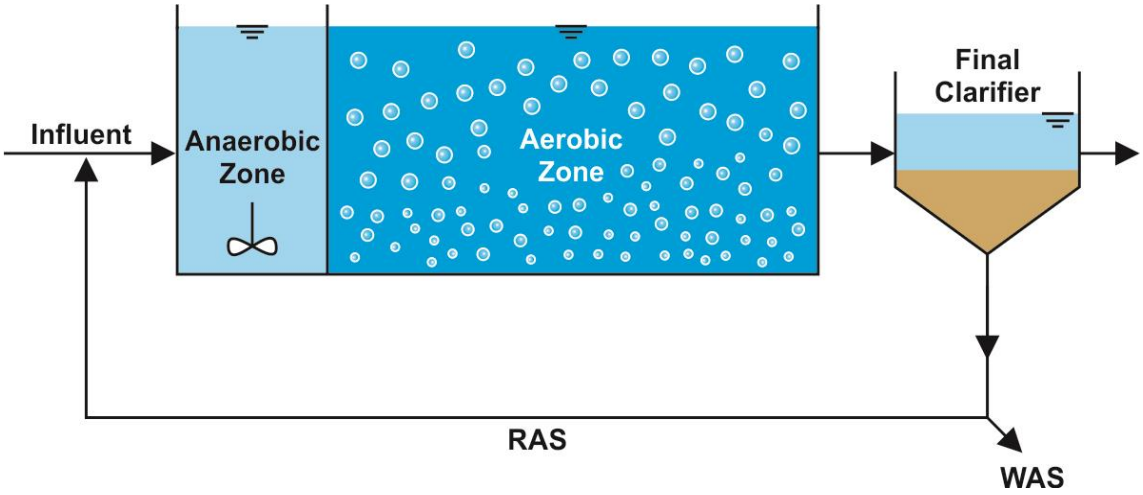


Figure 7.1: Anaerobic-Oxic Configuration (AO)

The anaerobic-anoxic-oxic (A2O) configuration (shown in **Figure 7.2**) is a plug flow system where influent and RAS enter an anaerobic zone followed by an anoxic zone and aerobic zone.

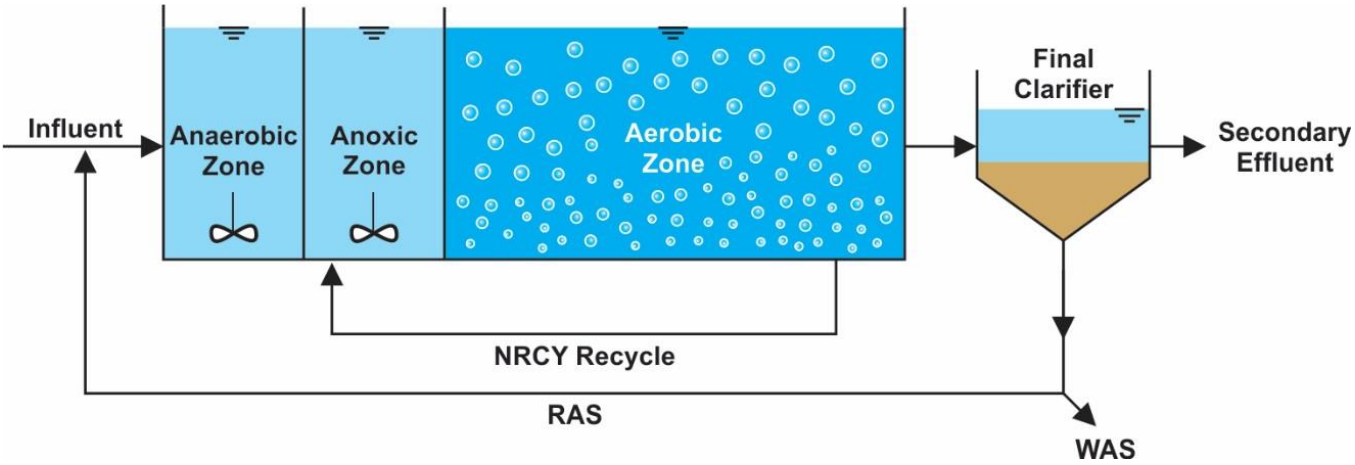


Figure 7.2: Anaerobic- Anoxic-Oxic Configuration (A2O)

Nitrified recycle streams (streams that contain nitrate) are shown as NRCY, and anoxic recycle streams are shown as ARCY. The University of Cape Town configuration (shown in **Figure 7.3**) is a plug flow system, similar to the A2O configuration where influent enters an anaerobic zone followed by an anoxic zone and aerobic zone.

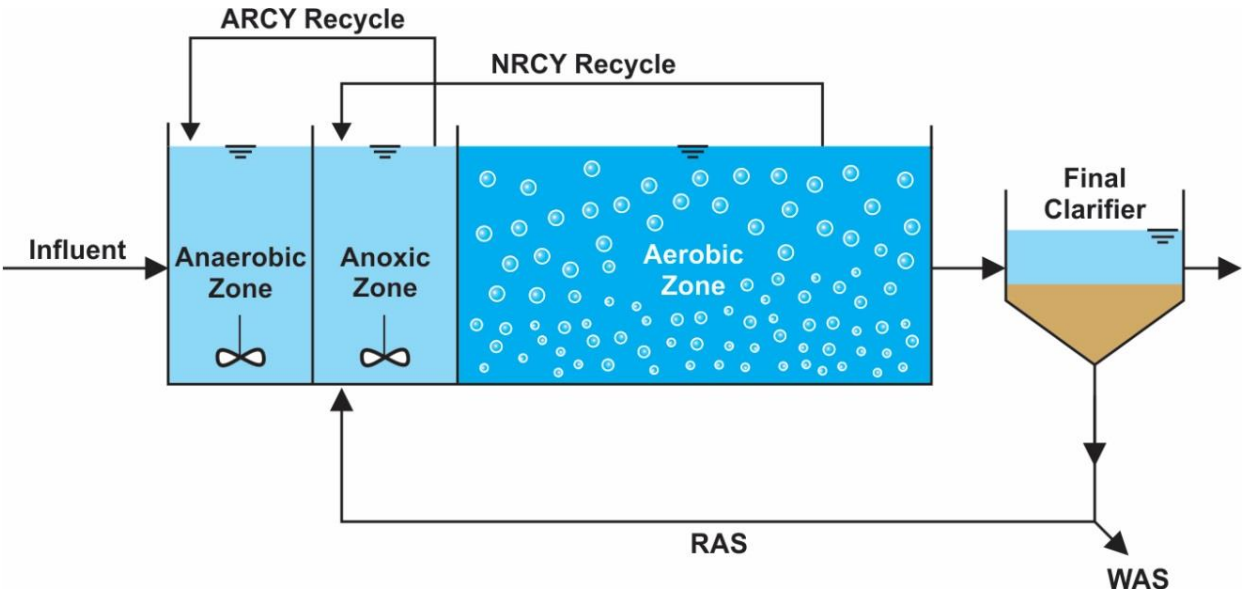


Figure 7.3: University of Cape Town Configuration (UCT)

A conventional AO process could be implemented at the WRF by incorporating an anaerobic zone in the upfront portion of the Biolac® basins, upstream of the basins or with a new aeration tank. However, there are several main challenges to effectively implementing EBPR at the WRF given the current unit processes at the facility. These include the following:

- The current selector boxes are not designed (sized) for optimization of PAO growth.
- The existing anaerobic zone exhibits back-mixing which likely decreases the anaerobic capacity for phosphorus removal.
- The long aerobic SRT of the Biolac® system (approximately 30 days) will result in biomass re-release of orthophosphate decreasing efficiency of the Bio-P system.
- The long SRT can promote the growth of glycogen accumulating organisms (GAOs), which compete with the PAOs for volatile fatty acids in the anaerobic zone.
- Operation at low to zero DO in sections of the Biolac® basins will reduce aerobic uptake of phosphorus, limiting the overall efficiency of the EBPR process. Low DO may also lead to secondary phosphorus release.
- One characteristic of the Biolac® basins is that they are not perfect plug flow reactors. This may limit the ability to remove phosphorus to low levels biologically.

- The long solids retention time in the sludge ponds leads to decay/lysis, leading to release of orthophosphate, which will be returned to the WRF influent. Thus, any gain in EBPR would be at least partially defeated by the long SRT in the sludge ponds.

Various options for implementing EBPR at the WRF were evaluated including:

1. Un-baffled In-Situ Anaerobic Zone: Turning off the upfront diffuser chains to create an in-basin anaerobic zone. Use intermittent air pulsing for mixing
2. Baffled In-Situ Anaerobic Zone: Installation of a concrete baffle to develop a dedicated anaerobic zone
3. Improving Selector Boxes or including a Separate Anaerobic Zone: Installation of an anaerobic selector upstream of the Biolac® basins. Installation of the anaerobic selector can be coupled with improved flow splitting to the aeration basins.
4. Conventional Aeration Tank: Replace one Biolac® basin with two aeration tanks each with 2 MGD capacity

7.1.2.3 Ultrafiltration

Ultrafilters have a nominal pore size of approximately 0.02 µm. To provide phosphorus removal these filters can be coupled with chemical addition systems. Chemical addition is followed by ultrafiltration to remove precipitated phosphate containing particles or particles that phosphorus has absorbed to. Providing an ultrafiltration unit process at the WRF will enhance the solids separation capabilities. The limit of technology for chemical addition and ultrafiltration is an effluent TP concentration of 0.05 mg-P/L. Ultimately the incremental improvement over the existing solids separation technology at the WRF, coagulation – flocculation, tertiary clarification and sand filters (0.1 mg-P/L), will need to justify the estimated costs to add an ultrafiltration unit process. **Section 7.4** provides conceptual estimates for adding an ultrafiltration unit process to the WRF.

The location in the treatment process should be evaluated. An ultrafiltration step could be included either before UV channels or after the storage ponds. Treatment of pond effluent that is going to MWC would likely be the most economical alternative, since only the flows that were to be discharged to MWC would need to be treated. If pond effluent is treated, ultrafiltration would help address other effluent compliance issues such as a potential pathogen TMDL. Solids from the filter reject stream would likely be returned to the head of the plant, however this would need to be evaluated. If the solids are returned to the head of the plant, these solids would eventually be separated out by the secondary clarifiers and removed as waste sludge. The possibility of returning the recycle stream to the AWT system could also be analyzed. With this option the loading from the filter reject stream wouldn't present an additional load to the secondary treatment system, given that the secondary treatment system is approaching its capacity (see **Section 6**).

7.1.2.4 Alternative Sludge Management

Whole plant sampling at the WRF showed that the sludge pond supernatant, which is returned to the influent waste stream, contributed a significant phosphorus load to the system. This is typical of recycle streams in wastewater treatment plants. Thus, alternative sludge management practices that eliminate or

reduce recycle stream phosphorus loading may be viable options for phosphorus reduction at the WRF. Several alternative sludge management processes are evaluated in **Section 11** with the main goal primarily to reduce the amount of solids that need to be removed from the WRF each year. A few of these processes may also have beneficial impacts to phosphorus in the recycle stream that re-enters the plant.

Sludge drying beds are an alternative management strategy that many plants employ. This process typically involves thickening of sludge and desiccation via drying beds. The mechanical thickening of WAS and tertiary clarifier sludge will create a filtrate or supernatant that will either be returned to the plant or stored in a settling pond. While the impact of recycle streams with high nutrient content can be minimized with the implementation of sludge drying beds (because phosphorus stays with the drying sludge), sludge drying beds are not recommended for the WRF because of the substantial footprint required and the odors that would be created with this option.

Anaerobic digestion a common sludge management option at WWTPs. It will not reduce the phosphorus load in the recycle stream. In plants that utilize anaerobic digestion for sludge stabilization, recycle streams can represent about 40 percent of the plant influent TP load. Implementing anaerobic digestion alone will likely exacerbate the phosphorus discharge at the WRF as the recycle stream phosphorus concentration will increase. In digestion, the breakdown of biomass results in phosphorus release. The phosphorus release in an anaerobic digester recycle stream would likely be greater than in sludge stabilization ponds due to increased breakdown of biomass. With the high concentrations in the recycle stream, however, this option does provide a potential for resource recovery. The resource recovery option is described in **Section 7.1.2.5**.

Mechanical thickening, such as gravity thickeners, gravity belt thickeners centrifuges, or rotary drum thickeners, can be used to thicken WAS upstream of the sludge stabilization ponds to increase pond capacity. While filtrate from these thickening processes is returned to the head of the plant for treatment, this stream does not typically have high nutrient loading as there is little to no time for degradation in the thickening processes. Mechanical thickening followed by pond stabilization will likely not provide a benefit for phosphorus removal as the sludge pond supernatant will have a similar phosphorus load compared to the current recycle stream. Instead, immediate disposal of thickened WAS (TWAS) would eliminate post stabilization recycle streams that have high nutrient loads because there would be no pond recycle stream. However, disposal of unprocessed solids, TWAS, would still require further processing and WRF operations would need to handle more frequent trucking and additional operation and maintenance of the thickening process infrastructure

Of these alternative sludge management strategies, only sludge drying beds and mechanical thickening with immediate disposal of thickened WAS (TWAS) reduce the phosphorus concentration of the recycle stream. Other methods of reducing the phosphorus concentration in the recycle stream should be explored. This could involve side stream treatment by chemical addition. This is further discussed in **Section 13**. The following section analyzes resource recovery for the recycle stream.

7.1.2.5 Resource Recovery

Resource recovery is the purposeful precipitation of struvite in a dedicated reactor for harvesting and use as a fertilizer. Struvite is a chemical compound containing magnesium, ammonium, and phosphate. With

strict nutrient removal requirements and an abundance of ammonium and phosphate, WWTPs can be advantageous places to implement struvite precipitation.

At plants with both EBPR and anaerobic digestion, recycle loads can represent 40-50 percent of the influent nutrient loading to the facility. Resource recovery can reduce recycle TP load by as much as 85 percent while recovering struvite for beneficial use. With this additional phosphorus sink, struvite fertilizer, effluent TP is reduced. This is a green technology that can be a source of revenue for a facility while also improving effluent quality.

The WRF does not currently have anaerobic digestion or solids dewatering facilities. Recent additional sampling showed that the sludge pond supernatant had relatively low TKN concentration, approximately 15 mg-N/L, but high phosphorus concentrations, 40-60 mg-P/L. This ratio, typical for aerobic digestion and sludge stabilization, is not favorable for struvite recovery. Resource recovery (struvite production) could become a viable option for phosphorus reduction should the Town ever decide to implement anaerobic digestion and sludge dewatering. It should be noted that the timeline for implementation of anaerobic digestion and dewatering would likely exceed the step-down limit schedule imposed by the NCRWQCB.

7.2 Phosphorus Removal Credits

Under the current permit, the Town's discharged phosphorus load may be offset with phosphorus removal credits. The NCRWQCB approved the Water Quality Trading Framework (Framework) for the Laguna de Santa Rosa on July 11, 2018, which describes the procedure for determining the amount of credit needed and acquiring such credits. The amount of credit needed is based on the actual amount of phosphorus discharged plus the amount needed to satisfy credit ratio requirements which can increase the total amount of credit needed by up to a factor of up to 2.5. Phosphorus removal credits would be generated by implementation of best management practices or improvement projects that reduce phosphorus load discharged by other local dischargers within the Laguna de Santa Rosa and the MWC drainage area. The value of the credit would be expressed in pounds of phosphorus and would be determined by the NCRWQCB based on a proposal from the Town and documented phosphorus removal.

Ultimately, to reach a zero-discharge phosphorus limit, credits will have to be acquired because existing treatment technologies and pond optimization (excluding complete effluent storage) cannot completely remove all phosphorus loads for all weather conditions. Any phosphorus removal plans to achieve zero phosphorus discharge will therefore need to be supplemented with some level of credits.

Two types of offset projects could be available to the Town as follows:

1. Credit Purchase. The Town would buy credit from another entity such as a landowner that has obtained prior approval and completion of a project for credit, or from the City of Santa Rosa when it has excess credits. For example, the City of Santa Rosa has already completed its own projects and banked phosphorus credits. The market for phosphorus credits has only been partially developed. The market is expected to develop as additional suppliers and purchasers enter the market (see *Water Quality Trading Framework for the Laguna de Santa Rosa Watershed, California*, prepared by Sonoma Resource Conservation District). Currently, the Town of Windsor and the City of Santa Rosa are the only two entities needing phosphorus

credits under the Framework. Recent examples of credit producing projects completed by Santa Rosa are listed below in **Table 7.3**.

Table 7.3: Phosphorus Removal Credits Purchased by City of Santa Rosa

Supplier	Description	Credits (lb-P)	Cost ($1000)	Unit Cost ($/lb)
Local Dairy	Manure management	7,600	$375	$49
Nature Preserve	Erosion control	10,960	$377	$34
Local Dairy	Manure Management	23,345	$320	$14

2. Credit Development. The Town would identify, obtain approval and implement its own projects to reduce phosphorus loads to the watershed.

Figure 7.4 shows an example of a credit project implemented by the City of Santa Rosa. The 177-acre Ocean View Dairy manure cleanup is funded by the City of Santa Rosa in exchange for 23,345 pounds of phosphorus credits.



Figure 7.4: Oceanview Dairy Phosphorus Removal Credit Project (Source: The Press Democrat May 6, 2015)

Recent purchases of phosphorus credits show that the unit cost of the credits is highly variable. The cost of the credits would be linked to market conditions as the Town will be competing with other agencies for the purchase of credits. Based on prior market conditions, the estimated cost per lb-P for credits is approximately 20-50 $/lb-P depending on size of the project.

Apart from the per-pound cost for compliance, disadvantages of relying on phosphorus credits include the following:

- Potential scarcity of available projects/credits: Projects may not be available for the Town at the time when needed and there may be an inherent limit to the credits that can be purchased as local sources for credits are spent. Furthermore, credits may not be available in the same watershed as the WRF, as required by the Framework.
- Expiration of credits: While the Town may save purchased credits not used during a discharge season for future seasons, purchased credits generally diminish and/or expire after a certain number of years (the current Framework expires credits completely after five years). Expiration of purchased credits would ultimately be a sunk cost for the Town.

At the present time, a phosphorus credit marketplace does not exist. The potential use of phosphorus credits to achieve permit compliance would subject the Town to market forces and effectively relinquishes at least some of the Town's control over its own compliance. Credit-generating projects could be developed by the Town. These would involve Town leadership and may require complex project components such as engineering and permitting. Alternatively, the Town could elect to utilize a more passive approaching in which credits are developed by others and sold to the Town. As a result, heavy dependence on phosphorus credits through a future marketplace or through direct purchase can be a risky venture for the Town and is not the preferred method. However, purchase of credits would need to be part of a plan to achieve zero net nutrient discharge because removing all phosphorus from effluent or achieving zero water discharge to MWC under all hydrologic conditions are technically infeasible alternatives when they are each considered alone.

One advantage of the credit purchase method for compliance is that credits can be purchased as they are needed instead of the Town having to invest the development of credits under the credit development approach described above. Regardless of how they are obtained, credits allow for immediate reductions in phosphorus load counted towards compliance. Provided that credits do not expire or only expire after a long period of time, the saving of credits from one discharge season to the next enables the Town to build a safety net for potential emergency discharges or process upsets.

7.3 Implementation Strategies

With the tools described in **Sections 7.1** and **7.2** the WRF can achieve the effluent step-down limits imposed by the NCRWQCB. This section provides a summary of the alternative implementation strategies that were evaluated to meet the step-down limits. Pond management/storage optimization is not included as a stand-alone alternative. It is pre-screened as a low-cost and prudent measure to reduce phosphorus loading to MWC by reducing the volume of effluent discharged and it is already being employed for that reason. Therefore, it is included as an aspect of all the forthcoming alternatives presented in this section.

Table 7.4: Alternative Phosphorus Reduction Strategy Advantages and Disadvantages

Alternative	Description [1]	Benefits	Disadvantages
1. Credit Intensive	Purchase or development of phosphorus removal credits	• Least capital intensive • Minimal operational changes	• Limited phosphorus credit availability • Costly phosphorus credits • Uncertainty in phosphorus credit acquisition and duration
2. Chemical Intensive	Chemical phosphorus removal, and purchasing of phosphorus removal credits	• Minimally capital intensive • Minor operational changes • Easily implemented	• Increased chemical costs • Increased sludge production • Capital improvements may be needed • Phosphorus credits needed • Uncertainty in phosphorus credit acquisition and duration
3. EBPR Intensive	Biological phosphorus removal and purchasing of phosphorus removal credits	• Bulk of phosphorus removal done via EBPR that requires little maintenance • EBPR results in low operational costs/lb P removed • Decreased air demand (conventional aeration tank)	• Large capital investment • Footprint concerns (for external selector box option) • Increased process complexity compared to Biolac® system • Difficulty draining basins for construction • Impacts to denitrification • Need for alternative biosolids handing to prevent re-release of P • Phosphorus credits needed • Uncertainty in phosphorus credit acquisition and duration
4. Combination Approach	Combination of EBPR, followed by Chemical Removal and purchasing of phosphorus credits	• Combines the benefits of Chemical Intensive and EBPR Intensive Alternatives • Lower chemical costs than chemical intensive option	• Similar disadvantages as Chemical Intensive and EBPR Intensive Alternatives • Requires major capital improvements • Phosphorus credits needed
5. Ultrafiltration	Chemical precipitation, ultrafiltration and purchasing of phosphorus removal credits	• Operational flexibility by adding another solids separation technology • Multiple barriers to phosphorus removal provide redundancy • Least credit intensive as it achieves the lowest effluent TP	• Increased chemical costs • Increased sludge production • Capital investment • Operational complexity • Increased operational and maintenance cost of filters • Phosphorus credits needed • Uncertainty in phosphorus credit acquisition and duration
6. Additional Effluent Storage	Minimizes effluent discharges to Mark West Creek with pond management optimization and by using additional effluent storage	• Operational flexibility • May eliminate dependence on credits	• High capital costs • Cost of purchasing land and regulatory challenges permitting pond • Potential variability in customer demand of stored recycled water. • Storage needed to eliminate discharge is highly dependent on weather. Storage may be inadequate in wet years.

[1] All alternatives include effluent pond optimization, as this is recommended for all alternatives.

Figure 7.5 provides a summary of the alternatives and the target phosphorus concentrations considered for the WRF phosphorus reduction. It is estimated that by optimizing current unit processes the Town should be able to reduce effluent phosphorus concentrations to approximately 1.75 mg/L. This is 1 mg/L below the benchmark initial year phosphorus concentration of 2.75 mg/L. Other techniques will be required to further reduce phosphorus concentrations, as summarized in the **Figure 7.5**.

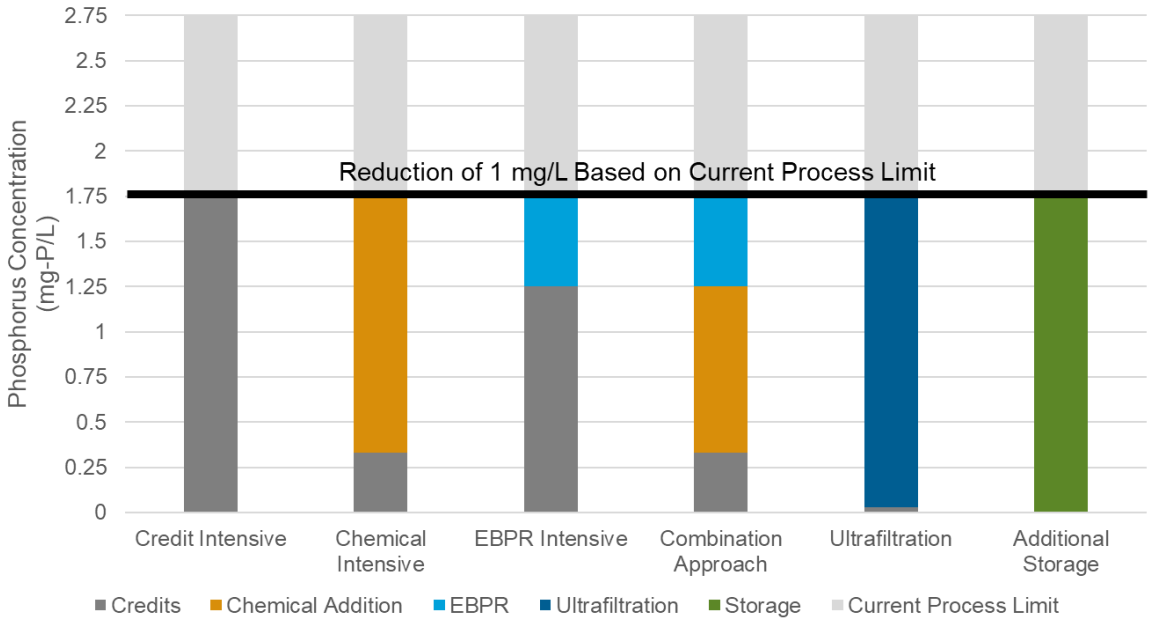


Figure 7.5: Alternative Summary and Limits of Technology for Effluent Phosphorus Removal

[1] *Note that the Additional Storage alternative meets the zero-loading limit only if adequate storage is available. If for a given wet year there is insufficient storage for zero discharge, the remainder would likely be offset with phosphorus credits.*

7.4 Comparative Costs

Capital and operational costs (incorporating only chemical, credit costs, and sludge hauling costs only) are summarized in **Table 7.5**. Purchasing of phosphorus removal credits is a tool that may be implemented relatively quickly (compared to other alternatives), provided a credit marketplace exists, to meet impending effluent TP limits. Credit purchase or credit development (via a project) to obtain phosphorus credits will be required to reach a zero phosphorus discharge limit. At the time of this study, the credit marketplace does not exist. In addition, credits may become scarce, the price of the credits could be unstable, and purchased credits may eventually expire. These are all reasons to limit the WRF's dependency on credits. To limit the WRF's dependency on credits, process changes and pond optimization are tools that can be employed with some certainty. These have been combined in various alternatives described in **Section 7.3** to meet WRF permit effluent step-down limits. The comparative costs of these alternatives are included in **Table 7.5**.

Table 7.5: Cost Estimate to Meet the Required Zero-Net Loading Compliance Schedule (2021)

Alt.	Description	Capital Costs ($M)	Chemical[1] Credit Costs[2] and Sludge Hauling [3] ($/yr)[4, 5]	2045 NPV[6] ($M)
1	Credit Intensive	$0	$548,000	$5.6
2	Chemical Intensive	$0.4	$226,000	$2.8
3	EBPR	$10.9	$299,000	$13.8
4	Combination	$11.8	$169,000	$13.5
5	Ultrafiltration	$26.2	$228,000	$28.7
6	Additional Effluent Storage[7]	$53	-	$53.0

[1] Alumer cost is based on the current price of $1.91/gal. Chemicals are applied during the discharge season only. Based on an assumed 2.75 mg-P/L baseline.
[2] Assumed credits available at $50/lb-P and the approximated annual average discharge to MWC is 160 MG. A ratio of 2.5:1 of lbs phosphorus credits purchased to lbs discharged was used. See Section 7.2.
[3] Assumed $1020/ dry ton hauled.
[4] Values are in 2022 dollars based on 3% annual inflation rate.
[5] Assumed WRF influent flows were updated from the original study (Appendix F) to be consistent with Section 3. Population growth was extrapolated through 2045 using the annual average growth rate of 1.39%. See Section 3.
[6] An interest rate of 5% was assumed.
[7]Assumes that all additional storage is in the form of new ponds, as this is the most conservative option. This estimate does not include transmission costs, land acquisition or environmental permitting.

The table shows a comparative cost of the capital and operational (chemical, credit and sludge hauling) costs associated with various alternatives. These net present values assume an interest rate of 5 percent. This high-level estimate illustrates the financial burden compliance can have on the Town and its rate payers. Heavily capital-intensive options such as Alternative 4 through 6 may have lower operational cost, but over time, these may not justify their initial capital purchase. Economically, Alternative 2 is the most viable.

7.5 Accelerated Phosphorus Removal

In the February 2016 submittal to the NCRWQCB, there was one additional alternative that was considered, but that was not accepted by the NCRWQCB. This alternative involved performing additional chemical phosphorus removal in the present to get ahead of the required step reductions and maintain an effluent TP concentration of approximately 1 mg-P/L for an extended period of time, or until a phosphorus TMDL can be established. This accelerated approach would result in the Town discharging a lower overall mass of phosphorus to MWC than provided by the initial interim step-down limits through the 2021 discharge season.

The accelerated phosphorus removal alternative was minimally capital intensive, easy to implement and required minimal operational changes; however, the alternative would have led to increased chemical costs and sludge production, which would lead to a decreased sludge pond capacity, and it may have required some minor capital improvements. The NCRWQCB did not accept the accelerated phosphorus removal approach.

7.6 Summary

7.6.1 Phosphorus Optimization Planning

As part of the Study process, on December 9, 2015, the Town met with NCRWCQB staff to discuss its ongoing efforts to comply with Waste Discharge Requirements (WDRs) for its wastewater treatment facility and the progress towards accomplishing nutrient removal compliance. The Town and the NCRWCQB staff discussed the benchmark that would be used for measuring phosphorus reductions. Additionally, the Town provided an overview of the phosphorus reduction options to meet the permit compliance schedule. The NCRWQCB responded to the Town's proposed approach in a letter dated January 14, 2016. In the letter, the NCRWQCB proposed a benchmark of 2.75 mg-P/L for the initial year phosphorus reduction baseline.

This study has indicated that several options would result in compliance with the phosphorus discharge step-down schedule. Alternative 2, the Chemical Intensive Plan appears to be the most agreeable approach for both the Town and the NCRWQCB and includes enhanced chemical addition. This approach was presented to the NCRWQCB in the February 1, 2016 Modernization and Optimization Study Report submittal and was accepted by the NCRWQCB in a May 2016 letter.

The approach to compliance includes the following steps.

1. Develop a plan for and conduct toxicity testing to identify impacts, if any, of alumer (and/or other potential chemicals used for phosphorus addition). This work has been initiated however additional work is needed (see **Section 7.6.2**).

2. Optimize storage by using the modified storage curve and refining the storage curves as needed. This may involve multiple model runs testing different shaped storage curves to determine which curve provides the lowest MWC discharge volume, while considering all operational constraints, across the range of hydrologic conditions.

3. Implement changes required for **Alternative 2 – Chemical Intensive** to meet the permit phosphorus reduction schedule. These changes should include adding apparatus for rapid mixing along with the potential for additional dose points, additional chemical feed pumps, and increased alumer storage facilities as needed. Switching from alumer to a more cost-effective alum solution can also be considered for this purpose.

4. Develop nutrient credit project such that sufficient credit is available to offset the quantity of phosphorus discharged after implementation of chemical removal. The needed timing of phosphorus credit depends on the timing of implementing the chemical removal option relative to the ramp down period of the phosphorus offset requirement. Only one of the two phosphorus load limit compliance strategies is needed initially, but both will be needed to comply with the eventual 100 percent load reduction requirement.

5. Participate in NCRWQCB's TMDL development process to anticipate and respond as appropriate to the timing of phosphorus load limits that may be different from those in the current permit.

The Water Quality Trading Framework for the Laguna de Santa Rosa Watershed was approved by the RWQCB in July 2018. The framework allows for banking of phosphorus credits between 3 and 5 years before they expire, depending on the project. Because of the framework's adoption, and the potential long lead time in performing its own credit development project, the Town should consider sources of phosphorus credits and prepare to bank them in anticipation of the upcoming October 2021 zero net discharge season.

7.6.2 Phosphorus Optimization Update

Since the 2016 Modernization and Optimization Study Report submittal, the Town has been completing the implementation plan to ensure compliance with the stepdown effluent phosphorus limits. The Town has been working to optimize the phosphorus removal first using biological processes (as evidenced recently by lower secondary effluent phosphorus concentrations footnoted in **Table 7.1**). Via full scale testing, the Town demonstrated that the WRF has an ability to remove phosphorus by 1 mg/L (from 2.75 mg/L down to 1.75 mg /L) using the existing infrastructure, thus phosphorus credits are not expected to be relied on until the October 2020 discharge season. The Town has already met the October 2017 discharge season phosphorus stepdown requirements using the existing infrastructure.

Chemical addition testing is forthcoming. A summary of the results of recent alum toxicity testing has been submitted to the NCRWQCB. During a full-scale test, at the maximum anticipated alum dose (greater than the "optimal" dose), the WRF suffered clogging of the sand filters as well as turbidity issues in the UV disinfection process. The analysis concluded that the testing may not have been representative of the tested alum dose due to less than ideal mixing with four mixers being out of service. The Town is in the process of systematically replacing faulty mixer gear boxes and is planning to install a static mixer downstream of the chemical feed pumps to better mix the alum prior to the flocculation tanks and the tertiary clarifiers. At such time that the mixers are established, a new optimal alum concentration for phosphorus removal can be determined. The optimum alum concentration should try to balance the effective operation of downstream processes with phosphorus removal.

A re-evaluation of the optimum dose based on newer secondary effluent concentrations and incorporating better mixing should be conducted. Following this, toxicity testing should occur. Lower alum doses may also need additional testing in case the optimal dose shows toxic effects.

In addition to the above other potential CIP improvements to reduce effluent phosphorus concentrations such as upgrading/optimizing the aeration basin selector boxes for better PAO growth and sludge pond supernatant chemical addition are also being considered by the Town for future implementation. These additional process improvements could potentially reduce the amount of alum needed to achieve similar phosphorus reductions outlined herein; however, phosphorus credits would still be needed in similar proportion to meet the zero net loading requirement in October 2021.

8. Effluent Storage

This section describes alternatives to increase effluent storage capacity or modify effluent pond operation to support an increase in the permitted flows to the Windsor WRF.

8.1 Background and Purpose

The purpose of this section is to evaluate alternatives that optimize reclamation and effluent storage usage to maximize five operational objectives:

1. Reduce phosphorus mass loading discharged to Mark West Creek per permit requirements.
2. Increase reclamation and storage to accommodate an increase in the WRF's permitted ADWF capacity from 1.9 MGD to 2.25 MGD.
3. Maximize recycled water consistent with PFS 3.6.
4. Minimize discharge to Mark West Creek.
5. Maximize system reliability for regulatory compliance and meeting recycled water demand through the appropriate combination of self-sufficiency and regional partnerships.

These objectives are in alignment with Town Council adopted reclaimed water priorities outlined in Section 8.2. The NPDES permit (Order No. R1-2013-0042) under which the Town of Windsor operates its reclamation system limits the quantity of recycled water that can be discharged to Mark West Creek (MWC) and the conditions under which it can be discharged. The reclamation system includes 143.7 million gallons (MG) of recycled water storage ponds, plus an additional 18.9 MG in high flow storage for a total of 162.6 MG. High flow storage is reserved to equalize peak wet weather flows.

The effluent storage ponds store recycled water to meet seasonal irrigation demands or discharge under appropriate conditions. The Town also provides recycled water to the Geysers Steam Field project according to the terms of an agreement with the pipeline owner, the City of Santa Rosa. The combined capacity of these reuse and discharge facilities is specified in Order No. R1-2013-0042 as 1.9 MGD. However, the design capacity of the water reclamation facility (WRF) specified in Order No. R1-2013-0042 is 2.25 MGD average dry weather flow (ADWF). Thus, reuse and storage limit the WRF's permitted capacity.

Storage capacity is considered the limiting factor for the reuse and disposal system and Section IV.C.3 in Order No. R1-2013-0042 states the following regarding storage capacity in relation to possible increases in permitted treatment capacity:

> "The Permittee shall maintain, at a minimum, a storage capacity of 149 million gallons and maintain the capability to irrigate 393 equivalent acres[1] per year at an average daily flow of 1.9 MGD. Prior to allowing an increase in the permitted water reclamation flows, the Permittee shall submit to the Regional Water Board and CDPH, an engineering report detailing modifications to the treatment and/or reclamation capacity. The engineering report shall demonstrate that the Permittee has increased its total storage capacity and associated irrigation areas in accordance with Table 8, in order to gain authorization to increase its average dry weather flow above 1.9 MGD (up to the Facility design capacity of 2.25 MGD)."

This section describes options available to increase the permitted ADWF to 2.25 MGD consistent with permit Section IV.C.2. The two broad categories under consideration are (1) addition of storage and (2) increase of flow to the Geysers. As this is a high-level analysis, a full engineering review of storage pond design, site selection processes and operations and maintenance costs are not considered as part of this report. The objectives for achieving the additional storage capacity include providing storage of recycled water in a reliable, practicable manner that encourages the best use of water resources, while protecting public health and the environment. The Town's overall goal is to use recycled water in a manner that maximizes reuse opportunities, particularly where recycled water will offset the demand for potable water supplies. Other supporting objectives include:

- Providing additional storage capacity in an economically feasible manner;
- Maximizing the use of existing infrastructure;
- Maintaining a reclamation storage system that is manageable and reliable; and
- Providing flexibility to coordinate the storage and transfer of recycled water with neighboring agencies as deemed appropriate by the Town.

8.2 Existing Storage System

The effluent storage ponds consist of three earthen ponds (Ponds S2, 5 and 7) that are located at the WRF site. Additionally, three earthen ponds are located on the northern edge of the Windsor Golf Club (Ponds 8, 9 and 10). Pictures of these ponds can be found in **Section 5, Figures 5.1** and **5.2.** These six ponds have a total storage capacity of approximately 143.7 million gallons. An additional 18.9 MG in untreated wastewater storage is available in the high flow storage ponds (Ponds 1 and 2). **Table 8.1** summarizes the Storage Pond Design Criteria.

The ponds are earthen basins with a 2:1 or 2.5:1 interior slope and a 4-inch layer of rock rip-rap on the interior slopes. The maximum water depth in each pond varies from about 10 feet to 14 feet with a typical minimum freeboard of 2 feet. The exception is Pond 7 which has a 3-foot design freeboard.

[1] An acre of land that uses 30 inches of irrigation water per season. For example, an acre of vineyard that uses approximately 5 inches per year while pasture or golf course turf uses approximately 30 inches per year. Thus, one acre of pasture or golf course is considered on equivalent acre, while six acres of vineyard would be considered one equivalent acre.

Table 8.1: Storage Pond Design Criteria

Pond Number	Normal Storage Capacity (MG)	Maximum Operating Water Depth (ft)	Minimum Freeboard (ft)
S2	6.9	12	2
5	14.4	12	2
7	78.5	14	3
8	12.0	10	2
9	15.1	10.3	2
10	16.8	10.3	2
Effluent Storage Subtotal	**143.7**	**-**	**-**
High Flow Storage Ponds (1 and 2)	18.9	Varies	2
Total	**162.6**	**-**	**-**

A schematic of the existing storage system is provided in **Figure 8.1**. For simplicity, the figure reflects typical operation of the reclamation storage system and does not show all connections and bypasses.

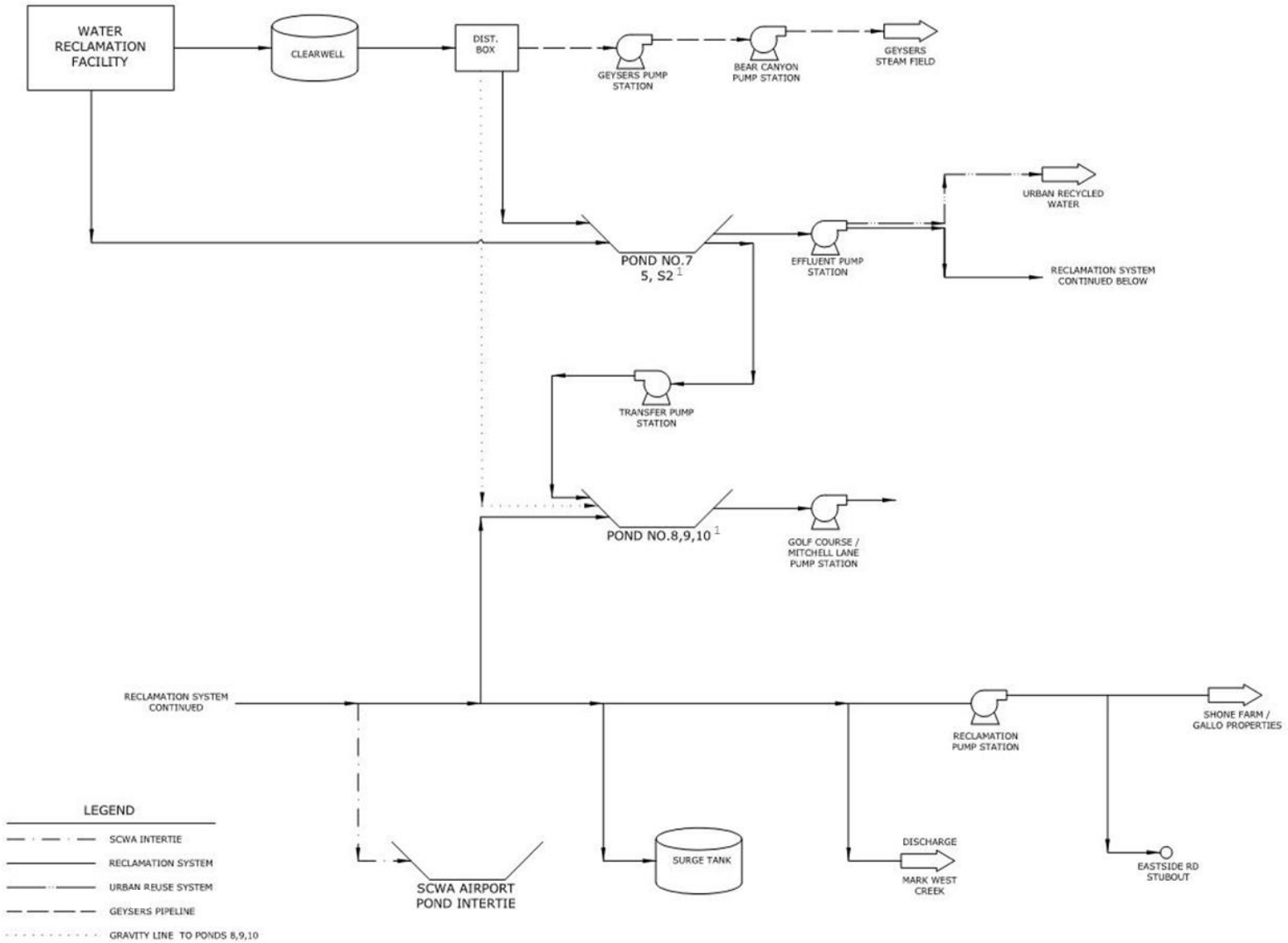


Figure 8.1: Simple Schematic of Existing Reclamation System Conveyance and Storage

[1] Pond Volumes are provided in Table 8.1.

Treated effluent from the WRF is either conveyed to Pond 7 or sent to the clear well and a distribution box, where it can be conveyed either to the Geysers Steam Field via the Geysers Pipeline or to any one of the effluent ponds. The typical operation of the reclamation system infrastructure and the major uses of WRF effluent are summarized below:

- Pond 7 is used year-round and is hydraulically connected to Ponds 5 and S2 which are used seasonally.
- The Effluent Pump Station is used to send Pond 7 water to urban and agricultural recycled water users or to MWC. The Effluent Pump Station discharges water to MWC through activation of the control valve located near the surge tank and is generally controlled via level sensors in the surge tank.
- The Geysers pipeline is typically fed directly from the UV channel through the clear well. Any additional flow that is needed can be taken directly from Pond 7. The mixing is performed in the distribution box which feeds the Geysers Pump Station.
- Ponds 8, 9, and 10 are off-site irrigation ponds, generally used only for the golf course. Pond 8 is the master pond and 9 and 10 are fed water from 8. Pond 8 is filled via the Transfer Pump Station which can draw water from the effluent pump station well, or directly from ponds S2, 5 or 7. During the summer Ponds 9 and 10 usually remain dry.
- The Transfer Pump Station can also move water among storage Ponds 7, S2, and 5.
- Urban and agricultural recycled water includes Town-owned irrigation fields and Town parks.
- Recycled water is provided to the High School and Fire Station #2 for flushing toilets and landscape irrigation. This water is treated by the No. 3 Water System (hypochlorite) prior to end use and is not from Pond 7.
- Shone Farm and Gallo properties are fed water via the Reclamation Pump Station.

The Town provides recycled water for irrigation of almost 570 equivalent acres of pasture, crops, vineyards, and urban parks, playgrounds and residential parcels. The Town Council-approved 2001 Water Reclamation Master Plan and adopted the following priorities regarding the use of the Town's reclaimed water:

- Priority 1 - Reuse within the Town limits or sphere of influence. Examples: irrigation of front and backyard landscaping in single family homes, Town parks, commercial and industrial landscaping, cooling system water and other nonpotable uses.
- Priority 2 - Agricultural reuse outside of Windsor utilizing both Windsor distribution system and the Geysers pipeline. Examples: vineyards, pasture lands, recreational lands and other non-potable uses along Eastside, Westside, Trenton Healdsburg Roads and any other agricultural lands along the Geysers pipeline route.
- Priority 3 - Reuse at the Geysers. Examples: when weather conditions preclude either irrigation or discharge to Mark West Creek water could be pumped up to the Geysers for reuse in the steam

fields. This may require the Town to contribute a steady minimum amount to the Geysers in order to make it cost effective for Calpine Corporation, with an opportunity to send more water to the Geysers under certain operating conditions when reuse is limited or constrained.

- Priority 4 - Discharge to Mark West Creek. Examples: every effort would be made to dispose of the Town's reclaimed water either through irrigation or pumping up to the Geysers. The Town's lowest priority would be discharge to Mark West Creek. Under any conditions, it is expected that the Town would continue to discharge a steady minimum amount to Mark West Creek each year.

Figure 8.2 shows the approximate distribution of the recycled water use from 2014-2017. Distribution varies from year to year, depending on annual rainfall. During drought years, it is possible that no water is discharged to MWC. Thus, irrigation becomes a much larger fraction of recycled water use.

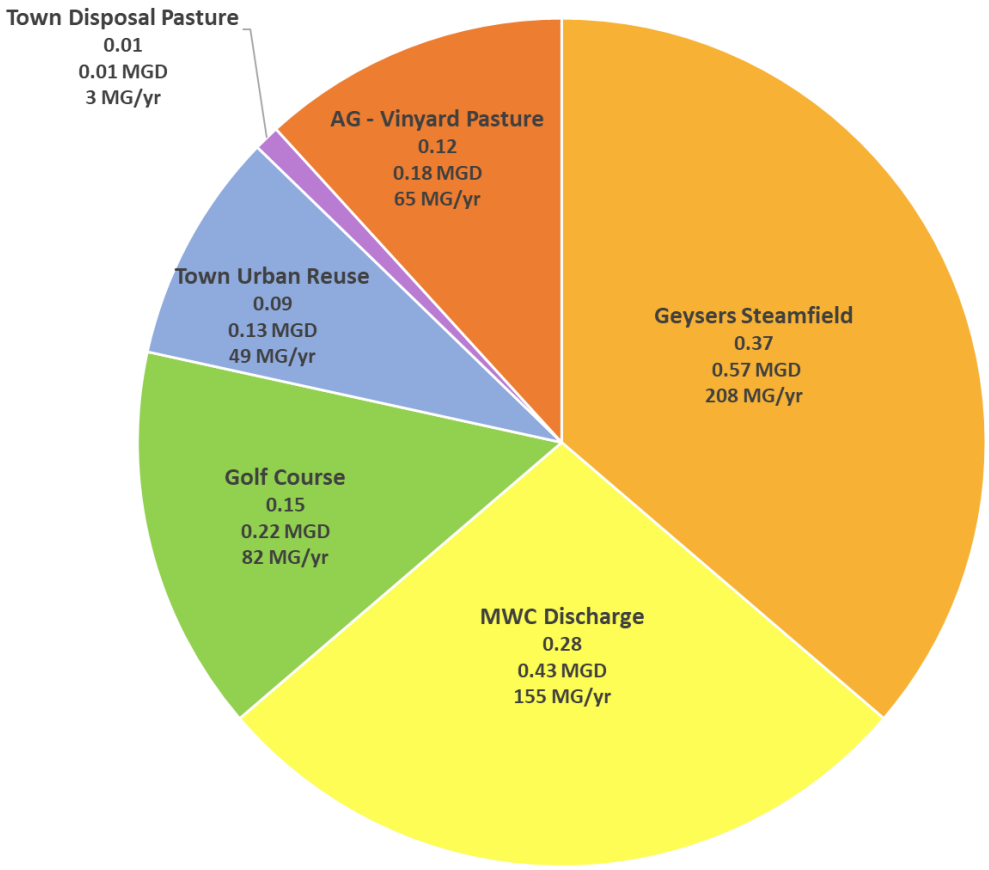


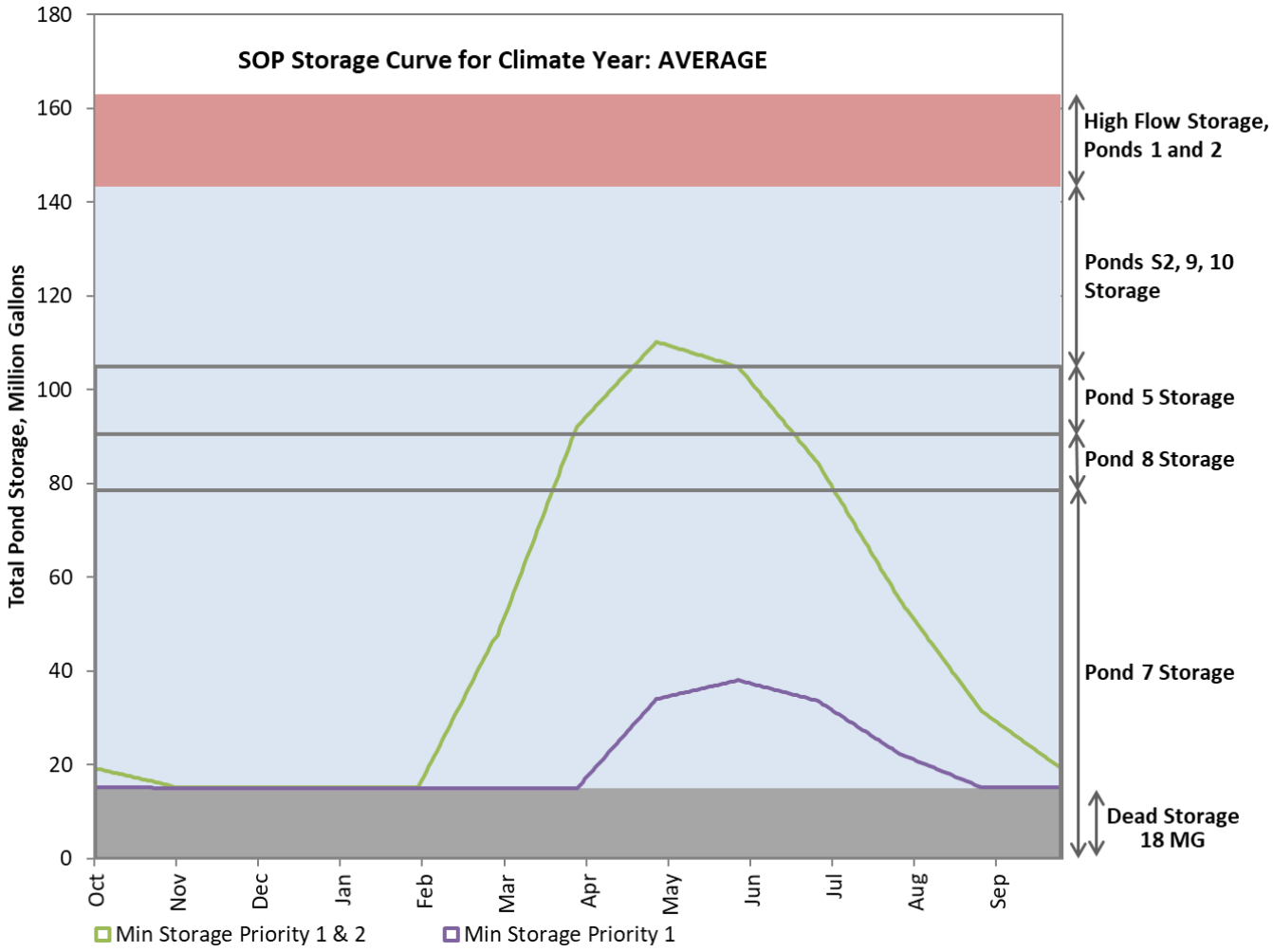
Figure 8.2: Distribution of Recycled Water Uses (2014-2017)

The drought summers of 2014 and 2015 likely skew the data set for **Figure 8.2**. An approximate rule of thumb for current normal year operations is: one-third of recycled water is used for irrigation, one-third is used for geysers recharge, and the final third is discharged to Mark West Creek.

The operation of the ponds is closely coordinated with the operation of the treatment, irrigation, and creek discharge facilities. Discharge to MWC is permitted only between October 1 and May 14 and is subject to

restrictions based on the conditions of the receiving waters. The flowrate of discharge to MWC is limited to 10 percent of creek flows from November 1 through April 30, and 1 percent of creek flows for October 1 through 31 and May 1 through 14. Operations staff keeps MWC flows below 1 percent during non-rain events throughout the discharge season.

Discharge to the Geysers Steam Field is permitted year-round. Because of this, the ponds, irrigation, and discharge facilities are operated on a yearly cycle that begins October 1 of each year. From October 1 to May 14 ("the discharge season"), the ponds are filled, leaving some of the existing storage reserved for a possible surprise rain event in May or June (which can depress irrigation demand and increase WRF flow) as well as to prepare for the increased storage needed when the Geysers discharge rate is reduced for the summer. This reserved storage could be thought of as a "safety factor." Water is discharged to MWC to avoid over-filling the ponds. From April to October, the WRF treated effluent is used for agricultural and urban irrigation and the ponds are generally drawn down to minimum levels by October 1. Ponds S2, 5, 9 and 10 are usually empty by the end of the summer. **Figure 8.3** shows the previous standard operating practice for water storage for an average rain year. Operations staff may deviate from this storage curve and discharge more flow to MWC to prepare for periods of significant rainfall or to plan ahead for periods in which more free storage is desired but future creek flows will prohibit significant discharge. Mark West Creek flows, and thus the discharge potential, are highly variable.



Priority 1 storage is for reuse within the Town limits or sphere of influence (as defined in the 2001 Reclamation Master Plan).
Priority 2 storage is for agricultural reuse outside the Town limits, utilizing the Town's distribution system and the Geysers Pipeline (as defined in the 2001 Reclamation Master Plan).

[1]Dead Storage is 10% of total storage. See Table 8.2.

Figure 8.3: Average Year Storage Curve

8.3 Water Balance Analysis

A spreadsheet-based water balance model developed for the Town by Brelje and Race was updated to estimate monthly recycled water production and storage needs. The full Brelje and Race report is included as **Appendix G**.

8.3.1 Description of the Model Development

The water balance model uses various inputs to model the conditions that impact the operation of the storage system. The model was used to illustrate operational changes that could reduce MWC discharges (Oct. 1 – May 14). The following water balance model input variables were updated for model runs:

- Average dry weather flow
- Frost protection
- Storage management (as displayed in **Figure 8.3**)
- Storage capacity

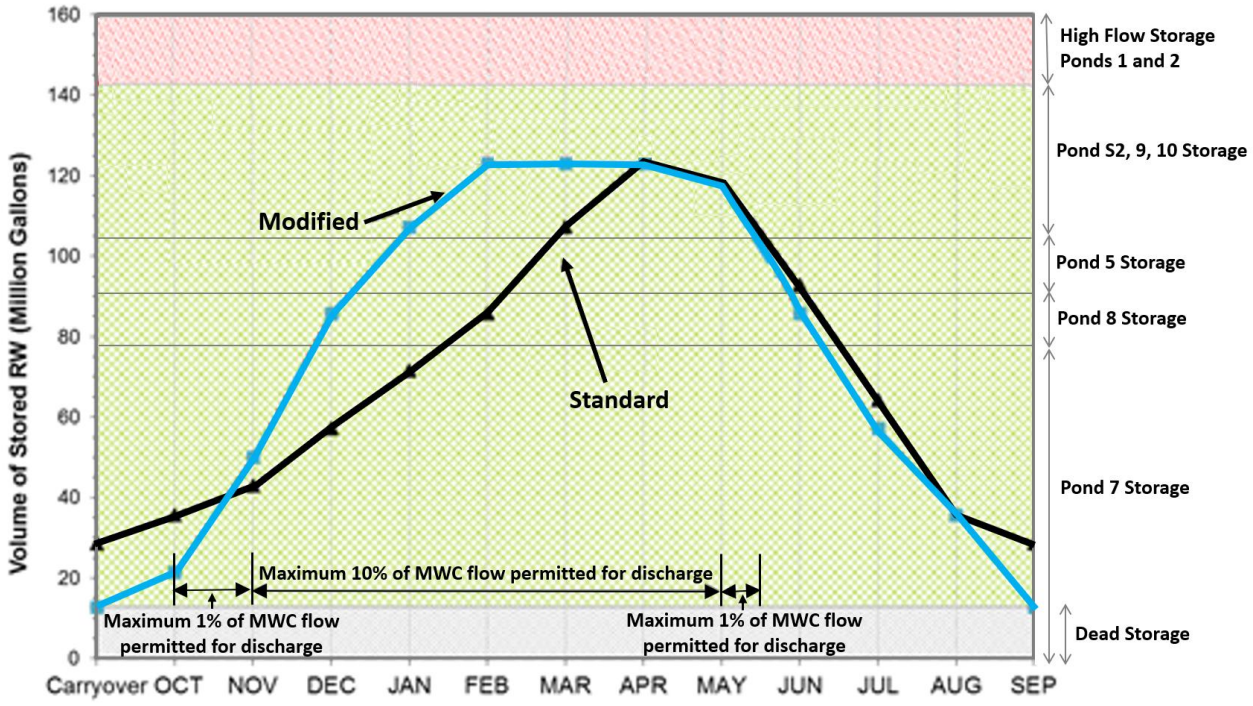
The major modifications included updating the Average Dry Weather Flow (ADWF) input values and irrigation distribution factors to reflect future projected operations. It was noted that vineyards and other irrigation users are turning to technologies that use significantly less water (or even no water at all) for frost protection. It was determined that the effects of recycled water use for frost protection were relatively minor and would not affect the findings of the analysis.

The water balance model was run for ADWF conditions anticipated in 2018 and 2021 when the 10 percent reduction in phosphorus mass loading and the 100 percent reduction are required, respectively. The variables of note that were adjusted in the analysis were climate conditions (wet, wet/dry, and dry rainfall conditions) and whether or not recycled water is made available for frost protection. The following combinations of major variables were evaluated:

- Wet climate conditions with recycled water used for frost protection,
- Wet/dry climate conditions with recycled water used for frost protection,
- Dry climate conditions both with and without the use of recycled water for frost protection.

Other hybrid combinations of hydrologic conditions were also evaluated, and historical data were considered, however the above bullets summarize the key combinations of variables. Storage management and capacity were manipulated to determine potential operational changes that could be made to reduce discharges to MWC. The transfer of water to outside storage or to the Geysers Pipeline was also modeled. In total, fifty-nine combinations of various inputs were evaluated.

Two different storage curves were assumed (standard, and modified storage curves), see **Figure 8.4.** The standard curve reflects previous operating procedures with the ponds filling in the spring. The modified curve (now generally being followed by Operations staff after this analysis was completed) is designed to hold more water in the ponds in October through May to avoid discharging during the shoulder months (when flow is limited to 1 percent of MWC flow). The modified curve assumes the high flow storage ponds are not used for storing effluent.



[1]*Dead storage is 10% of total storage. See Table 8.2.*

Figure 8.4: Modeled Storage Curves

Under the modified curve, less water is discharged to MWC overall, and thus nutrient loadings are diminished. The model determined that the modified storage curve would result in an average of 8 MG per year reduction in discharge to MWC. Because the Town is required to meet strict phosphorus limits (zero phosphorus load), offsetting any amount of discharge reduces the phosphorus loading sent to MWC. This is a key benefit of operating on the modified curve compared to the standard curve (previous operation). It should be noted that the Town's operation staff does not generally operate Pond 7 below about 30 MG; however, during the fall of 2018, to implement the more aggressive storage curve, the staff successfully brought effluent storage down to approximately 24.7 MG using irrigation on Town property prior to the wet season.

The model runs used in this analysis are based on total effluent storage volume of 129 MG, which was computed using storage volumes from the WRF O&M manual, which does not include Pond 5. Storage volumes have been modified in **Figure 8.4** to reflect the actual total storage of 162.6 MG (including high flow storage ponds) and 143.7 MG effluent storage (without the high flow ponds). It should be noted that operations staff generally maintain a margin of safety in terms of pond storage during the winter such that the maximum storage in winter is capped at 130 MG and the 143.7 MG is not realized. Recently a daily time-step water balance model was developed for the WRF under a separate project with the City of Santa Rosa. The model results are still under review by the Town, and the results of this revised water balance model have not been incorporated into the analysis in this section. Prior to modifying the graph to reflect revised storage, a decision should be made regarding the preferred model to be used for this analysis (Brelje and Race water balance model or the new, daily time-step model being prepared by Merritt Smith Consulting). It should be noted that operators have since revised the daily time-step model to better fit

their needs. The dead storage and the peaks were modified to provide a more conservative operational scheme.

Overall, the conclusions of the daily time-step model are consistent with this analysis. The anticipated reduction in discharge to MWC in the daily time-step model ranged between 7 and 11 MG per year. This is comparable to the 8 MG per year that was determined from the Brelje and Race water balance model.

8.4 Increasing Reclamation and Storage Capacity to Re-Rate the WRF

The total storage capacity of the WRF is approximately 162.6 MG (143.7 MG of reclaimed water storage and 18.9 MG of high-flow storage). The NPDES Permit issued by the NCRWQCB prescribes a minimum total storage capacity in order to re-rate the WRF for a new ADWF; these requirements are provided in Table 8 of Section IV.C.3 in Order No. R1-2013-0042 and recreated in **Table 8.2** below.

Table 8.2 NPDES Permit: Projected Storage Capacities for Reclamation System Based on Diversion Rates to Geysers Project

Treatment and Reclamation System Rated ADWF Capacity (MGD)	Minimum Total Storage Capacity (MG)[1,2]	
	0.53 MGD Diversion	0.75 MGD Diversion
1.6	149	149
1.7	149	149
1.8	149	149
1.9	149	149
2.0	165	149
2.1	165	149
2.2	196	149
2.25	207	165

[1] Dead storage was assumed to be 10% of total storage for all flow conditions. All scenarios provide 20 days of storage reliability under dry‐weather conditions.
[2] Total storage does not include County storage facilities totaling 50 MG that are available to the Town through 2014. Upon approval of the Joint Use Program, at least 50 million gallons of additional storage capacity is expected to be available to the Town.

Based on the storage volume of 162.6 MG, the NCRWQCB only allows for an ADWF of 1.9 MGD. A total storage capacity of 207 MG is required to re-rate the WRF to the design flow of 2.25 MGD ADWF (assuming a yearly average discharge to the Geysers pipeline at a flowrate of 0.53 MGD is continued). Thus, under the current permit 44.4 MG of additional storage is required to re-rate the WRF to match the design capacity of 2.25 MGD.

8.5 Reclamation and Storage Alternatives

The following alternatives were considered and are presented in the Alternatives Analysis **Section 8.6**. These proposed alternatives are not listed in a particular order and each will allow the Town to work towards various objectives such as reducing phosphorus mass loading discharged to MWC or increasing reclamation and storage capacity to increase the WRF's permitted ADWF capacity to 2.25 MGD. The list of Town's operational objectives is presented in **Section 8.1** and is used in the analyses in **Section 8.6.3**.

Alt. No. 1 Operational Modifications. This alternative is based on the results of the recently modified water balance model which suggests that the Town may be able to use a more "aggressive" approach to retaining recycled water. By doing this, the Town may be able to maximize existing reclamation storage system capacity. This alternative will not resolve storage issues related to re-rating the WRF; however, it will reduce discharges to MWC which will help reduce the WRF's phosphorus loading. The model results indicate that discharge to MWC can be reduced by approximately 8 MG per year, on average through the range of hydrologic years.

Alt. No. 2 New Storage Pond(s). In order to increase storage capacity in the reclamation system this alternative considers the potential for a new storage pond or ponds that total 44.4 MG. This alternative is analyzed using existing cost information from the Town's previous construction considerations as well as the City of Santa Rosa's Seasonal Storage Project. The Town had previously considered a site to construct new storage facilities (then called Ponds S, T1 and T2) on Town-owned properties along Eastside Road. Site studies and an Environmental Impact Report (2001) were conducted; however, to avoid extensive tree loss associated with such ponds, the Town Council directed the construction of the ponds be postponed as long as possible and encouraged the use of alternative ponds sites. The Town has since developed an agreement to supply effluent to the Geysers steamfields as part of the preferred alternative in the 2001 Water Reclamation Master Plan. The Geysers project has offset a portion of the Town's effluent storage needs.

Alt. No. 3 Increase Existing Pond(s) Capacity. A potentially efficient approach to increasing storage capacity is to increase the berm heights of existing ponds. Because all the pond edges are sloped, incremental increases in berm height can increase storage volumes considerably. Lateral expansion of the existing ponds would be moderately difficult due to constraints by surrounding neighborhoods, the creeks, and bordering railroad tracks.

Alt. No. 4 Joint Use of Ponds Owned by Other Agencies. This alternative considers that the Town may want to take a regional, cooperative approach to sharing storage with nearby agencies such as the Sonoma County Water Agency and the City of Santa Rosa. Both agencies have recycled water connections with the Town's system and sharing storage is a possibility already being considered among the three agencies. With this option, there may be permit issues to overcome regarding the potential discharge of "blended" reclaimed water.

Alt. No. 5 Increase Flow to Geysers. The Town currently exports an annual average flow of 0.53 MGD to the Geysers Steam Field via the Geysers Pipeline. The Town could increase its delivery schedule to the maximum delivery schedule (an annual average of 0.75 MGD). This has been accounted for in the Town's NPDES Discharge Permit (as seen in **Table 8.2**); increasing Geysers flow to the next tier (0.75 MGD) decreases the overall need for storage from 207 MG to 165 MG, thereby decreasing the NCRWQCB storage requirements by 42 MG. Under the increased Geysers discharge scenario, only an additional 2 MG of storage would be needed to re-rate the WRF to its 2.25 MG ADWF design treatment capacity.

Alt. No. 6 Combination of Alternatives. This alternative combines the above proposed alternatives to achieve a mix of alternatives that best suits the Town's needs.

8.6 Alternatives Analysis

This section presents the evaluation criteria, provides a comparison of the alternatives, and describes an apparent best alternative. The analysis included four main categories – economic, social, environmental and technical criteria - to evaluate the alternatives. These four categories make up the triple-bottom-line-plus analysis. Social, environmental, and technical criteria are discussed in **Section 8.6.1**. A discussion of major assumptions for the economic analysis is provided in **Section 8.6.2**. The alternatives are then ranked, and the apparent best alternative is presented.

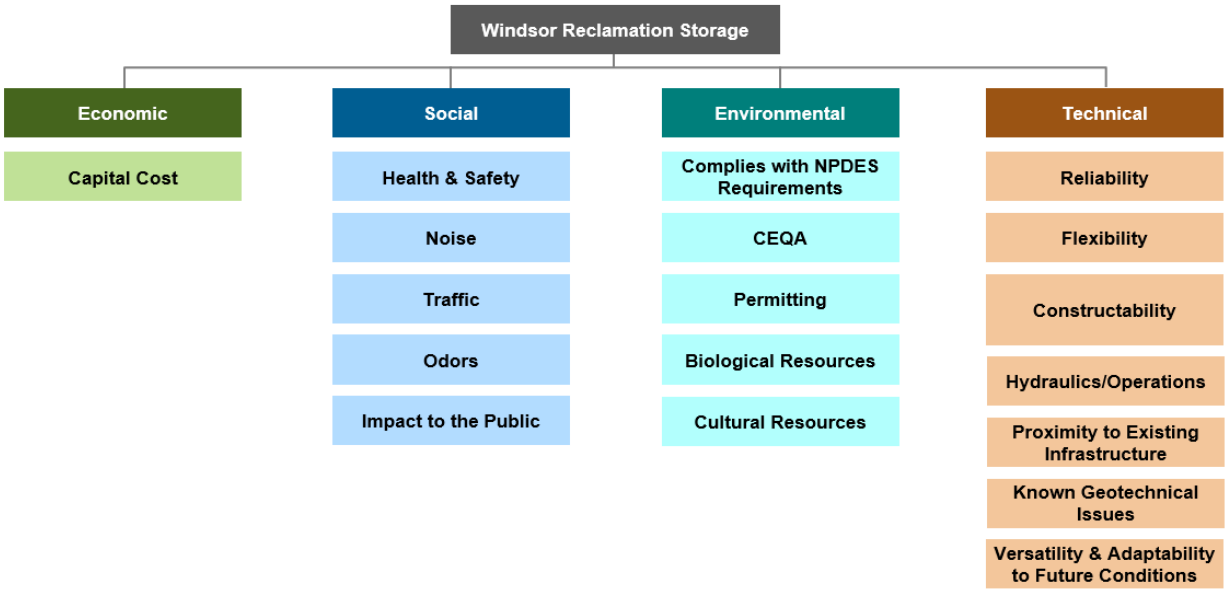


Figure 8.5: Triple-Bottom-Line-Plus Criteria

8.6.1 Social, Environmental, and Technical Analysis of the Alternatives

The following discussion of each alternative addresses social, environmental, and technical criteria. Economic evaluation is discussed below in **Section 8.6.2** and **Table 8.3**.

8.6.1.1 Alternative 1: Operational Modifications

Preliminary results of the water balance modeling work and analysis by Brelje and Race Consulting Engineers and members of the Hazen Team indicate the Town may be able to reduce discharge by approximately 8 MG per year, on average, if the Town modifies its storage management approach by using the modified storage curve described in **Figure 8.4.** Making operational changes is not currently an approved alternative to provide "official" storage capacity. Therefore, to be conservative, it was assumed

that this method would not assist with re-rating the WRF. These operational modifications have, however, been approved to offset phosphorus loading to MWC. The Town has already begun to pursue this option.

8.6.1.2 Alternative 2: New Storage Pond(s)

The Town Water Reclamation Master Plan (2001) identified potential pond sites and further examined options in a certified EIR. The supported alternative included additional storage on Town-owned property along Eastside Road. Three potential pond configurations were evaluated, Ponds S (125 MG), and Ponds T1 (249 MG) and T2 (146 MG). Estimated costs for these three ponds were $4.9, $9.4, and $5.8 million in 2001. These costs included the cost of land that the Town had already paid for the property. Under Resolution No. 1006-1 the Town Council supported the alternative to seek 182 MG of additional storage along with discharge to the Geysers Pipeline; however, the construction of Ponds S, T1 and T2 were postponed as long as possible in order to avoid extensive tree loss. The use of alternative pond sites was encouraged. Acknowledging this resolution, the focus of this section is to evaluate economic and non-economic criteria for alternative storage sites.

The City of Santa Rosa investigated potential storage pond sites and documented the results in their 2008 Seasonal Storage Engineering Report[2]. Potential sites were located from Alexander Valley to Cotati, and from Forestville to Oakmont. After excluding sites in active fault zones, or less than 20 acres in area, 34 sites were identified. Of these 34 sites, six were identified that provided at least 100 MG of storage which potentially makes them suitable for the Town's use. A new multi-criteria decision analysis would be required to determine the suitability of any of these sites for additional storage for the Town. Storage at a new off-site location would allow the Town to construct without impacting the reclamation system operations.

Several factors are important to consider. Locating storage at a remote site would increase the maintenance costs to the Town and distribute its staff across a larger area. Further analysis is required to determine how the stored flow would be delivered to and transported from the new storage site. Off-site storage may require replacing or adding pumping and pipeline infrastructure which would increase the cost and require acquisition of additional rights-of-way.

If the new pond(s) are shared among agencies, part of the infrastructure costs (including shared pipelines) may be burdened by each. There is also the added benefit of potential grant funding for projects of regional value that benefit multiple agencies and portions of the Sonoma County. Recent Proposition 1 funding from the State has prioritized projects of regional benefit that can store and reuse stormwater. Stormwater impacts peak flows at the WRF. Treated effluent peak flows have similar potential to be stored and reused.

The potential construction impacts of noise, dust, and traffic would have to be evaluated. Political and community concerns would need to be identified and addressed when proposing new recycled water storage facilities.

Environmental impacts could potentially be the largest concerns with developing a site for storage. A new storage site would require costly extensive permitting and environmental review. The Town Water

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[2] 2008 Seasonal Storage Report, City of Santa Rosa

Reclamation Master Plan (2001) identified potential pond sites and further examined options in a certified EIR.

The Town opted to delay any new pond construction on the Eastside Road property at that time to avoid extensive tree loss. It is unlikely the existing environmental document could be used for a new siting study, as conditions have changed since 2001. As part of the CEQA review, an environmental survey of sensitive species, trees and potential cultural resources would be required.

The Town may choose to require ownership easements to maintain an offsite pond according to NCRWQCB requirements. Use of recycled water by private land owners would still be under NCRWQCB regulation regarding commingling with surface and well water and irrigation control. More in-depth geotechnical and hydrological studies would be needed to ensure the suitability of any sites selected. These costs would all be in addition to the construction costs described in **Section 8.6.2.**

8.6.1.3 Alternative 3: Increase Existing Pond Capacity

Hazen conducted evaluation of raising the berm at Pond 7 to increase the existing pond capacity at the WRF. Pond 7 was chosen since it is the largest pond and increases in berm height would yield the largest volume increase. The additional storage gained was calculated based upon design information from record drawings. For the purposes of this evaluation, the volume of the pond was approximated as an inverted, truncated pyramid. The calculated volume was checked against the "Pond Elevation VS Volume" Table 28.3 from Chapter 28 the WRF O&M Manual by extrapolating the volumes to the appropriate berm height. The two calculations agreed within 10 percent, within the expected accuracy for this level of analysis.

Based upon record drawings, the water surface elevation (WSE) in the UV channels is 105.7 feet. Raising the berm height of Pond 7 by 3.5 feet to accommodate a WSE just below 105.5 feet provides 21.3 MG of additional storage, which is 23.1 MG less than the 44.4 MG goal.

Furthermore, any increase in berm height may necessitate collaboration with the Division of Safety of Dams. Because Pond 7 would likely require major rehabilitation to meet current dam safety requirements, increasing the elevation by 3.5 feet may be cost prohibitive and the increased volume would still not fully meet the Town's storage needs to meet the 44.4 MG goal.

In order to achieve the desired additional storage of 44.4 MG, the Pond 7 berm height would need to be raised 7.3 feet to accommodate a WSE of 109.3 feet. To accommodate the increase in water level downstream of the UV channel, a pump station would be required. A 7.2 MGD pump station was estimated to be sufficient to meet typical peak flows, because this is greater than the current treatment capacity of the WRF. The pump station may not always need to be operated. The pump station may only need to be operated during the final fill of the spring when water storage peaks prior to summer. The additional pump station would increase the operational and maintenance burden on plant staff. Furthermore, it would add an additional element of risk during high storage level periods; backup power considerations would have to be addressed. Raising the Pond 7 berm would allow the Town to maintain the additional storage while also allowing staff to remain on-site. This analysis focused only on storage requirements. Further analysis of the effects of expanding the storage pond footprint on existing

infrastructure and siting of a 7.2 MGD pumps station are required to assess any potential impacts to treatment processes.

The design, engineering, and permitting work required to increase the existing pond capacity is less than the new storage alternative (Alternative 2). By keeping all storage onsite, no additional land or rights-of-way would need to be acquired. Since this alternative is utilizing existing storage ponds, CEQA requirements and potential environmental impacts, related to endangered species, heritage trees and cultural sites should be less than a new off-site pond. Construction within the plant area would also reduce construction concerns related to noise and dust. Traffic control during construction would still require coordination with the community, but no permanent impacts would be expected.

While a geotechnical study would still be required, the analysis would be condensed, focusing mostly on the stability of the existing slopes and effects on increasing berm heights of Pond 7. Increasing the height of Pond 7 dikes would require coordination with the California Division of Safety of Dams (DSOD). Because Pond 7 is not currently within DSOD jurisdiction, a complete reconstruction may be necessary to bring the pond into DSOD compliance. The NCRWQCB may also require the installation of a liner. These costs are not included in the probable construction cost estimate provided in **Section 8.6.2**.

Expansions of the Pond 7 footprint to raise the berm height may be limited by surrounding creeks, homes, roads, and railroad tracks. Consideration would also have to be given to the impacts on treatment processes during construction. While the pond is being expanded, some or all of the storage capacity would likely be lost, and temporary storage or operational changes would be necessary and require further planning. The Town would not likely be able to easily accommodate a large loss of Pond 7 capacity for a significant length of time. In order maintain recycled water demands, emptying Pond 7 would likely need to be performed at the end of the non-discharge season (summer/fall) and a limited time window of work would be available before the pond would be needed again for storage. This timeframe would mainly be driven by the available storage remaining in Ponds 5, S2, 8, 9, and 10. In addition, current plans to install a floating solar panel system on the Pond surface may impact the implementation of this alternative. Additionally, new solar panels are planned to be installed in Pond 7 and would need to be removed and reinstalled to accommodate expansion of the pond.

Increasing the berm heights of Ponds 5 and 8 was also considered by Town Staff. These ponds are separated from neighborhoods by other bodies of water. The berm roads levels could potentially be raised to maintain the required freeboard. Increasing Pond 5 by 2 feet should provide an estimated additional 3.2 MG. Increasing Pond 8 by the same height would provide an estimated additional 2.8 MG. Increases to Ponds 5 and 8 can also be combined with increasing Pond 7 to provide the 44.4 MG needed to re-rate the WRF. Because the Pond 7 height would still need to be increased above the maximum water level of the UV channels (to meet the overall 44.4 MG goal), a UV effluent pump station would still be needed. It was assumed that this option was approximately equivalent to increasing the berm height of Pond 7 by 7.3 feet when considering both cost and feasibility. The cost evaluation for Alternative 3 is based on increasing berm height of Pond 7 only. Increasing the berms of Ponds 5 and 8 are presented only as alternative options to increasing existing pond storage and need to be further evaluated during a more detailed study of options to increase existing pond capacity.

8.6.1.4 Alternative 4: Joint Use of Ponds Owned by Other Agencies

The Town's need for additional storage is also shared by other adjacent agencies who operate similar weather-dependent reclamation systems. Therefore, the Town is working with the adjacent agencies to develop a more holistic approach to storage. While the Town is already pursuing this concept with the SCWA and the City of Santa Rosa, developing the details of this alternative requires multi-agency collaboration and is beyond the scope of this study. A major hurdle for this alternative will be coordination regarding the potential discharge of commingled reclaimed water.

8.6.1.5 Alternative 5: Increase Flow to Geysers

The Town currently delivers 0.2 MGD to the Geysers steam fields via the Geysers Pump Station and Pipeline during non-discharge months, and 0.7 MGD during the discharge season. The Town currently conveys an annual average flow of 0.53 MGD to the Geysers. Under the Town's contract with Santa Rosa, the Town could increase its flow to the Geysers to a maximum delivery schedule of an annual average 0.75 MGD. According to the Geysers Contract, the Town must provide advance notice (up to a year in advance) of an increase in flow delivered to Geysers. It is at Santa Rosa's discretion to change the yearly amount of Geysers deliveries.

Based on the current NPDES Discharge Permit, increasing Geysers transfers to the next tier (0.75 MGD), decreases the overall ultimate need for storage from 207 MG to 165 MG. Thus, increasing Geysers discharge decreases storage requirements by 42 MG, nearly eliminating the need for additional storage needed to re-rate the WRF. Under this scenario, only an additional 2.4 MG of storage would be needed to re-rate the WRF to a 2.25 MG design treatment capacity.

Increasing to this next tier would require that the Town also increase summer transfers to the Geysers to 0.42 MGD, slightly more than double the flowrate that the Town now provides during the summer months. This would reduce available water for irrigation and other uses during the summer by approximately 26.8 MG. This water is valuable as the Town's irrigation demands grow and should be weighed carefully against the need to re-rate the WRF to the design capacity. The Town currently receives approximately $4.33 per 1,000 gallons of recycled water from its customers; thus, the value of this water could be up to $116,000 per year. The Town could return to the 0.53 MGD annual average flow (the current agreement) if this water is needed for the Town's irrigation users, however the Town may have to provide notice to the City of Santa Rosa up to a year in advance of any change.

In the winter, the Town would benefit by increasing Geysers flow to the higher tier; this would serve to reduce the mass of phosphorus discharged to MWC during the discharge season.

Since this alternative is already implementable with the existing Geysers Pump Station, environmental and permitting issues should be minimal, as would potential concerns related to construction activities in the community. The firm capacity of the Geysers Pump Station is 1.67 MGD; reliability of the equipment under the higher flow rate should be confirmed.

As with any reuse alternative, conveying more tertiary disinfected Title 22 water into the Geysers system does run the risk of, at a future time, the Geysers or the City reducing or eliminating its need for outside water. Were that to occur, the Town would have to find another destination for the water. Costs for storage may be higher at that time than they are now.

In order to completely eliminate the need for additional storage, a flow of approximately 0.76 MGD would have to be conveyed annually, on average, to the Geysers. This is calculated by extrapolating the required flow needed at the current storage capacity of 162.6 MG from **Table 8.2**. Increasing flow to 0.76 MGD would require discussions with the City of Santa Rosa, Calpine, and other concerned agencies as the Town is currently limited to a maximum annual average discharge of 0.75 MGD. The Town would also have to negotiate with the NCRWQCB for this scenario to be included in the permit, similar to what is seen in **Table 8.2**.

8.6.1.6 Alternative 6: Combination of Alternatives

The Town may also consider a combination of the previous alternatives. The appropriate combination of alternatives would be dictated by the Town's priorities and preferences. The operational modifications alternative is recommended for consideration in combination with all options. By utilizing the storage curve presented in **Figure 8.4**. the Town should be able to reduce its discharge and thus reduce phosphorus mass loading to MWC. This alternative is already being pursued.

Increasing Pond 7 berm height by 3.5 feet would provide 21.3 MG of storage without requiring a pump station. In order to re-rate the WRF, an additional 23.1 MG of storage would still be needed. This could be provided by new ponds or by negotiating the use of storage owned by other agencies. The combination of increasing Pond 7 berm height and new storage is presented as Alternative 6a. This would reduce the footprint of new storage needed.

There are several possible "spinoffs" of Alternative 6a; however, they are all technically equivalent and would likely have a similar cost. Town staff evaluated increasing the berm height of Pond 5 and 8. Increasing the berm height of Pond 5 by 2 feet should provide an estimated additional 3.2 MG. Increasing Pond 8 by the same height would provide an estimated additional 2.8 MG (6 MG total). These could be combined with increasing the berm height of Pond 7 by 3.5 feet, so that the water surface elevation is below that of the UV channels. This would provide 21.3 MG of storage from Pond 7 and would not require a pump station. In order to meet the 44.4 MG needed to re-rate the WRF, an additional 17.1 MG of storage would need to be provided by constructing new ponds or by entering into agreements with other agencies. Whether Pond 7 berm height is increased or a combination of Ponds 5, 7, and 8, the combination of increasing existing storage with new storage is more or less equivalent in terms of technical and cost considerations. These are considered as similar and all being represented under Alternative 6a.

The Town could combine increased Geysers flow with other alternatives to provide sufficient reclamation and storage capacity to re-rate the WRF to an ADWF of 2.25 MGD. The increased Geysers flow would reduce the required storage needed and lower overall capital costs of the other options. For example, the Town could increase Geysers flow in conjunction with constructing a new pond. Increasing Geysers flow could reduce the footprint needed for a new pond, since only a 2.4 MG pond would be needed to meet the permit requirements for rating the WRF to 2.25 MGD ADWF. The reduction in pond footprint would reduce the overall cost of this alternative. This is presented as Alternative 6b.

Increasing flow to the Geysers could also be combined with increasing the berm height of Pond 7 to provide the remaining 2.4 MG storage. This is presented as Alternative 6c. The Town may also need to pursue permitting for Pond 7 with the DSOD. These costs are not included in this analysis. Similar to

Alternative 6a, there are several "spinoffs" of this which could combine increasing the flow to Geysers with increasing the berm heights of either Ponds 5, 7, or 8. These are considered technically equivalent, and would likely have a similar cost. A combination of increased Geysers flow and increasing the capacity of an existing pond is taken together as Alternative 6c.

Use of storage owned by other agencies could potentially be negotiated. If done in conjunction with Geyser flow, the needed quantity of such additional storage could be minimized. This would reduce the Town's financial and operational investment in offsite resources. Combining increased Geysers flow with storage owned by other agencies could be an interim solution to allow the Town to re-rate the WRF. This could buy the Town additional time until new storage can be built. New storage is an ideal long-term solution, as it is the most effective alternative for meeting the Town's goal of being able to operate the WRF without reliance on outside agencies (listed in **Section 8.1**).

Any alternative requiring an increased flow to the Geysers would cause the Town to lose a significant amount of recycled water that it provides to its irrigation customers. As discussed in **Section 8.6.1.5**, increasing the Geysers flow would reduce the amount of recycled water to customers by approximately 26.8 MG per year. Alternatively, the Town could provide an additional 26.8 MG of storage, which would allow the Town to continue to provide this water to irrigation customers. This would present a very significant cost. If providing water for irrigation customers is a primary concern, the Town should keep the current delivery schedule for Geysers flow. The cost of 26.8 MG of storage is not included in any of the Geysers flow cost estimates. It is assumed that if the increase in Geysers flow is pursued, the primary concern is to re-rate the WRF, and the water for irrigation customers will be lost.

Determining the correct mix of alternatives depends upon input from Engineering and Operational staff. An analysis is provided in **Section 8.6.3** which ranks the alternatives based on an array important factors. Additional combinations of these alternatives may be necessary to optimize the Town's progress towards the operational objectives stated in **Section 8.1.**

8.6.1.7 Alternatives Requiring Further Evaluation

It was determined that additional storage could be provided by reducing the freeboard of Pond 7 from 3 feet (as listed in **Table 8.1**) to 2 feet, or from a WSE of 102 feet to a WSE of 103 feet. This would provide an additional 6 MG of storage. The spillway elevation is 102.5 feet and would have to be raised. If the spillway elevation is not increased, only 3 MG of additional storage would be provided. This alternative should be analyzed further. It should be confirmed that Pond 7 can safely accommodate this increase in water surface. The design and integrity of the pond should be inspected. The Town is already considering this potential freeboard reduction and considering how it will impact the design and installation of the future floating solar array. The design of the solar panels should ensure that the array anchors and the maximum height of the floating solar array will allow for the potential change to a 2-foot freeboard. It should be evaluated whether a 2-foot freeboard can safely accommodate wave action and can meet proper design standards. Additional review of this alternative is recommended.

Based on the preliminary analysis, increasing the Geysers flow and decreasing the freeboard of Pond 7 would provide sufficient storage and reclamation capacity to re-rate the WRF. The feasibility of decreasing the freeboard of Pond 7 or increasing the berm heights of Ponds 5 and 8, should be further evaluated.

The alternative idea of expanding pond volumes by deepening one or more ponds was explored by Town Staff during this study. For example, it was determined that increasing the depth of Pond 7 by three feet would result in a storage gain of approximately 14.7 MG of storage. Should deepening of a pond be considered, a detailed engineering study should be performed. The reconstructed (deepened) pond may first need to be emptied and the impermeable core of the existing berms would have to be keyed into the newly established deeper pond bottom. Compared to constructing new ponds, pond siting would not be an issue and CEQA permitting may be slightly more cost effective. The change in pond construction may require a permit re-opener or other regulatory hurdles. An in-depth analysis of pond deepening is not included in this report and should be analyzed in further detail in subsequent studies prior to implementation.

8.6.2 Economic Evaluation

Estimates of Probable Construction Cost have been developed for the alternatives; detailed cost breakdowns are included in **Appendix H**. The estimates were developed in accordance with the guidelines of the Association for the Advancement of Cost Estimating International (AACEI) for a "Class 5" estimate. A "Class 5" estimate is based on planning level data. The following are assumptions that were used in the development of the estimates:

- New Storage Ponds: The new storage pond costs were developed by escalating the costs from the 2008 Seasonal Storage Project Engineering Report by the City of Santa Rosa to August 2017 using the Engineering News-Record (ENR) construction cost index for San Francisco. This report developed costs in accordance with the guidelines of the AACEI for a "Class 5" estimate. The costs include the provision of an additional 1 mile of 24-inch pipe and pump station to connect the new storage ponds to the existing transmission system. No analysis of the existing conveyance infrastructure was performed to verify adequate capacity was available to handle additional flow.

- Existing Storage Pond Expansion: Cost estimates that involve increasing the existing storage pond volumes use costs that are estimated and derived for Pond 7. Although increasing either Pond 5 or Pond 8 should also be considered, Pond 7 was assumed for this analysis. The resulting cost estimates are approximately representative of expanding any pond. The expansion of existing storage pond capacity was developed based upon maintaining existing pond slopes and raising berm elevations. This alternative does not include analysis of any costs which may be required to modify existing infrastructure (e.g. piping, outlet adjustments, raising existing pump stations) to accommodate the increased pond footprints. Furthermore, it does not include permitting and other costs associated with complying with the DSOD requirements. Based upon record drawings, the water surface elevation at the ultraviolet (UV) effluent channel is 105.7 feet, Additional pumping will be required to lift the water above 105.7 feet. Because Pond 7 berm heights would need to be raised approximately 7.3 feet to accommodate the 44.4 MG storage goal, this alternative includes the cost of a 7.2 MGD pump station to convey the flow from the UV effluent channel to the expanded pond.

- Labor rates were based upon wage rates local to the Town and productivities based upon published resources such as RS Means.

- Additional factors for site prep, grading, drainage, erosion control, mechanical additions and site improvements were included based upon adjusted ratios from the 2008 Seasonal Storage Project Engineering Report.
- It was assumed that any costs that would be incurred for increasing the flow to the Geysers would negligible compared to the other alternatives considered. The existing Geysers Pump Station capacity is 1.67 MGD. Increasing the Geysers flow by 0.22 MGD will increase the total flow to 0.75 MGD. Since the new flow is still within the existing pump station capacity, no additional work was assumed to be required to achieve this increased flow.
- The costs presented for this analysis are estimated construction costs. No provision was included for land acquisition, environmental mitigation, engineering, construction management, administrative, legal, permitting, operations and maintenance or change order reserves.

A summary of the Estimated Probable Construction Cost for each alternative is provided in **Table 8.3** below. Detailed cost data is provided in **Appendix H**.

Table 8.3 Estimate of Probable Construction Cost

Alternative Number	Alternative Description	Estimated Probable Construction Cost (Millions)[1,2]
1	Operational Modifications	N/A
2	New Off-site Storage Ponds (44.4 MG of Storage)	$41
3	Increase Existing Pond(s) Capacity (raise Pond 7 by 7.3 feet for 44.4 MG)	$13.5
4	Joint Use of Ponds Owned by Other Agencies	N/A
5	Increase Flow to Geysers	N/A
6a	Combination of New Ponds and Increasing Existing Pond Capacity (2.4 MG)	$28.4
6b	Combination of New Ponds (2.4 MG) and Increasing Flow to Geysers	$12.3
6c	Combination of Increasing Existing Pond Capacity (2.4 MG) and Increasing Flow to Geysers	$3.5

[1] Estimated Probable Construction Cost does not include costs for: land acquisition, environmental mitigation, engineering, construction management, administrative, legal, permitting, operations and maintenance or change order reserves
[2] Costs were escalated from 2015 to 2017 dollars using ENR's Construction Cost Index

8.6.3 Ranking of Alternatives

The alternatives were ranked based on (1) their ability to progress towards the Town's stated operational objectives and (2) a triple-bottom-line-plus evaluation. The triple-bottom-line-plus evaluation includes the typical criteria in a triple-bottom-line evaluation: (1) economic, (2) social, (3) environmental, plus (4) technical considerations. These criteria are listed in **Figure 8.5**. The Town's operational objectives are provided in **Section 8.1**. The rankings are presented in **Table 8.4.**

Table 8.4: Effluent Storage Alternatives Ranking

Alternatives	Town Operational Objectives[1]					Triple-Bottom-Line-Plus[3]			
						Economic Criteria	Social Criteria	Environmental Criteria	Technical Criteria
	Increase Permitted ADWF Capacity[2]	Reduce Phosphorus Mass Loading	Increase Available Recycled Water	Zero-Discharge to MWC	Self Sufficiency	Capital Cost	H&S, Impact to the Public: Odors, Noise and Traffic	CEQA, NPDES, Permitting, Biological Resources, Cultural Resources	Reliability Flexibility Constructability, Hydraulics, Operations, Proximity to Existing Infrastructure, Known Geotechnical Issues, Future Conditions
1. Operational Modifications	5	3	4	4	1	1	1	1	5
2. New Storage	1	1	1	1	1	5	2	4	1
3. Increasing Existing Pond Capacity	2	1	2	2	1	4	3	5	2
4. Joint Use of Ponds Owned by Other Agencies	2	1	2	2	5	2	1	2	5
5. Increase Flow to Geysers	2	1	5	2	4	1	1	1	3
6a. Combination New Ponds and Increased Pond Capacity	1	1	2	2	1	4	3	5	2
6b. Combination New Ponds and Increased Flow to Geysers	1	1	3	2	3	4	2	3	2
6c. Combination Increased Pond Capacity and Increased Flow to Geysers	2	1	3	2	3	3	2	5	2

[1] *Scale 1 to 5. A ranking of 1 indicates that an option is well suited to meet a goal.*
[2] *Scale 1 to 5. Ranking of 1 has the least impact or the lowest cost.*
[3] *Triple-bottom-line-plus refers to an analysis that considers the triple bottom line criteria (economic, social, and environmental) plus technical criteria.*

8.7 Apparent Best Alternatives

Key conclusions are listed below and are not in any order of ranking:

Alt. No. 1 Operational Modifications. Operational Modifications are recommended to be combined with any option. This can be implemented quickly. It was assumed that this alternative will not provide the required additional storage to re-rate the WRF to an ADWF capacity of 2.25 MGD. Operational modifications should decrease discharge to MWC, and thus decrease phosphorus loading.

Alt. No 2 New Storage Pond(s). The construction of new storage best suits the Town's operational objectives; however, it is the most costly alternative.

Alt. No. 3 Increase Existing Pond(s) Capacity. An increase in the berm height of Pond 7 is not preferred, as permitting with DSOD may make this option cost-prohibitive. Bringing Pond 7 into compliance with DSOD standards would likely require reconstruction of the pond. These requirements should be carefully weighed. Further evaluation is required to determine whether this alternative is cost-effective; permitting costs were not included in this analysis. Another potentially less-costly option would be increasing the capacities of Ponds 5 and 8; however, these two ponds alone cannot meet the overall storage goals without implementing an additional option. This would need to be evaluated further.

Alt. No. 4 Joint Use of Ponds Owned by Other Agencies. The joint use of ponds owned by other agencies is an attractive option because of the use of existing infrastructure and ability to implement in a short timeframe. This alternative requires further agreements which are beyond the scope of this study, and Town staff are currently exploring this alternative.

Alt. No. 5 Increase Flow to Geysers. Increasing the flow to the Geysers is the least costly option, however, this alternative also reduces the Town's ability to provide water to its recycled water customers. This could be an interim solution to provide additional capacity for re-rating the WRF. The Town could revert back to its current agreement in the future (annual average of 0.53 MGD) if the City of Santa Rosa is given sufficient notice per the Geysers Contract.

Alt. No. 6a Combination. A combination of new ponds and increasing the existing pond capacity has high capital costs. Additionally, bringing Pond 7 into compliance with DSOD standards would likely require reconstruction of the pond. These requirements should be carefully weighed. Further evaluation is required to determine whether this alternative is cost-effective; permitting costs were not included in this analysis.

Alt. No. 6b Combination. A combination of new ponds and increased flow to Geysers provides an intermediate alternative. The increased flow to Geysers would reduce recycled water that can be provided to customers, however it would also decrease the capital costs needed to construct a new pond because a smaller pond would be needed to meet the Town's storage needs.

Alt. No. 6c Combination. A combination of increased Pond 7 capacity and increased flow to Geysers is not preferred. Permitting with DSOD may make this option cost-prohibitive. DSOD requirements and permitting would need to be evaluated.

Alternatives Requiring Further Evaluation	The Town should consider a rise in the nominal operable freeboard of Pond 7. Raising the freeboard from 3 to 2 feet could result in a gain of 6 MG. Additionally the deepening of Pond 7 should be studied. These alternatives require further analysis to determine their feasibility.

Based on these findings, the combination of operational modifications to reduce phosphorus loading, and the construction of new storage would best suit the Town's operational objectives. However, either the joint use of ponds owned by other agencies or a combination of increased flow to Geysers and new ponds might present a cost-effective solution if the Town has the short-term goal of increasing the WRF's permitted ADWF to a maximum of 2.25 MGD. As previously mentioned, increased flow to the Geysers would come at the loss of approximately 26.8 MG of stored recycled water for users during the summer months. A final determination of the best alternative would require an extensive review of many additional factors and would require input and prioritization by Engineering and Operational staff. An analysis should be performed to identify the most important factors for the potential array of solutions, including both economic and non-economic factors. Weights should be carefully assigned to prioritize the needs of the Town.

As previously mentioned the daily time-step model indicated that there could be a reduction in discharge between 7 and 11 MG per year. The water balance model indicated an average reduction in discharge of 8 MG per year. The two models predict similar reductions in discharge, and thus it is anticipated that the results of the daily time-step model will not affect the overall conclusions of this analysis. A decision should be made to determine which model will be carried forward to determine the Town's optimal operational storage curve. Currently the Town is using a revised modified storage curve with a more conservative dead storage and peak storage.

It should be noted that in **Sections 3 and 6**, the anticipated flows through 2040 are analyzed. Based on the assumptions of population growth which is evaluated in those sections, it is not anticipated that ADWF will exceed the current 1.9 MGD permitted limit until 2040. Thus, there is no immediate need for additional reclamation and storage capacity to re-rate the WRF to 2.25 MGD. Because the analysis completed in **Sections 3 and 6** inherently has a high level of uncertainty, the Town should consider increasing the permitted ADWF to provide flexibility and added redundancy for the future. Future growth is difficult to predict, and the Town should ensure that the WRF is operating with a buffer between typical ADWF and the permitted ADWF capacity. Re-rating the WRF to provide additional capacity may be necessary. This section provides planning-level guidance to prepare the WRF for future growth.

9. Fine Screen and Grit Removal – Headworks

This section explores alternatives for the possible replacement of the grit system and fine screen in the Windsor WRF's headworks facility.

9.1 Purpose

The purpose of this section is to summarize the existing fine screening and grit removal facilities and describe planned and needed improvements. The design of an influent screen is currently in progress. The WRF's influent pumping and headworks facility will include the future influent screen as well as the existing facilities for influent pumping, fine screening, grit removal, and a Parshall flume for flow measuring. The Town recently replaced the grit pump and is designing a new 3/8" influent screen to protect the influent pumps and reduce maintenance. The existing headworks fine screen is performing at a substandard level and repair or replacement is tentatively scheduled after the installation of the mechanical influent screen.

This section summarizes the evaluation of the existing fine screening and grit system. An analysis of potential equipment repair, rehabilitation, and replacement is provided.

9.2 Existing Facility Description

The headworks facility is located downstream of the influent pump station. It was constructed in 1996 and consists of a mechanical bar screen, grit chamber, and Parshall flume. A process flow diagram for the WRF is provided in **Section 5**. The main structure is concrete, extending 5 feet above grade and from 2 to 11 feet below grade. Fiberglass grating and aluminum handrails provide access to the deck area.

The screen is a Parkson™ Corporation Aquaguard® Screen with a 6-millimeter (mm) spacing. The screen has peak flow capacity of 9.6 MGD. The screen conveys the collected screenings directly to a waiting dumpster. If the screen fails, it can be pivoted out of the flow path. When this occurs, some work is required to relocate a level sensor.

Immediately downstream of the mechanical bar screen is a Smith & Loveless® vortex grit removal chamber, grit cyclone, classifier and screw conveyor. The grit removal chamber has a top-mounted propeller that creates a pressure differential between the top and bottom of the chamber. The pressure differential assists with grit collection at the bottom of the chamber. The settled grit falls to a bottom hopper. The collected grit is pumped to the grit classifier on an intermittent basis. The grit classifier empties directly into a dumpster through a screw conveyor for disposal offsite.

Influent flow is measured downstream of the grit removal system via a Parshall flume. At the flume, the channel width is 1.5 feet, and the depth is between 0.5 and 2 feet. An ultrasonic sensor mounted above the channel measures the water surface to determine an influent flowrate.

A plan view of the headworks fine screening and grit chamber is shown in **Figure 9.1.** The existing grit chamber and pump are outlined in orange and the existing screen is outlined in green. **Figure 9.2** shows a plan view of the grit washing/dewatering equipment with the existing classifier outlined in orange.

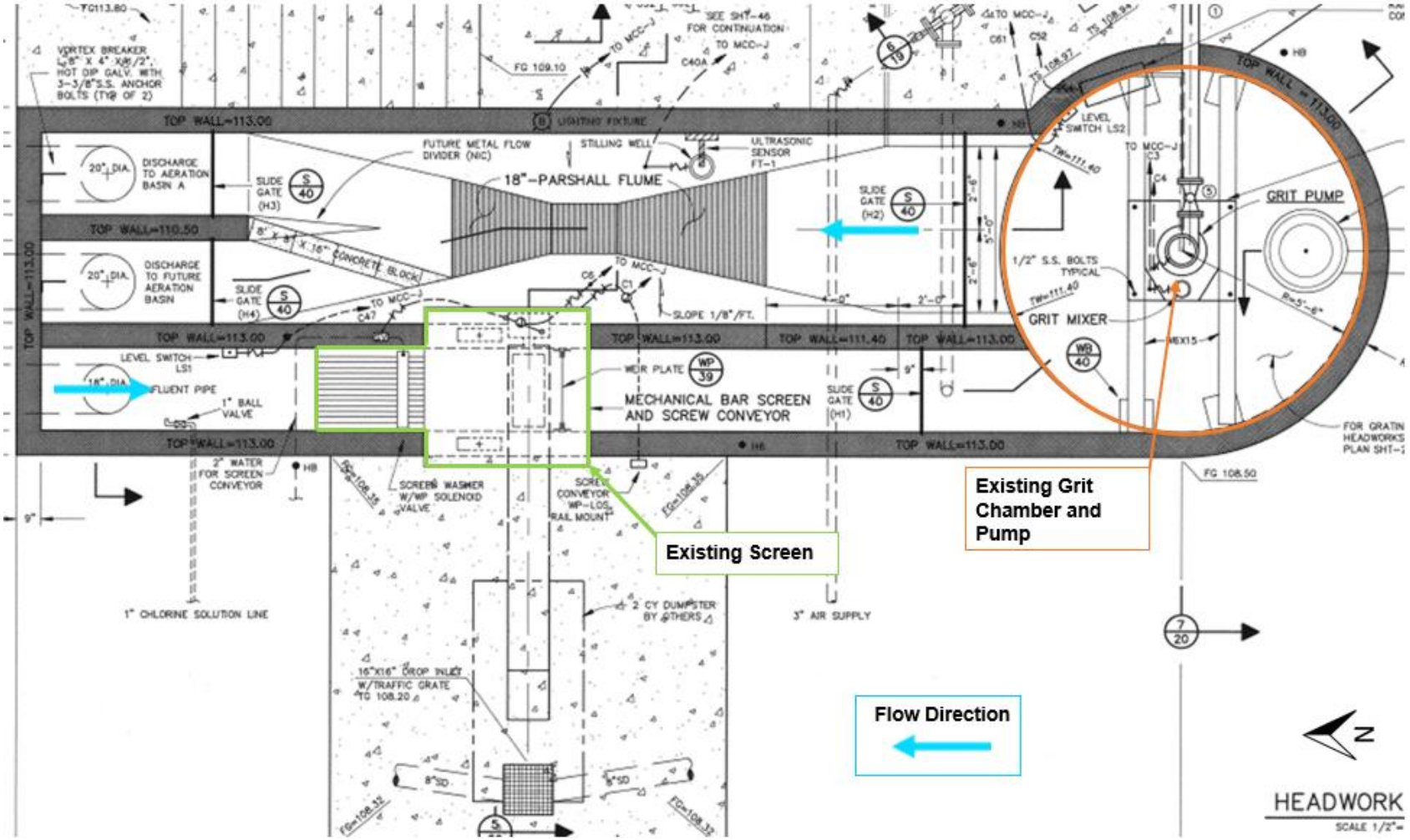


Figure 9.1: Headworks Plan View (Source: Phase-1B Stage II Improvements)

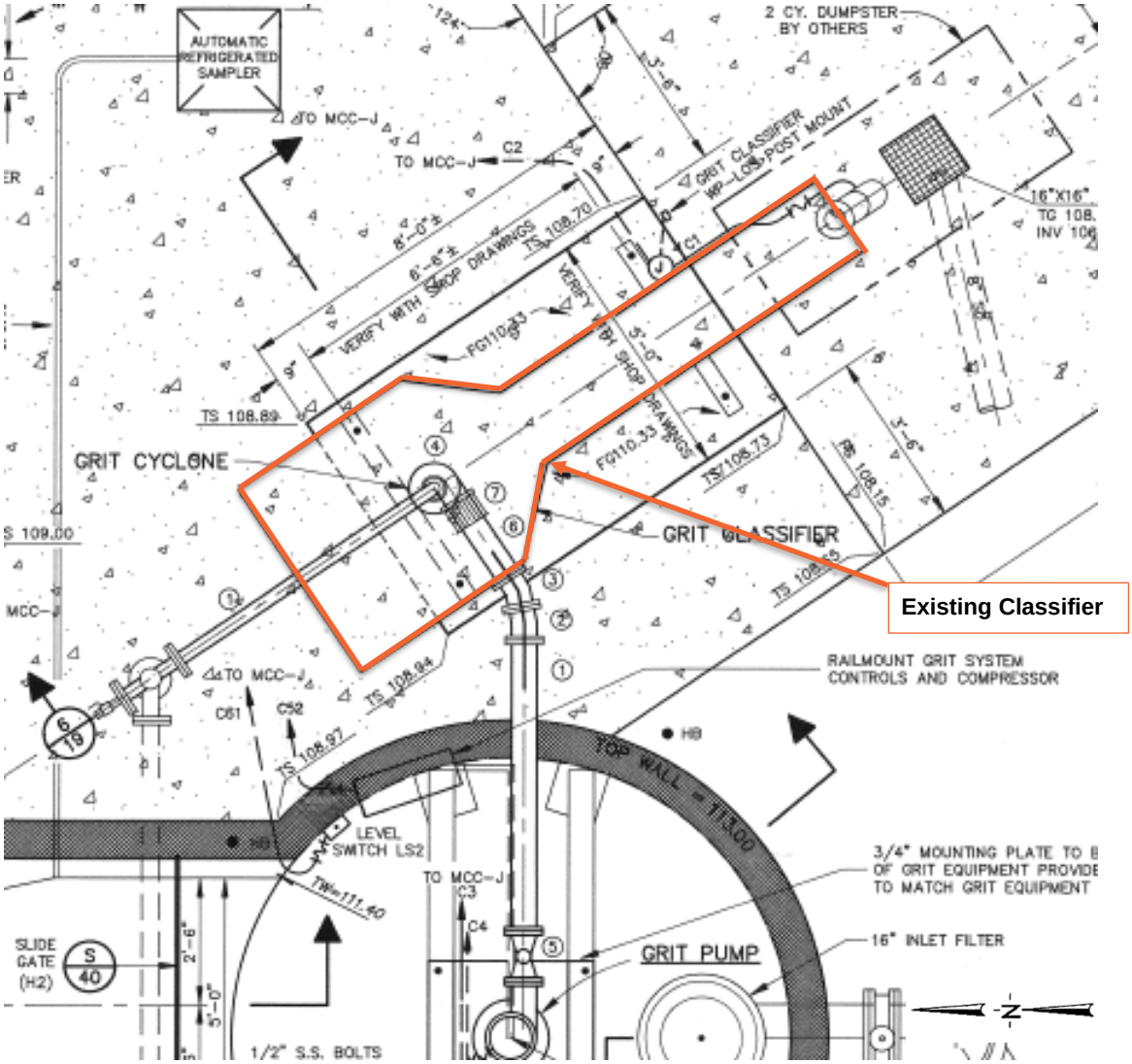


Figure 9.2: Grit Classifier Plan View (Source: Phase-1B Stage II Improvements)

Table 9.1 to **Table 9.3** summarizes the design criteria as provided in the WRF Operations and Maintenance Manual (2002) for the headworks fine screen, grit chamber, and Parshall flume.

Table 9.1: Fine Screen Design Criteria

Parameter	Design Criteria
Channel Width	2.5 ft
Screen Openings	6 mm
Motor Power	0.75 HP
Head Loss (clean)	Approx. 10 in.

Table 9.2: Headworks Grit Chamber Design Criteria

Parameter	Design Criteria
Main Chamber Diameter	11 ft
Lower Chamber Diameter	5 ft
Main Chamber Depth	5.67 ft
Lower Chamber Depth	6.67 ft
Paddle Rotation	Counterclockwise
Paddle Drive Motor Power	1 HP
Grit Pump Power	7.5 HP

Table 9.3: Headworks Parshall Flume Design Criteria

Parameter	Design Criteria
Width	1.5 ft
Water Depth	0.5 – 2 ft

The Town recently refurbished the grit chamber equipment in December of 2015. Hazen performed a visual assessment of the headworks facility. Highlights of the assessment are noted below. **Figure 9.3** to **Figure 9.8** illustrate the typical components of the headworks as well as their condition.



Figure 9.3: Fine Screen

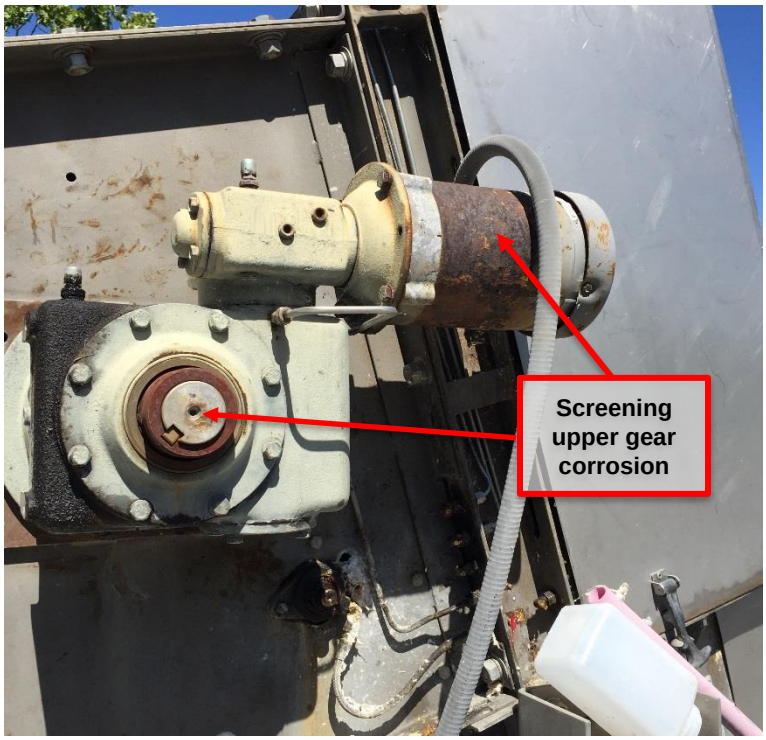


Figure 9.4: Screening Upper Gear Motor Corrosion



Figure 9.5: Screening Conveyor and Dumpster

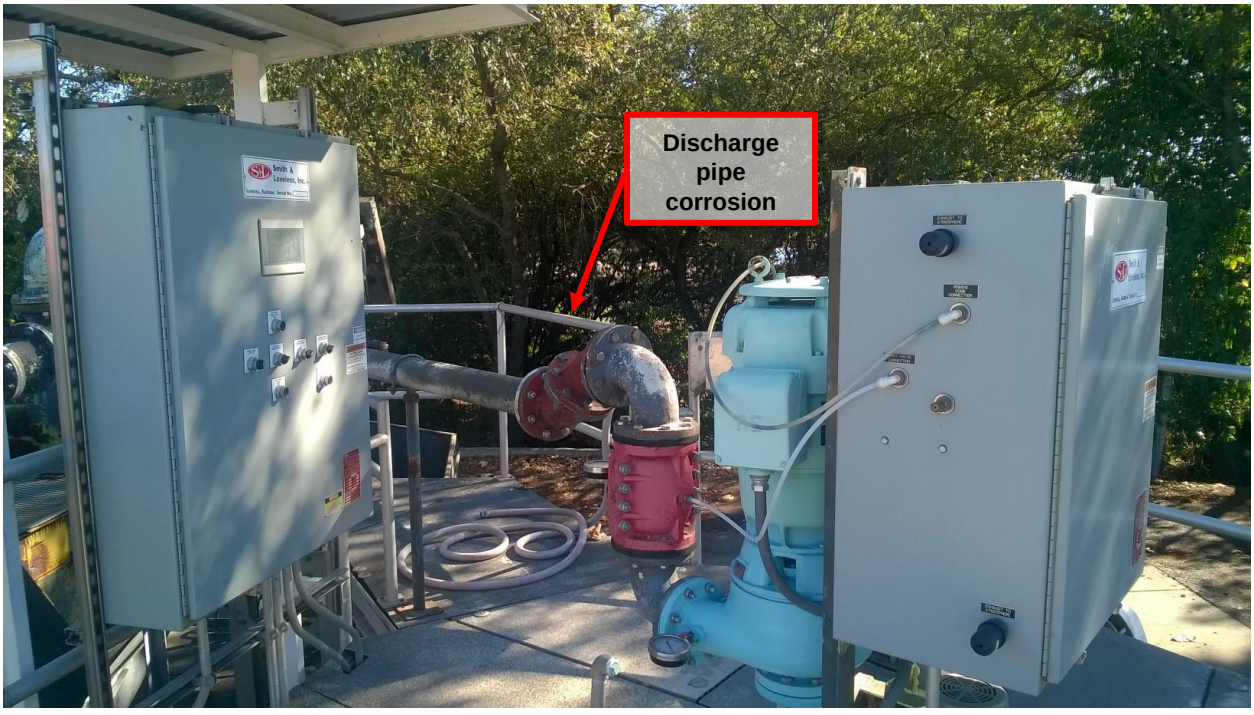


Figure 9.6: Grit Unit Corrosion



Figure 9.7: Grit Dewatering – Classifier and Screw Conveyor

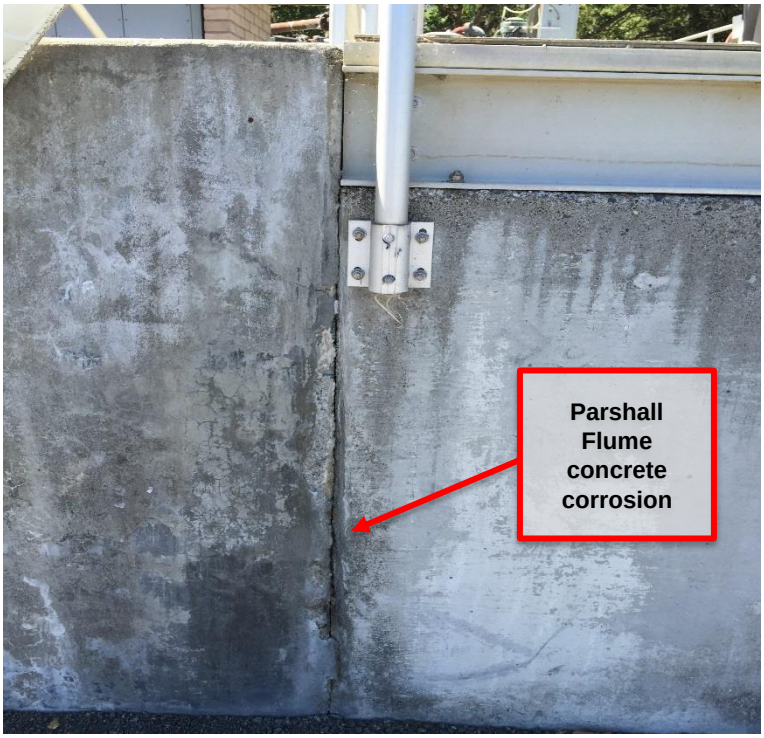


Figure 9.8: Parshall Flume Concrete Corrosion

The following is a list of specific findings on the condition of the headworks as well as information gathered from interviews with operational staff on the headworks process and surrounding area.

- There was minor corrosion on the screen housing.
- Noticeable corrosion on the screen upper gear motor housing.
- There was corrosion on the grit removal discharge pipe and valve on the discharge pipe.
- The grit classifier box is corroded and in need of replacement
- There are concrete cracks on the flume exterior.
- There is no redundancy or option to bypass around the headworks.

9.3 Technology Evaluation

9.3.1 Materials of Construction

Due to the corrosive and abrasive environment encountered at the headworks, exposed metal parts should be 304 or 316 stainless steel. Some manufacturers of center feed screens offer an ultra-high molecular weight (UHMW) polyethylene option for the perforated screen. The UHMW panels can reduce hair pinning, when hair or fibrous material becomes attached around the plate and leads to blinding.

9.3.2 Screening

Because the Town is considering refurbishment or replacement of the headworks screen within the next few years, the following section provides various alternatives and screen technologies for the Town's consideration.

9.3.2.1 Treatment Objectives for Screening

Removing floating and suspended large particle debris, hair, rags, and trash (herein referred to as "screenings") protects downstream equipment from "ragging" (an event characterized by rotating equipment binding with entangled rags); from physical damage from impact, abrasion, or clogging; and from unsightly and disrupting buildup of floatables within the process units. Operations staff have noted that screenings have affected the Biolac® diffusers. Prior to adjustment of the screen operation from timed controls to level controls, the screen allowed fibrous materials to pass beneath the screen. These materials eventually combined and collected on the diffusers. Adjusting to level controls significantly alleviated this issue; however, due to the poor condition of the screens, screenings still pass through the fine screen and settle in the aeration basins during diffuser down time.

9.3.2.2 Available screening technologies

Screening technologies commonly used for fine screen applications include step screens, filter (bar) screens, double-pass perforated plate (band) screens, and center-flow single-pass perforated plate (band) screens. The selection of an appropriate screening technology is an important design consideration for the Town. Many constraints must be taken into consideration. Some of these include:

- Ratio of average design flow to peak wet weather flows
- Desired screening capture rate
- Available head loss capacity
- Available footprint space
- Wastewater characteristics

The ratio of the average dry weather flow (2.25 MGD) to the peak week flow (7.2 MGD) is significant, because the system will have to handle a wide range of flowrates. Since the wet weather (peak) flow will be conveyed through the screen, the screen must be sized for peak flow conditions. Screen channel size should be designed for approach velocities between 2 and 3.5 feet per second (fps), regardless of the type of screen used. Velocities below 2 fps lead to accumulation of debris within the channel unless mixing is provided. Velocities above 3.5 fps may result in debris bypassing the screens.

The desired screening capture rate is typically the main parameter considered in the selection of screening technology. Higher capture rates provide the best protection for downstream processes. However, higher capture rates are associated with higher head losses and will require a larger washer-compactor and conveyor.

Wastewater characteristics such as the type of debris to be expected are also critical to the screen technology selection. As the existing headworks is downstream of the existing influent pump station where a new 3/8” influent screen is to be installed, screening material that passes through to the headworks is anticipated to be fine particles and stringy materials (little to no large debris). A brief description of available technologies for a replacement headworks screen is provided in the following sections.

9.3.2.2.1 Step Screens

Step screens consist of two step-shaped sets of thin vertical plates, one fixed and one movable. The fixed and movable step plates alternate across the width of an open channel and together form a single screen face. The movable bars oscillate in an elliptical motion. Through this motion, solids captured on the screen face are automatically lifted up to the next fixed step landing and are eventually transported to the top of the screen where they are discharged to a collection hopper. A mat of screened material is formed on the screen surface that allows the screen to achieve higher removal efficiencies. The mat is then lifted up the screen via the step function. For this process, ensuring a consistent mat is paramount, as excessive buildup can lead to balling and other issues while too little buildup reduces screening capture.



Figure 9.9: Step Screen (Source: MN Water Treatment)

9.3.2.2.2 Double Pass Perforated Plate Screens

Double pass perforated plate screens have panels attached to a drive chain. This technology has a high capture rate and can remove relatively large particles when operated correctly. The wastewater passes through the front and back side of the screening band. A brush or spray bar is used to remove screenings for disposal. If this process is inefficient, carryover can contribute to a reduction in capture efficiency. Carryover is the tendency for captured screenings to cling to the screen on the upstream side, not be removed at the top of the screen, and then simply be washed off on the downstream side decreasing overall screenings removal efficiency.



Figure 9.10: Fully enclosed double pass perforated plate screen (Source: Huber)

9.3.2.2.3 Center Feed Band Screen

In a center feed band screen (or single pass perforated plate screen) the wastewater enters the screen through the center and exits on both sides. This type of screen has a high capture rate and can handle higher flow rates as compared with a double pass screen. Due to a larger screening area, carryover is not a concern and only wash water is needed for removing the captured screenings. The disadvantage of this technology is that it requires a widened channel around the screen. It also has difficulty removing large debris.

An important side note is that perforated plate screens can be constructed out of either stainless steel or UHMW polyethylene. UHMW polyethylene panels with a thickness of at least 9 mm greatly reduce hair pinning and are preferable over stainless steel. Hair pinning or rag stapling is when fibrous material becomes attached around the plates and leads to blinding of the screen and requires rigorous cleaning to remove.



Figure 9.11: Band Screen (Source: Headworks International)

9.3.2.2.4 Drum Screens

Drum screens come in a variety of types and configurations. Some are in-channel and some are self-contained boxes. Some have perforated plate panels and others use wire-mesh. Overall, drum screens have the highest capture rates and reliability. For this reason, they are typically used in wastewater plants that operate membrane bioreactors. Due to their high capture rates, drum screens are generally more expensive than other similarly sized screening options. Given the screening needs at the WRF, the higher capture rate would not justify the higher cost.

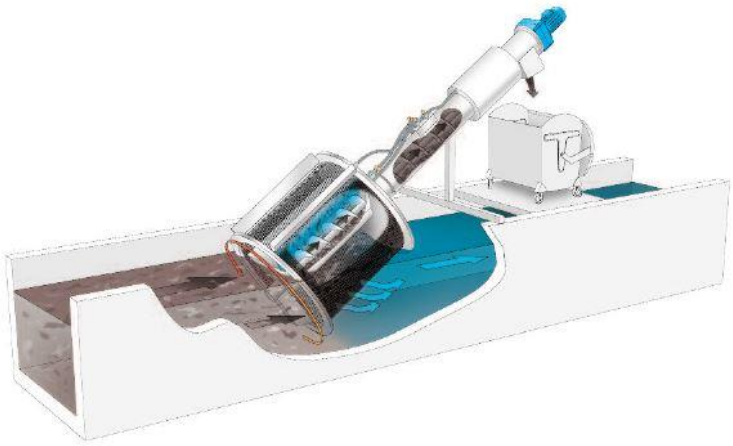


Figure 9.12: In channel drum screen (Source: Huber)

9.3.3 Screenings Washing and Dewatering

9.3.3.1 Treatment Objectives for Screenings Washing and Dewatering

The act of washing and compacting captured screenings is typically referred to as "conditioning." The current headworks does not condition the screenings prior to discharge to the dumpsters for hauling. While not required, it is preferable that a washer-compactor be provided to receive the discharge screenings. A washer-compactor will wash the captured screenings to remove organic material using non-potable water. It then compacts the captured materials to compress the material and remove excess water. Excess water is returned to the screen channel. The washed and compacted materials sent to the dumpster will have minimal water content, leaving little to no standing water in the dumpster. This should help reduce odor emissions and attract fewer vectors as compared to unwashed material. Washer compactors can also be fitted with automatic bagging attachments, which also help reduce odors and discourage birds and insects.

9.3.4 Grit Removal

9.3.4.1 Treatment Objectives

Removing entrained and suspended solids, such as sand, and debris that readily settle out in the treatment processes (herein referred to as "grit") protects downstream equipment from physical damage from impact and abrasion. It also reduces the build-up of these materials within process units (such as the aeration basins) which would otherwise reduce treatment volume. Grit particles are small, measured in microns, and therefore typically pass through screenings removal equipment. Grit removal is typically accomplished by providing a process area with a lower stream velocity to allow heavier particles to settle while keeping lighter-weight organic material in suspension. Organic material is not removed so that it can be used in biological treatment processes. Many grit removal technologies induce vortexes in the flow to assist in separating heavier particles.

Section 9.3.4.2 and **Section 9.3.4.3** compare two available technologies that could be used for grit removal: a forced vortex type system and a stacked-tray gravity type system. Each system has an accompanied grit separation and washing process (discussed in **Section 9.3.5**). It should be noted that the grit separation equipment does not necessarily have to be supplied by the same supplier as the grit removal equipment. Additional grit separation systems are discussed in **Section 9.3.5**.

9.3.4.2 Stacked Trays

Hydro International is the only manufacturer of stacked tray grit removal systems. Their system is referred to as the Headcell™ and is shown in **Figure 9.13**. The manufacturer claims that the system is able to remove grit particles down to 140 mesh while maintaining a low organic material content in the removed grit. While capture may be more effective, there are two primary disadvantages of this type of system: (1) the head loss is typically higher than a vortex system and, (2) the systems must be sole-sourced and therefore are often significantly more expensive than a comparably sized vortex system.

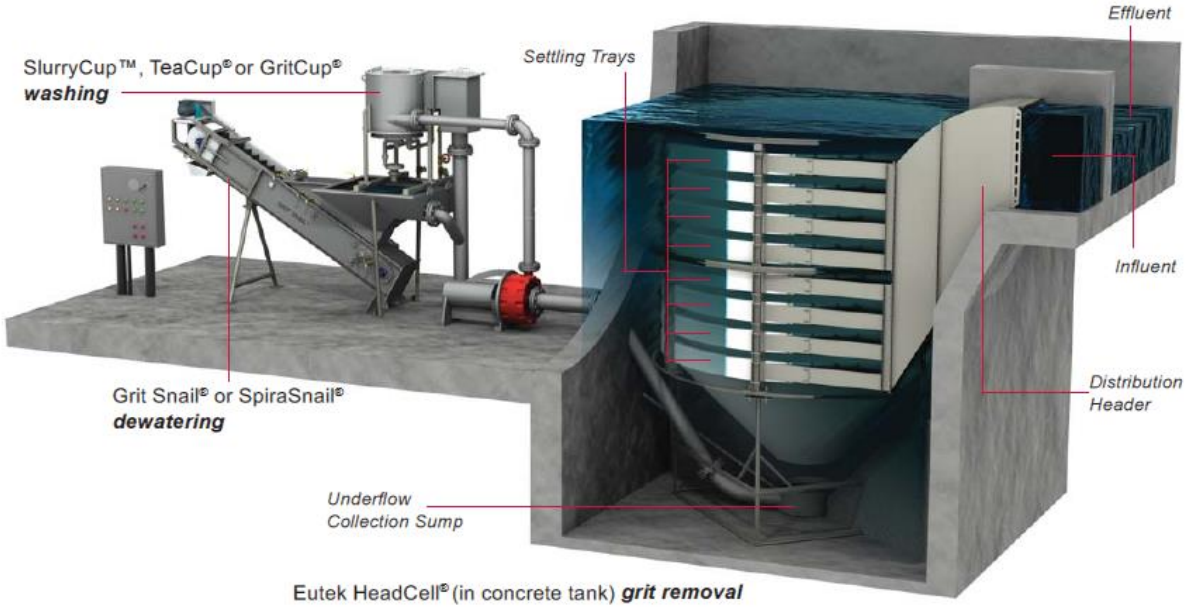


Figure 9.13: Eutek Headcell™ (Source: Hydro International)

Sizing of a stacked tray system is based on the design particle size removal and design flow. The stacked polyethylene trays are submerged within the grit chamber. The influent flow is evenly distributed tangentially to the stacked trays. The tangential flow creates a vortex flow pattern that allows solids to settle along the sloped surface of the stacked tray. The settled solids fall toward the center of the stacked tray where it enters a common sump. The collected solids are either intermittently or continuously pumped to the washing/dewatering equipment.

Conversations with Hydro International indicated that the WRF's existing headworks facility will not accommodate a stacked tray system. In order to install a stacked tray, a new grit chamber would be required, either in place of the existing grit chamber or at another location. This would add significant cost to the replacement of the equipment.

9.3.4.3 Vortex Chamber

A common type of grit removal system is the mechanically induced vortex system. These systems create a vortex by introducing the flow to the circular chamber tangentially; some systems also use a propeller located in the center of the chamber. There are two basic design types. One type of vortex design has been available for decades. This design has a sloping bottom with a large opening to a center hopper, uses a propeller and is referred to as a "stirred" vortex system. This is the system that is currently in place at the WRF as provided by Smith & Loveless®. This type of design is currently available from several suppliers, such as the Veolia Mectan™ system, Jones & Attwood® Jeta® Grit Trap 2500. These systems are low head loss chambers with head losses typically less than 3 inches. A turning paddle is used to enhance a rotary motion inside a cylindrical chamber. These systems are widely used on small-, medium-, and largescale facilities

The other type of vortex design has a flat bottom, strategic baffles, and a small opening to the center grit hopper. This is referred to as a forced vortex system, available options are the Smith & Loveless® Pista®Grit system (see **Figure 9.14**) and the WesTech® Vortex Grit Chamber. The manufacturers claim that grit settles in the sloped influent channel to the grit chamber so that grit is concentrated near the floor of the chamber upon entry. The forced vortex in the chamber then creates a hydraulic flow along the surface, down the face of the chamber at the outer edge, then in toward the center essentially pushing the grit already concentrated at the bottom toward the center where it enters the hopper.

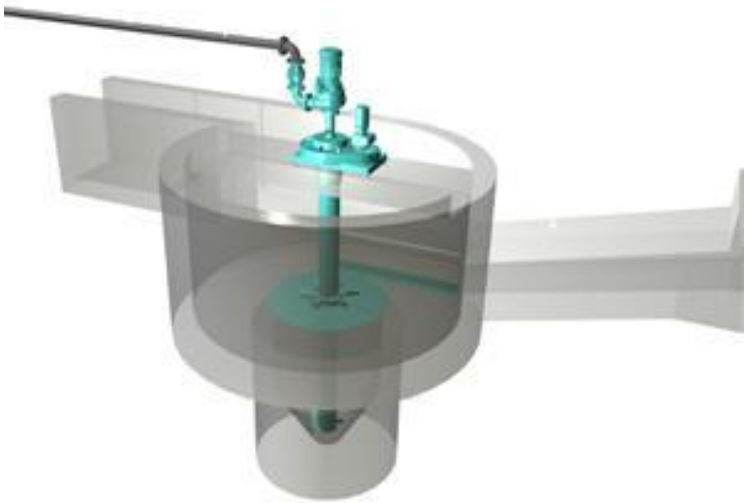


Figure 9.14: Pista®Grit Chamber (Source: Smith & Loveless®)

Sizing of a forced vortex system is proprietary in nature and is provided by the equipment manufacturer. Sizing is reported to be based on physical testing and particle movement modeling rather than by empirical formulas that can be independently verified. Sizing must inherently be based on trust in these testing results and past performance.

Conversations with the manufacturer of the current system indicate that the current grit chamber does not meet the current standards to act as a forced vortex system. However, the manufacturer indicated that additional baffling could be added to the existing grit chamber to increase the removal efficiency of the

system in lieu of replacement of the existing grit chamber. While the system would not achieve the same removal efficiencies of a forced vortex system, it would provide additional removal over the current design.

9.3.5 Grit Pumping and Dewatering

9.3.5.1 Treatment Objectives

Once the grit is removed from the raw sewage flow stream, the collected grit solution must be transferred to a grit separation and washing process. The grit is separated so that the organic and liquid waste can be returned to the wastewater stream. Grit is then washed prior to disposal in a waste dumpster. The grit separation design is just as critical as the grit removal technology; if not done properly, grit is simply returned to the process liquid stream which reduces the effectiveness of the system.

The grit slurry generated by the grit removal system will be pumped to the separator/washer by the grit pump in an on/off grit cycle. Typically, a recessed impeller pump is used to pump the grit to the separator/washer. The separator/washer and grit pump will need to be sized to the peak flow of the system. The separator/washer conditions the grit for off-site disposal by concentrating and washing the separated grit. Although many manufacturers offer a package of grit removal equipment and grit separator/washing equipment, it is Hazen's experience that it need not be specified that the same manufacturer provide both. Several types of grit separation/washer systems are described in the following sections.

9.3.5.2 Hydrocyclone and Screw Classifier

Hydrocyclones consist of a centrifugal separator and concentrator. They remove the heavier particles from the lighter particles in the wastewater stream and create a separate, more concentrated stream of grit. A diagram of the process is shown in **Figure 9.15**. This is the system currently installed at the WRF. The concentrated grit is then transported to a settling tank with a screw classifier which washes and dewaters the grit to remove the organic content. This serves to reduce both the organic load of the waste stream and the odors of the collected grit. **Figure 9.16** represents the schematic of a typical screw classifier. The grit exiting the hydrocyclone is introduced to the classifier, where larger grit settles and finer grit and organics stay in suspension and are discharged over the weir. The settled larger grit is then moved up an incline by the screw classifier. The grit is dewatered as it moves up the perforated plate conveyor. The grit is then transferred to a receiving bin for disposal.

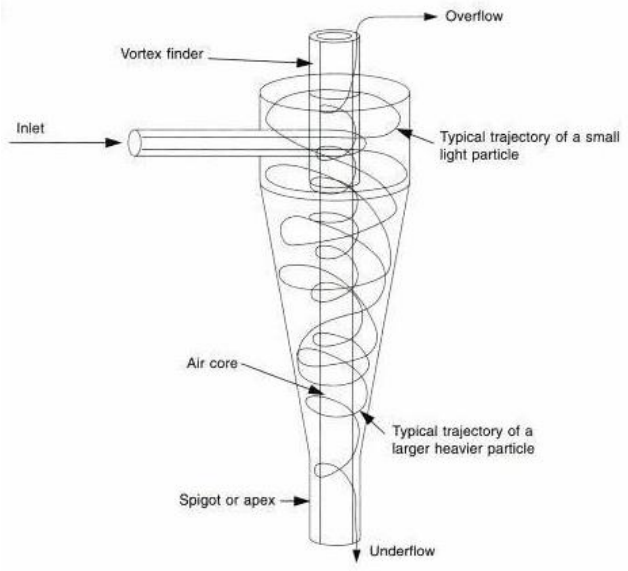


Figure 9.15: Hydrocyclone Operation (Source: Lenntech)

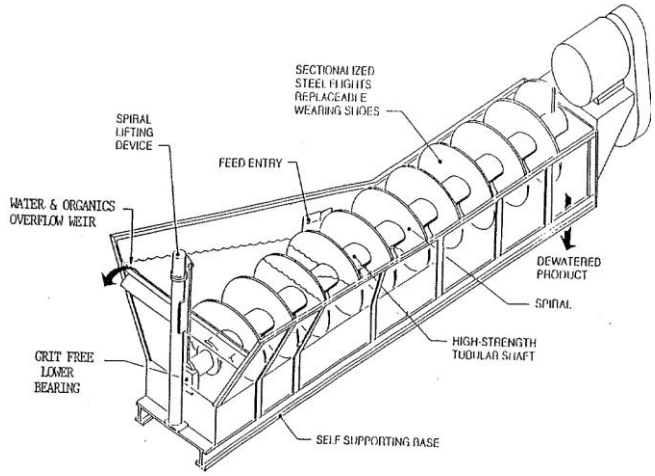


Figure 9.16: Screw Classifier

9.3.5.3 Eutek SlurryCup™ and GritSnail®

The Eutek SlurryCup™ and GritSnail® by Hydro International can reportedly remove up to 90 percent and 95 percent of 75-micron and larger grit particles, respectively. The Eutek SlurryCup™ uses a combination of open free vortex and the boundary layer effect to capture, classify and remove grit. The concentrated grit is then transported from the SlurryCup™ to a quiescent dewatering escalator, known as a GritSnail®. As the escalator slowly moves the grit slurry up to the top of the GritSnail®, the water drains out by gravity. When the dewatered grit reaches the top, it is scraped off the belt cleats and discharges

into a disposal container. **Figure 9.17** shows a photo of the Eutek SlurryCup™ and GritSnail® system. Odor control covers can be added to the GritSnail® system to prevent odor issues.



Figure 9.17: Eutek SlurryCup™ and GritSnail® (Source: Hydro International)

9.3.5.4 Hydrocyclone and Grit Washer (Coanda Grit Classifier)

The Coanda Grit Classifier by Huber claims a grit removal efficiency of up to 95 percent of grit sized 200 micron and larger when operated at optimum flow and organic content of less than 5 percent. The unit combines the typical functions of a grit classifier, washer and conveyor into a single package. The wastewater stream flows into a vortex chamber at the top of the unit where a rotary motion is induced. Automatic air intake takes place through the centrically located drilling hole on the vortex chamber. The flow then passes into trumpet-shaped Coanda Tulip. The outer, faster-moving, organically-laden wastewater flowing in from the vortex chamber adheres to the tulip wall and is diverted to the horizontal, below-water-surface weir by the Coanda-Effect (where a liquid flow adheres to the contour of a curved surface even when the surface moves away from the original flow direction). Gravity and inertia cause the grit particles to settle out of the flow and move toward the bottom of the unit. Wash water is fed from the bottom of the unit, through a perforated plate and diaphragm, creating a fluidized bed of grit particles which separates the denser grit from the less dense organics which have adhered to the grit particles. The collected grit is swept into a central hopper where it drops into the classifier section of the unit. The classifier screw transports the clean grit up an inclined plane where it is dewatered under gravity before discharge into a dumpster. **Figure 9.18** represents the schematic of the Coanda Grit Classifier.

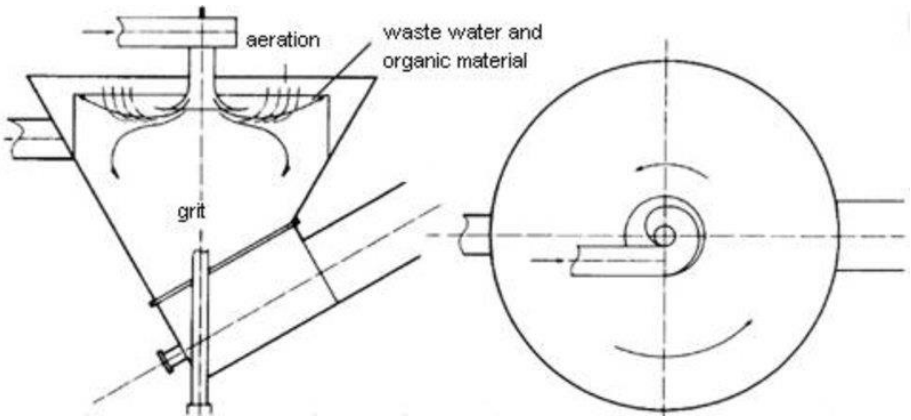


Figure 9.18: Coanda Grit Classifier

9.4 Current WRF Headworks Implementation Plan

A description of the current state of the headworks and the Town's established plan is presented. During the course of the Modernization Study, an implementation plan was developed with consultation between the Town and Hazen.

9.4.1 Fine Screening

The Town is currently in the process of design and installation of a 3/8-inch multi-rake automatic screen at the influent wet well. This is tentatively scheduled for the summer of 2019. Until that screen is operable, the Parkson™ Aquaguard® screen, located at the headworks, serves as the only screen in front of the secondary processes. After the multi-rake screen is installed the characteristics of the flow and debris entering the headworks could be substantially different. The replacement of the Parkson™ screen will be formally designed once the characteristics of the new waste stream are determined. There is some speculation that the 3/8-inch screen will perhaps render the replacement of the Parkson™ screen unnecessary. The Town could then consider replacement of the Parkson™ screen with a finer screen.

As part of the Draft Feasibility Study Report (August 2016), Hazen prepared a table regarding potential capital, O&M, and 20-year net present value costs for a variety of potential headworks fine screens. This information is included below as **Table 9.4** for the Town's future budgeting needs should the Town plan for replacement of the screen.

Table 9.4: Headworks Screen Construction, and Operation and Maintenance Costs

No	Alternative	Construction Costs ($) [1]	Annual O&M Cost ($) [2]	20-Yr Net Present Value ($) [3]
1	Parkson™ Aquaguard ®	$376,000	$11,130	$542,000
2	Huber StepScreen	$760,000	$10,720	$919,000
3	Huber Escamax	$827,000	$11,110	$992,000
4	Huber Rakemax	$708,000	$7,090	$813,000
5	Headworks MS2	$801,000	$9,660	$945,000
6	Smith & Loveless® Mark IX-A	$605,000	$5,760	$691,000
7	Duperon Flexrake	$965,000	$7,110	$1,071,000
8	Aqualitec Raketec	$603,000	$6,670	$702,000

[1] Construction cost includes labor, material, and equipment costs require to build each alternative, plus Div. 1 requirements, contractor overhead, profit and bonding, and a design contingency
[2] O&M costs include routine maintenance, recommended spare parts and cost of electricity
[3] Net present value is the sum of the construction cost and the total O&M costs over a 20-year period at a 3% discount rate

If the headworks screen is replaced, it is anticipated that a screenings washer/compactor would be installed. The timing should coincide with the restoration and full operation of Aeration Basin A.

9.4.2 Grit Handling

The Town's existing Pista®Grit stirred vortex system was rehabilitated in 2015 with all new Smith and Loveless® Parts, a new control panel, and a new vacuum pump (see **Figure 9.6**). Additional improvements to the Pista®Grit system are not anticipated for the foreseeable future, perhaps with the exception of potential rehabilitation of the concrete surface of the previously existing grit chamber.

The current grit classifier and grit auger are currently planned for replacement in the summer of 2019. Hazen has already prepared a draft feasibility study outlining the options and costs for this project and the Town has budgeted for this in the current Capital Improvement Plan. Based on the feasibility study, the estimated capital and construction cost of the replacement grit classifier and grit auger is approximately $340,000.

Due to their current deteriorated state, replacing the grit classifier and grit auger is a priority. After this replacement is complete, the Town can consider making minor improvements to the current vortex grit system by adding baffling. For budgetary purposes the estimated cost of installing the additional baffling is on the order of $30,000.

9.4.3 Concrete Chamber, Channels, and Parshall Flume.

When any element of the headworks is replaced, the Town may opt to either resurface/rehabilitate or completely replace the aging concrete of the headworks (see **Figure 9.8**). For budgetary purposes, the cost for rehabilitating the various concrete sections of the headworks are included in **Table 9.5**.

Table 9.5: Headworks Concrete Repair and Replacement 2018 Capital Costs

Concrete Portion of Headworks	Refurbishment Cost ($) [2]	Replacement Cost ($) [2]
Screen Channel Portion	$55,000	$150,000
Grit Chamber Interior and Exterior Surface	$79,000	$250,000
Parshall Flume Portion	$59,000	$250,000

[1] Refurbishment costs include costs for materials, labor, mobilization and demobilization
[2] Replacement costs include demolition of the structure, excavation for new structures (as needed), pouring concrete, coating the interior, and minor pavement repairs. Costs do not include bypassing the Headworks (see below).

Costs for bypass pumping area an important consideration. The cost for bypassing the headworks is approximately $107,500 per month. For a three-month construction period, this amounts to approximately $322,500, a significant addition to the replacement cost. It was assumed the bypass pumps run 24 hours per day, 7 days per week, with a diesel generator and an operating engineer present at all times. Bypass pumps are sized for a peak flow of 5 MGD. Bypass is provided to ensure flow is maintained to the secondary treatment process; screening of the bypassed flow is not provided in these costs, but a temporary system could be implemented depending on the cost, duration of construction, and other requirements specific to the WRF. To reduce bypass needs, the Town may opt to install a permanent bypass, or prepare the existing screening channel for a new screen installation while the grit removal system is being installed.

9.4.4 Implementation Plan Summary

As previously mentioned, the Town rehabilitated the existing Pista®Grit vortex system in 2015. In addition, the Town is currently in the process of designing and installing a 3/8-inch multi-rake automatic screen at the influent wet well and has plans to replace the current grit classifier and auger during the summer of 2019.

After the influent pump station screen has been installed, its performance should be evaluated. The performance of the existing headworks screen and vortex grit system should be evaluated based on the characteristics of the new waste stream. The Town should then decide whether headworks screening is needed and if the current grit removal system is sufficient. If either system is to be replaced, the new waste stream should be considered in its design.

10. Ultraviolet Disinfection

In 2001 the Town replaced the chlorine gas disinfection system with a new ultraviolet (UV) disinfection system to produce tertiary effluent. The transition to UV disinfection provided an improvement in safety and a reduced potential for the formation of chlorinated disinfection by products. Currently, the Town still uses hypochlorite to apply a chlorine residual to a small portion of the effluent that is used by the high school and Fire Station No. 2.

The Town has been satisfied with the existing UV system, and plans to continue to use UV disinfection in the foreseeable future. This section presents an evaluation of the existing UV disinfection system performance, energy efficiency and overall operation and maintenance (O&M) costs and considers replacement with newer technologies.

10.1 Introduction

The WRF currently has an Ozonia 40 VLS low-pressure, vertical lamp, UV system that was installed in 2001. Information on the existing system is included in **Appendix J**. This evaluation reviews the existing UV system and compares three new UV systems that could be considered for replacement of the existing UV system. The review evaluates the original and present design criteria and estimates annual O&M costs of the existing UV system. This evaluation also provides an estimate of the lifecycle cost (equipment and O&M) for the three replacement UV systems. The purpose of this evaluation is to compare the existing system's cost and performance against currently available competitive options and configurations, not to suggest a particular UV disinfection manufacturer. Therefore, not all UV manufacturers are represented in this evaluation.

10.2 Existing UV System

The existing UV disinfection system was supplied by Ondeo Degremont (ODI) which was subsequently rebranded as Ozonia and is now distributed by Suez. The equipment is still branded as "Ozonia" and for simplicity both existing and new technology will be referred to as "Ozonia" equipment. The current Ozonia UV equipment is similar to the installed equipment, although the current generation includes a number of upgrades that address original design and operation issues. Additionally, the interior surfaces of the UV channels were recently resurfaced to reduce potential for coliform violations.

The UV influent flow enters in a common influent control box and is then directed into one of the two channels. Each channel includes a perforated influent baffle plate and channel level control gate. The system consists of 2 parallel channels with 10 banks of lamps per channel. Each bank is divided into two modules which are bolted together, with a lifting plate. Each module has 5 rows (in the direction of flow) with 8 lamps across the flow) for a total of 40 lamps per module. This terminology is illustrated in **Figure 10.1**. Photos of the existing UV channel are shown in **Figure 10.2 and Figure 10.3.**

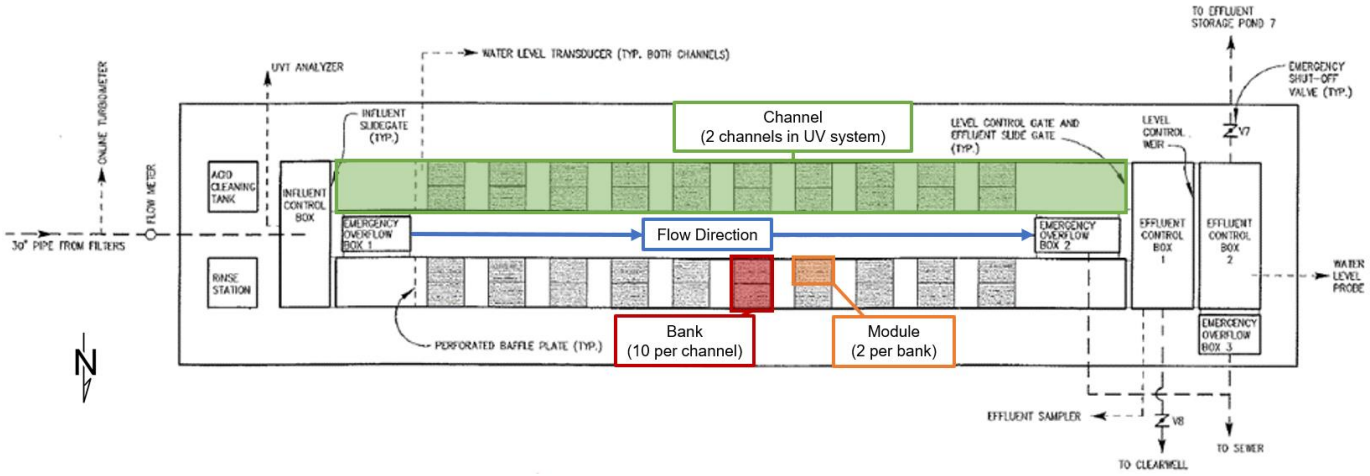


Figure 10.1: Illustration of UV Disinfection Terminology



Figure 10.2: Existing WRF UV Disinfection Facility



Figure 10.3: (L) Side View of UV Bank with 5 Lamps per Row and (R) Back View of UV Bank with 16 Lamps per Row

Table 10.1: Existing UV System Configuration

Parameters	Ozonia 40VLS
Number of Channels	2
Number of Banks per Channel	10
Number of Modules per Bank	2
Number of Lamps per Module	40
Number of Rows per Bank	5
Number of Lamps per Row (each Bank)	16
Total Number of Lamps	1600
Number of UV Sensors	20
Number of Level Gates	2
Lamp wattage	0.075 kW per lamp
Total Power Consumption	120.0 kW

Ancillary facilities include bridge crane, cleaning tank and rinse station, air blower, and two power distribution / control panels. A plan, section, and piping information are included in **Appendix J**.

Effluent from the UV disinfection process may be routed through the clear well and directly to the Geysers project, high school, and fire station, or it may be stored in effluent storage ponds for later reuse (primarily irrigation) or discharge to Mark West Creek.

The original design parameters are detailed in **Table 10.2**. This information was extracted from the Whitley Burchett and Associates MS2 Bioassay Report dated January 2002 and the Brelje & Race Final Engineering Report, dated August 2002.

Table 10.2: Original Design Parameters

Design Parameter	Original Design
Peak Flow Rate (both channels)	8.6 MGD [1]
Average Daily Flow Rate	2.25 MGD
Minimum UV Transmittance	70 %
Maximum Total Suspended Solids	5 mg/L
Minimum Effluent Temperature	10 ºC
End of Lamp Life Factor (after 13,000 hours)	0.70
Fouling Factor	0.80
UV Dose	140.0 mJ/cm^2
Number of Channels	2
Redundancy	1 Bank per Channel
Permit Limit Total Coliform, 7-day median	2.2 MPN/100 mL
Total Coliform, one sample limit in 30-day period	23 MPN/100 mL
Total Coliform, not to exceed limit	240 MPN/100 mL

[1] Note that this is the original design. The approved flow rate is 10.1 MGD for both channels.

The flow rate listed in the table is the original design criteria. The actual approved flow rate per channel is 5.1 MGD and 10.1 MGD for 2 channels based on the UV Disinfection Engineering Report (Brelje & Race, 2002). Additional UV disinfection system design criteria are included in **Appendix J**.

The UV system is typically operated such that one channel is in service at a time. Operations staff alternates the channel that is in service each month. Banks are acid cleaned monthly on the out of service channel. For the channel that is in service, a baseline number of lamps are turned on all the times. Additional lamps are turned on (beginning from the downstream module) as flow increases or UVT decreases. Lamps are replaced after approximately 13,000 hours of active time. Quartz sleeves are changed approximately every seven years.

10.3 Current Design Criteria

This section discusses the critical parameters used to size a replacement UV disinfection system: design flow, UV transmittance (UVT) and UV dose. Each parameter is discussed below, and the final design parameters are presented in **Section 10.3.4.**

10.3.1 Effluent Flow vs. Design Flow

The WRF average daily effluent flows were recorded from January 2012 to December 2017 and the data is presented in **Figure 10.4**. The typical maximum average daily effluent flow rate recorded over the

period was approximately 5 MGD. A peak flow of over 6 MGD has been successfully treated during wet weather. The average effluent flow rate over this period was 1.9 MGD. Note the facility's design average dry weather flow rate is 2.25 MGD with a peak design wet weather flow of 7.2 MGD. Therefore, the system is well within design range.

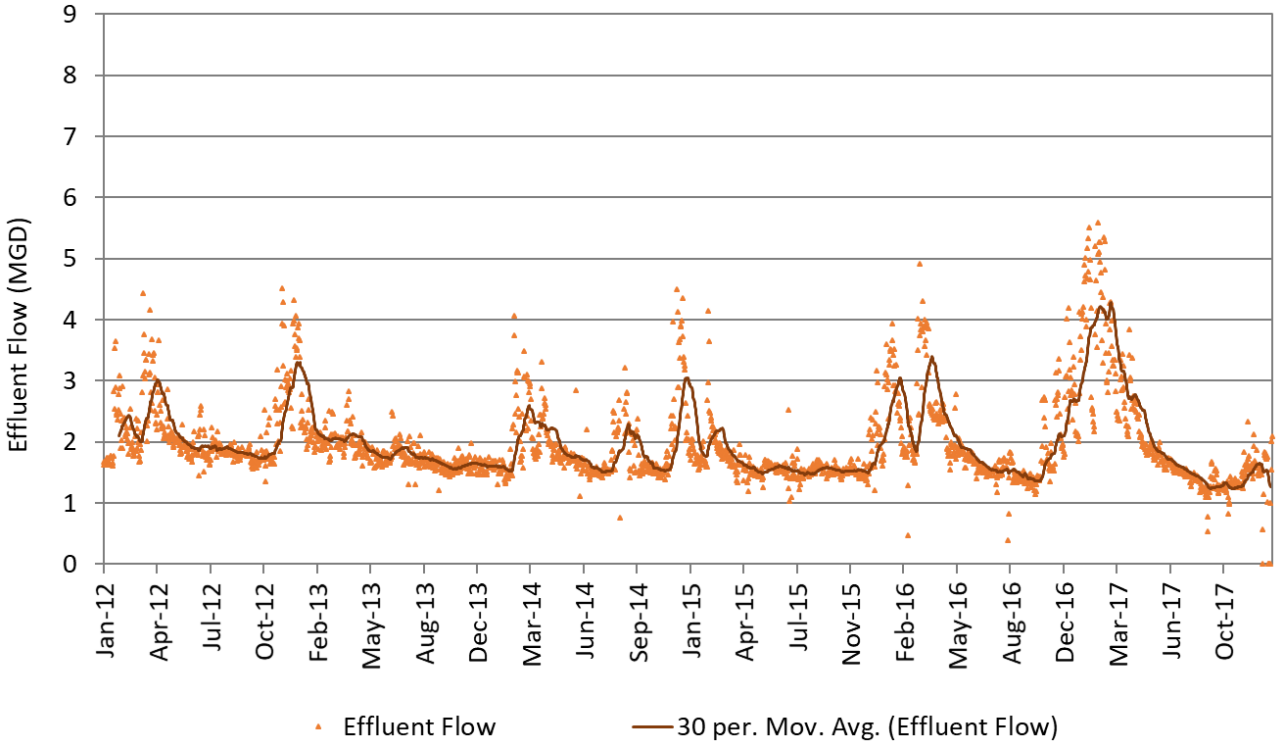


Figure 10.4: WRF Effluent Flow

10.3.2 UV Transmittance

The daily average and minimum UVT of the tertiary wastewater effluent were recorded from March 2013 to December 2016 and the data is presented in **Figure 10.5**. Over this period, the average UVT was 74.8 percent and the lower 10th percentile was 67.2 percent. The lower 10th percentile is commonly used as a conservative design value for new UV systems, as is used in this analysis.

Figure 10.5 shows that the daily average UVT for 2013 through 2016. The general trend has been a declining UVT over the year, and a rebound each spring. UVT is the most critical design parameter for a UV system. A 5 percent difference in the UVT can result is a 20 percent change in the required number of lamps for a UV system – which results in higher capital costs. This also affects the UV system's electrical and O&M costs since more lamps are required to be operating to provide the necessary dose. The UVT decline in 2015 seems to be an anomaly. The rate of the decline is steeper than in previous years and was continuously below 70 percent for the period of April to May 2015. Reasons for the decline are not clear, but UVT trends may have been impacted by drought conditions or a re-evaluation of chemical usage

which resulted in reduced chemical treatment in the AWT process (due to precipitation issues). UVT seemed to recover in 2016 and follows previous trends.

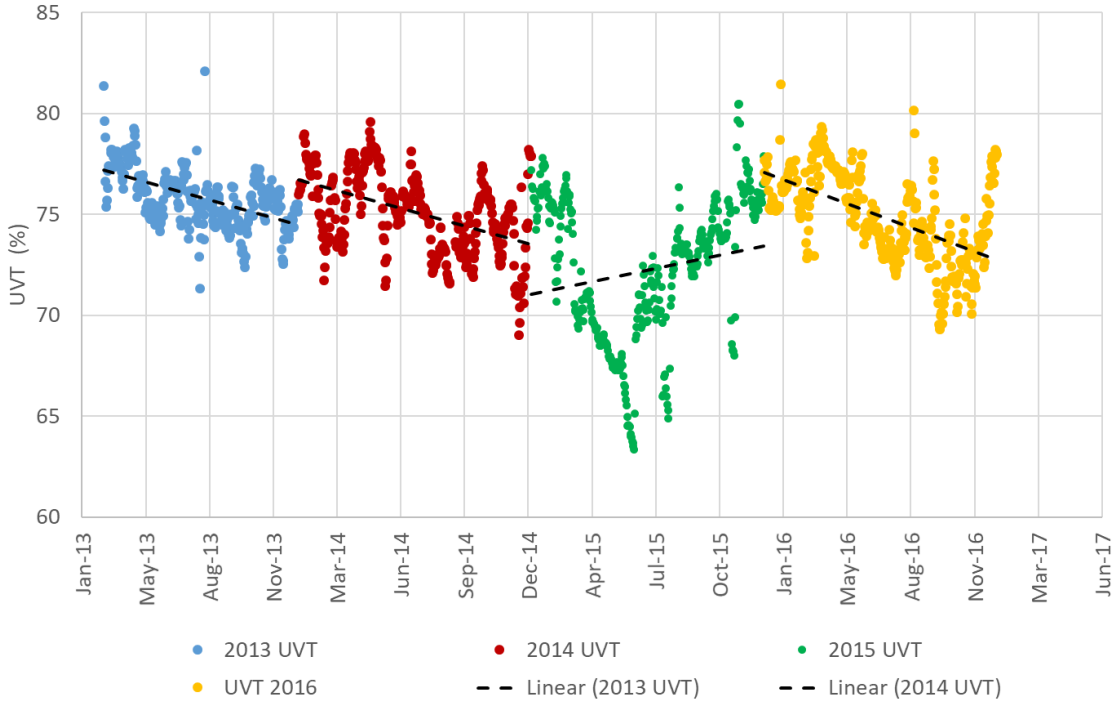


Figure 10.5: WRF Effluent UV Transmittance

10.3.3 UV Dose

UV dose is a function of the number of operating lamps, the power emitted from the lamps, the UVT of the water being treated, and the velocity through the reactor. The existing UV system was originally designed based on the 1993 edition of the California UV Disinfection Guidelines for Wastewater Reclamation. However, the 2000 National Water Research Institute (NWRI) Ultraviolet Disinfection Guidelines for Drinking Water and Water Reuse (UV Guidelines) were used as the basis for testing and rating the system during commissioning. The UV knowledge base has continued to evolve since the original UV system was approved. The latest edition of the NWRI UV Guidelines was published in 2012 and while the MS2 (an indicator bacteriophage) dose levels are the same for both editions, 100 mJ/cm^2, the way that the dose is determined is now more conservative. Therefore, systems operating based on the latest edition will require approximately 10 percent more power (thus more lamps) when compared to the older edition. This is evident when comparing the total power consumption of the existing UV system and potential replacement UV systems in **Section 10.4** and **Section 10.5**.

10.3.4 Summary of Design Parameters

Preliminary design parameters for a potential replacement system are outlined in **Table 10.3**. The End of Lamp Life and Fouling Factors as well as the permit limits remain the same as the original design parameters. These parameters are used in the remainder of this section when comparing the existing

system to potential newly available alternatives. Note that the aforementioned 2012 dose guidelines should be used for newly installed systems.

Table 10.3: Recommended Replacement UV Disinfection System Design Parameters

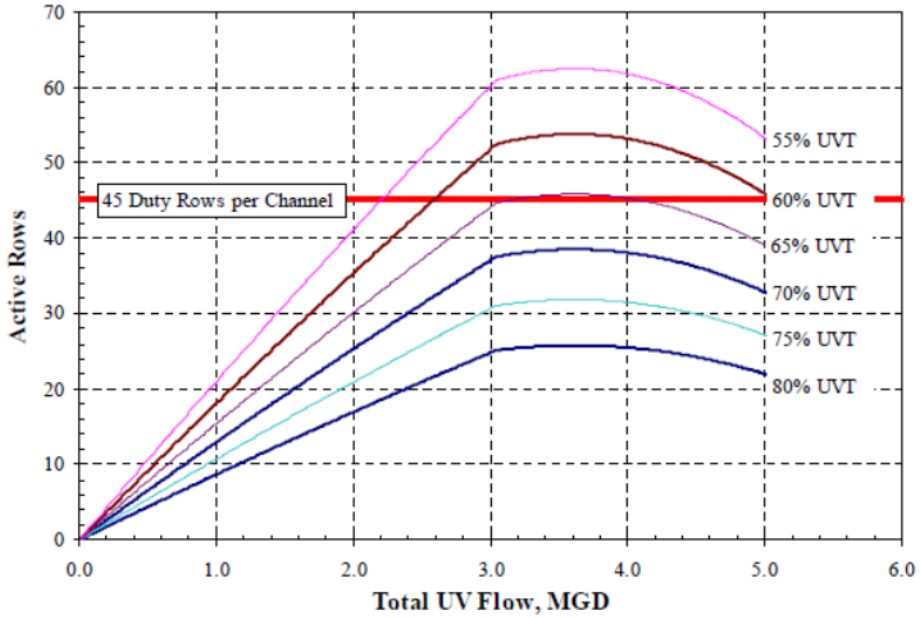
Design Parameter[1]	Present Design
Peak Flow Rate	8.6 MGD
Average Daily Flow Rate	2.25 MGD
Average UV Transmittance	75 %
Design UV Transmittance [2]	67 %
Average Total Suspended Solids	1.8 mg/L
Minimum Effluent Temperature	10 ºC
Min. MS2 RED, per NWRI 2012	100.0 mJ/cm^2
End of Lamp Life Factor (after 13,000 hours)	0.70
Fouling Factor	0.80
Number of Channels or Trains	2
Redundancy	1 Bank per Channel
Quartz Sleeve Replacement	Every 5 years

[1]All design parameters based on original (ODI) design drawings and supplier submittal unless otherwise specified
[2]Based on lower 10-percentile values as outlined in 2012 guidelines

10.4 Existing System O&M Costs

For this analysis, an estimate of the existing system O&M costs was prepared based on information from the Town and other recent evaluations. There was a recently prepared evaluation of remaining life and costs of an identical Ozonia 40 VLS UV system installed at the Livermore Water Reclamation Plant in the City of Livermore, CA. The Livermore system is a similar sized and age system to the Town's UV system. It was installed in 2001. The Livermore system has 2 channels and 1440 lamps, compared to 1600 installed in the Town's system. The Livermore report evaluation is used as a basis for this evaluation of operations costs where information was not available for the Town's system. For Livermore, the operation and maintenance costs were tracked for two years (July 2011 to July 2013). Livermore O&M costs includes annual replacements of lamps, ballasts and quartz sleeves as well as miscellaneous mechanical and electrical components. Other O&M costs included labor and energy consumption. The costs at Livermore were averaged on an annual basis and were adjusted/normalized to reflect the different lamps counts between the two UV systems.

The costs were also based on the Town's facility average conditions as detailed in **Table 10.3**. The Town replaces quartz lamp sleeves every 7 years, however the vendor recommended replacement of sleeves every 5 years was used, as it is more conservative. To calculate the energy costs, an estimate of the number of active rows of UV lamps for the average conditions was developed. The Brelje & Race Report included capacity curves based on different UVTs, shown in **Figure 10.6.** For the average conditions, 2 MGD and 75 percent UVT, a total of 20 active rows are required to achieve the required UV dose. Each row in a channel is comprised of 16 UV lamps; therefore, a total of 320 UV lamps are required for operation. Based on a per lamp power consumption of 75 W, the average power consumption used for the O&M cost is 24 kW.



[1]Curves indicate the number of rows required for various UVTs for a single channel

Figure 10.6: Capacity Curves for Ozonia 40 VLS (WRF)

The estimated O&M costs for the Ozonia 40 VLS installed at the WRF are detailed in **Table 10.4**. The total annual O&M cost is estimated at approximately $92,100. The estimate is based on the information from the Town supplemented with data from the Livermore study. Compared to the Livermore system, the overall O&M cost is reasonable for a system of comparable size and age.

Table 10.4: Annual O&M Costs of Existing UV System

Cost Component	Annual Values
Lamp Replacement	
Average number of lamps replaced in one year[1]	320
Lamp Cost (per Lamp)	$27.95
Total Lamp Replacement Cost	$8,944
Ballast Replacement	
Average number of ballasts replaced in one year[1]	24
Ballast Cost, (per Ballast)	$178
Total Ballast Replacement Cost[2]	$4,272
Sleeve Replacement	
Average number of sleeves replaced in one year[2]	240
Sleeve Cost (per Sleeve)	$26
Total Sleeve Replacement Cost	$6,240
Miscellaneous (Mechanical and Electrical Component) Replacement	
Total Miscellaneous Component Replacement Costs	$5,621
Total Annual UV Parts Replacement Cost[2] *Including: lamp, ballast and sleeve replacement plus miscellaneous replacement costs.*	$25,100
Labor Cost	
Labor Rate (per hour)	$75
Estimated time spent replacing UV parts[2]	16 hrs/mo
Estimated time spent cleaning UV channels/sleeves	20 hrs/mo
Total Annual Labor Cost	$37,600
Power Cost	
Electricity Rate	$0.14 / kWh
Energy consumed by UV system	210,240 kWh/yr
Total Annual Power Cost	$29,400
Total Estimated Annual Operation and Maintenance Cost *Including: replacement parts, labor, and power*	$92,100

[1]Based on average vendor lamp life.
[2]Based on Livermore operation and maintenance evaluation.

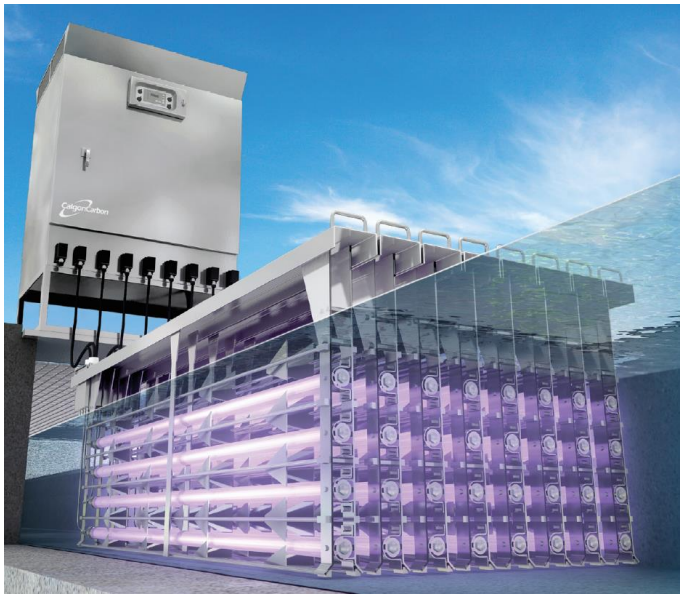
10.5 Potential Replacement UV Systems

Three UV equipment suppliers were contacted and requested to submit a budgetary proposal based on the proposed design parameters listed earlier; Ozonia (existing in-channel system supplier), Engineered Treatment Systems (an enclosed UV system) and an alternative in-channel system by Calgon Carbon. These suppliers where chosen since they all provide equipment that has been validated to meet NWRI guidelines, are Title 22 approved, and have been used to retrofit s UV systems similar to the Town's existing UV facility. Other open channel UV equipment suppliers including Trojan, and Wedeco could also be considered during replacement design They are considered represented categorically in this section, as they are similar in design and cost to the Calgon Carbon system. Pictures of these technologies are provided in **Figure 10.7 to Figure 10.9.**



Source: http://www.ozonia.com/uv.php#aqr40ho

Figure 10.7: Example Ozonia Module



Source: https://www.calgoncarbon.com/app/uploads/C3500.pdf

Figure 10.8: Example Calgon Carbon Open Channel System



Source: http://www.evoqua.com/en/brands/ETS_UV/Pages/ets-uvlw.aspx

Figure 10.9: Example ETS Closed Vessel Technology

All manufacturers' products operate on the same principles, but have different configurations, and different maintenance and replacement needs. The two in-channel systems would be similar to the existing system in that the existing open channels may be reused. The Ozonia equipment would readily replace the existing equipment. The equipment is physically similar; however, control modifications would be required. The Calgon equipment would require changes to the channel, and electrical/control equipment. The in-vessel system (ETS) provides another alternative, although it would provide more engineering challenges for the WRF's current gravity flow system (both in and out of the UV channels) due to the higher headloss through the vessels. This section compares the three equipment offerings and reviews equipment costs.

10.5.1 Replacement System Comparison

The replacement system configurations including the total lamp counts and power consumption are outlined in **Table 10.5**. The main differences between these UV systems are lamp power, lamp orientation, and system configuration. The Ozonia 40HO is an open-channel system that utilizes 155W (each) lamps oriented vertically and perpendicular to the direction of flow, similar to the Town's existing UV system. The Calgon Carbon $C^3$500D is also an open-channel system that utilizes 520W (each) lamps that are oriented horizontal and parallel to the direction of flow. The ETS 45800-30 is a closed vessel system that utilizes 800W (each) lamps, oriented parallel to the direction of flow in an L-shaped reactor (illustrate in **Figure 10.9**). All of these UV systems are modular, referring to individual racks or banks of lamps. The replacement configurations and specifications outlined in the **Table 10.5** reflect UV systems that are sized to match the performance of the current UV system at the WRF.

Table 10.5: Replacement UV System Configurations

Parameters	Ozonia40HO Open-Channel	Calgon Carbon $C^3$500D Open-Channel	ETS 45800-30 Closed Vessel
Design Factors			
End of Lamp Life Factor	0.90	0.85	0.85
Fouling Factor	0.80	0.90	0.81
Configuration			
Number of Channels or Trains	2	2	7
Number of Duty Banks per Channel	7	3	1
Number of Standby Banks per Channel	1	1	0
Total Number of Banks	8	4	1
Number of Modules per Bank	2	5	1
Number of Lamps per Module	40	7	45
Total Number of Lamps	1280	280	315
Number of UV Sensors	16	8	7
Power Rating of Each Lamp (W)	155	520	800
Number of Power Distribution Centers	2	8	4
Number of System Control Centers	1	1	1
Number of Weirs	2	2	N/A
Total Power Consumption (kW)	220.2	161.0	283.5

The replacement UV systems power consumptions range from 161 to 283.5 kW. The replacement systems are generally more efficient at achieving a particular UV dose than the existing UV system. The newer technologies use more efficient lamps and reactor geometries. In addition, the controls for newer

systems allow for more refined control, and thus may require less lamps during flows below the maximum design values. However, as previously noted, the latest version (2012) of the NWRI UV Guidelines establishes a more conservative method to determine the MS2 dose of a UV system. Therefore, replacement systems, which are designed to meet these updated guidelines, will require more energy consumption than the current WRF UV System (which is rated at 120 kW) to achieve the more conservative dose.

10.5.2 Cost Comparison

The equipment costs and operational and maintenance costs for the different systems are compared below.

10.5.2.1 Equipment and Installation Costs

The equipment and installation cost of the three alternate replacement UV systems is compared in **Table 10.6**. Both Ozonia and Calgon Carbon have the lowest equipment purchase costs, whereas the ETS system equipment cost is substantially higher (approximately double). The ETS system costs are higher due to the stainless steel enclosed reactor configuration. The other two systems are typically placed within a concrete channel, similar to the existing system. The replacement costs using the Ozonia equipment will be the lowest due to the minimal cost to modify the existing channels, controls and support facilities. The cost of a new in-channel UV system, such as the Calgon Carbon system will be higher due to the cost of modifications to the existing concrete channels. The cost of the enclosed vessel alternative will be the highest due to the extensive modifications and overall restructuring of the UV channels that would be required. As mentioned earlier, the ETS system's closed vessel configuration could provide engineering challenges to the current influent and effluent gravity flow arrangement at the WRF's UV channel. The installation cost of new UV systems generally range from 1.5 to 3 times the equipment cost.

Table 10.6: Replacement UV System Equipment and Installation Costs

Description	Ozonia40HO Open-Channel	ETS 45800-30 Closed Vessel	Calgon Carbon $C^3$500D Open-Channel
UV System Equipment Cost [1]	$960,000	$1,970,000	$986,000
Replacement Cost Factor [2]	1.5X	3X	2X
UV System Installation Cost	$1,440,000	$5,910,000	$1,970,000

[1] *Based on August 2015 vendor proposals.*
[2] *Estimated based on past experience and cost of other UV system installations.*

10.5.2.2 Operation and Maintenance Costs

The estimated annual operation and maintenance (O&M) cost basis for each replacement system are presented in **Table 10.7**. The O&M costs include periodic replacement costs of system components, labor costs associated with replacing the system components and cleaning the UV banks, and energy costs while operating at average flow conditions. The replacement costs for the individual components were provided by the manufacturers. The key factors for the developing the O&M costs are also presented in the table. The Ozonia system has the highest O&M cost estimate, while the Calgon Carbon system has the

lowest O&M cost estimate. In general, the Calgon Carbon system is a higher efficiency system than the Ozonia system and requires fewer lamps and less power; thus, it will have the lowest O&M costs. The ETS system is designed for easy maintenance. These units are designed so that the bulbs can be easily removed and cleaned. It also has lower O&M costs than the Ozonia system. O&M costs of the existing system are included in **Section 10.5.3.**

Table 10.7: Replacement UV System Annual O&M Cost Estimates

Description	Ozonia40HO Open-Channel	ETS 45800-30 Closed Vessel	Calgon Carbon C^{3}500D Open-Channel
Lamp Replacement (per Lamp)	$30	$440	$250
Ballast Replacement (per Ballast)	$200	$490	$400
Wiper Replacement (per Wiper)	$8	$25	$70
Sleeve Replacement (per Sleeve)	$30	$360	$90
Chemical Usage (unit)	$250	$250	$250
UV Sensor Replacement (unit)	$950	$850	$1,300
Sensor Calibration	$0	$600	$933
Electricity Rate	$0.14 per kWh		
Labor Rate	$75.00 per hour		
Cost Subtotals			
Parts and Replacement Estimate	$60,400	$80,700	$37,100
Labor Cost Estimate	$37,000	$25,800	$16,800
Energy Cost Estimate	$74,400	$45,500	$37,700
Total Estimate Annual O&M Costs	**$172,000**	**$152,000**	**$91,600**

10.5.3 Replacement System Life Cycle Cost Comparison

The replacement system cost comparison includes initial installation costs and estimated annual O&M costs. The summary of the lifecycle cost evaluation for each UV system is presented in **Table 10.8**. This analysis assumed a typical life cycle of approximately 15 years for UV equipment. The existing UV system's O&M cost is also compared to that of the replacement UV systems but as the existing UV system ages it in anticipated that the annual O&M costs will increase over time, subsequently increasing the life cycle cost of the system.

Table 10.8: Replacement UV Disinfection System Life Cycle Cost

Equipment	Installation Cost	Present Worth O&M Cost [1]	Present Worth Cost [2]
Ozonia 40HO	$1,440,000	$1,670,000	$4,490,000
ETS 45800-30	$5,910,000	$1,480,000	$9,330,000
Calgon Carbon C^{3}500D	$1,970,000	$889,000	$3,820,000
Existing UV System			
Ozonia 40 VLS [3]	N/A	$894,000	N/A

[1]Present worth of O&M cost is based on 15-year O&M at 6%
[2]Present worth cost based on installation cost plus present worth of annual O&M cost.
[3]Based on the annual O&M estimate in Table 10.4.

The Ozonia UV system fits within the existing UV channels without major modifications and will therefore have the lowest installation cost. The ETS UV system will require more modification since the proposed equipment would include a total of seven pressurized closed vessel in parallel, and therefore has the highest installation costs.

The Calgon Carbon $C^3$500D has the lowest 15-year present worth O&M cost, which offsets the higher construction costs. Overall it will have the lowest present worth costs, approximately $670,000 lower than the Ozonia 40HO. The ETS system has the highest life cycle cost. The Calgon Carbon UV system fits within the existing UV channels as well; however, channel reduction baffles would need to be installed to reduce the channel width so that the equipment will fit without channel modifications. In addition, a new level control device or a modification to the existing controls, would be required since horizontal and vertical UV systems have different channel level tolerances. It should be noted that fine tuning the level controls during the startup process for the existing UV system required significant effort. Hydraulics of the replacement system need to be considered to ensure the replacement system is compatible with the WRF hydraulics. Additional analysis would be required to determine whether the ETS system could continue the gravity flow through process that the current system employs.

10.6 Summary

The current Ozonia 40 VLS low-pressure UV system, installed in 2001, is nearing the end of its life, generally estimated at approximately 20 years. The Town may want to be prepared for replacement within the next five years. As elaborated here, options for replacing the existing system exist, although any new system will have higher energy costs due to the most recent requirements for system operation under the current NWRI guidelines.

The comparative cost analysis presented in the memorandum is similar to the results of similar studies conducted by this project team, which indicate that maintaining an existing UV system is less expensive than replacement with a new UV system or other new disinfection technology. Replacement of old UV systems are generally required when one or more of the following items occur:

- The UV supplier stops providing replacement components and WRF staff needs to fabricate materials on-site or search for parts from multiple and less reliable sources than the original manufacturer or certain after-market suppliers.
- The UV equipment does not meet permit requirements due to changes in water quality or changes in flow or failure of components.
- The UV equipment becomes overly burdensome to maintain, which may be a result of the items noted above.

Based on the condition of the existing UV system, and the fact that permit limits are still being met, it is recommended that the existing system be maintained for at least the next 5 years. The Town could then consider replacement if maintenance costs begin to increase or parts availability wanes. At this point in time, most cost advantageous replacement options are the Ozonia system (much like the existing system) or the Calgon Carbon $C^3$500D. When the time comes, a detailed engineering evaluation is needed to determine whether the life cycle cost savings with the Calgon type system (which are based primarily on lower O&M costs) can overcome the additional capital cost of infrastructure modifications. Special

attention should be given to flow control and maintaining the gravity flow regime through the UV channel.

Additionally, as a consideration for designing any replacement UV system, the Town may want to consider gaining the additional ability to re-treat (disinfect) effluent stored in Ponds 5, 7, and S2. As mentioned in **Section 4**, the pending NCRWQCB Pathogen TMDL could potentially require re-treatment should the effluent storage ponds exceed future Pathogen TMDL limits. Future considerations for re-treatment could also address water quality concerns related to other constituents as needed.

11. Sludge and Solids Management

The Town would like to review more sustainable alternatives to reduce the truck traffic associated with the contract dredging, dewatering and disposal operation. In addition, the Town would like to consider alternatives that would be consistent with a plant-of-the-future in terms of solids handling, disposal, and potential reuse.

11.1 Introduction

The WRF currently uses sludge ponds for storage and stabilization of the waste activated sludge (WAS) and sludge generated by the advanced wastewater treatment (AWT) clarifiers. The Town then contracts on an annual basis with an outside firm that specializes in sludge dredging, dewatering, and disposal. The dewatered sludge is then hauled and disposed of off-site for use as fertilizer on cropland or alternative daily cover in regional landfills.

The Town has stated that the operation is costly and results in transporting approximately 140 truckloads per year for offsite disposal, at approximately 26 wet tons per load. The trucking of this dewatered sludge from 40 to 130 miles away for disposal also contributes to the Town's carbon footprint with respect to greenhouse gas emissions. The Town wishes to consider other economical and operationally viable options for sludge and solids management.

This section reviews the existing method of sludge and solids management, including O&M costs. It then evaluates alternatives for solids handling and treatment that may reduce the volume of the material to be disposed off-site and the associated truck trips required for off-site disposal as well as diversify disposal options for the biosolids.

11.2 Existing Sludge and Solids Handling and Treatment

The Town's WRF stores and treats sludge and solids in dedicated on-site sludge ponds. The sludge ponds receive all the WAS generated in the secondary clarifiers as well as sludge generated by the AWT clarifiers. The sludge settles, and the organic matter is partially anaerobically digested in the ponds.

There are currently two sludge ponds in use, Ponds S1, and S3. Pond S2, originally designed as a third sludge pond, has been used only for effluent storage since 2002. Supernatant from the ponds overflows weirs in outlet structures in each of the two sludge ponds and enters the influent sewer main whenever the amount of flow exceeds the amount of pond evaporation. Surface aeration is provided by aspirator-type aerators to control odors via an aerated water cap.

Sludge Storage pond design parameters are outlined in **Table 11.1.**

Table 11.1: Sludge Storage Pond Design Parameters

Parameters	Pond S1	Pond S2 [2]	Pond S3
Pond [1]			
Dimensions (ft)	320 x 230	390 x 215	355 x 220
Depth at overflow structure (ft)	11.5	12.0	12.25
Freeboard (ft)	2	2	2
Volume at Overflow (MG)	5.3	6.8	6.8
Number of Aerators (original design)	6	6	6
Decant Tank			
Number	1		
Capacity (gal)	90,000		
Side Water Depth and Diameter (ft)	15 feet deep x 34 feet diameter		

[1] Information from O&M Manual
[2] Pond is now used primarily for effluent storage

The sludge ponds and decant tank are located on-site, to the south-east of the main facility as shown in **Figure 11.1**. A plant road for dewatered sludge hauling runs from the ponds to the WRF exit onto Windsor River Road at the south gate.



Figure 11.1: WRF Site Aerial View

Sludge solids are removed from the bottom of the ponds once or twice a year using a dredge. The dredged solids are pumped into the storage and decant tank (see **Figure 11.1**) for storage and equalization prior to

dewatering. The Town contracts with a sludge dewatering and disposal firm for the contract operation of a dredge and mobile dewatering belt press. Hauling and off-site disposal of dewatered sludge is also handled under the contract.

During the 2013 to 2016 dredging, dewatering, and disposal operations, the contract operator removed approximately 538 dry tons per year of dewatered sludge solids (average of 16 percent total solids by weight) from the ponds. This resulted in an average of 141 truck trips per year. The number of daily truck trips varied but ranges from 1 to 6 trips per day, averaging about 4 trucks per day during contract hauling operation. The biosolids were land applied in Sonoma County, used as compost, or used as alternative daily cover in landfills.

A summary of the 2013 to 2016 contract dewatering and disposal operations and the quantities removed are provided in **Table 11.2** and detailed in the Annual Solids Disposal reports, included in **Appendix K**. Photos of the typical dewatering setup and sludge hauling truck is shown on **Figure 11.2** and **Figure 11.3**.

Table 11.2: Solids Disposal Report Summary (2013 to 2016)

Parameter [1]	Value
Average Loads per Day	1 to 6
Average Truck Loads per Year	141
Average Annual Sludge Removed (dry tons)	538
Average Volatile Solids Reduction	50%

[1] *Based on 2013 to 2016 Annual Report of Solids Disposal*



Figure 11.2: Contract Dewatering Operation (2016) (Photo courtesy of BRCE)



Figure 11.3: Contract Hauling Operation (2016) (Photo courtesy of BRCE)

11.3 Design Criteria

This section outlines the sludge handling and treatment parameters used to develop criteria for solids handling and treatment alternatives. The two main parameters, sludge flow and solids content, are discussed below

11.3.1 Sludge Flow

The current estimated sludge (combined WAS and AWT sludge) flow to the sludge ponds average 56,000 gallons per day (gpd) (approx. 40 gpm) per aeration basin (the AWT Clarifiers produce a minor amount of sludge compared to the Secondary Clarifiers). This is based on modeling conducted by the team and was compared against the WRF's solids hauling records for 2014 to 2016.

11.3.2 Solids Production

The current reported sludge solids production at the WRF is approximately 2,800 pounds per day (PPD) on a dry solids basis. The concentration of solids in the sludge is approximately 3,200 to 3,500 mg/L total solids. For verification, the estimated solids production in pounds per day compared similarly to the reported weight of solids that were dewatered and disposed of by the contract dewatering operation during the last 5 years according to the Annual Reports (see **Appendix K**).

11.4 Existing System O&M Costs

In May 2015 the Town issued a Request for Proposals for annual biosolids removal services, including dredging, dewatering, transporting and reuse or disposal of the sludge solids. The proposal was based on an estimated quantity of 550 dry tons of solids. A single proposal for $558,950 was received, which is

approximately $1,020 per dry ton of solids. During the sludge removal operation, an engineering consultant provides oversight and management at a cost of approximately $30,000. Therefore, the overall cost is approximately $1,075 per dry ton, which is used as the basis for comparison with alternative treatment and disposal options. This estimate is consistent with the Town's most current contract for sludge and solids operations.

In addition, operation and maintenance staff spend time each week on the sludge pond operation and additional time during the contract dewatering and disposal operation. Estimates from the Town for operation and maintenance time include 4 to 8 hours per week on the pond operation and 8 to 16 hours per week during the time periods of the dredging, dewatering, and hauling operation.

11.5 Sludge and Solids Handling and Treatment Alternatives

The Town is interested in sludge handling and treatment alternatives to the current operation. The main goals are to reduce the number of truck trips associated with hauling operations and determine if there is a more economical and energy efficient approach to long-term solids handling and treatment. The Town would also like to diversify disposal options given the regulatory climate surround biosolids as described in **Section 4**.

Four alternatives were considered for this evaluation and include:

- Sludge drying beds,
- Mechanical thickening of WAS and AWT sludge,
- Mechanical dewatering equipment, and
- Anaerobic digestion with the mechanical thickening and dewatering.

These alternatives are described below.

11.5.1 Alternative 1 - Sludge Drying Beds

Sludge drying beds are typically used for drying digested sludge. This alternative would increase the solids content of the sludge by both thermal drying in the summer and year-round gravity drainage. Sludge drying beds are most effective when used in arid climates where the sludge can be dried to a concentration that is greater than 20 percent. Sludge drying beds were used to dewater over half the sludge in the US in the early 1970s and are still used today. A schematic of the sludge drying bed alternative is shown on **Figure 11.4**. The components of this system include:

- constructed shallow drying beds, with optional sunshade/weather structure,
- equipment for solids removal and truck load such as a loader,
- existing ponds would remain for sludge stabilization.

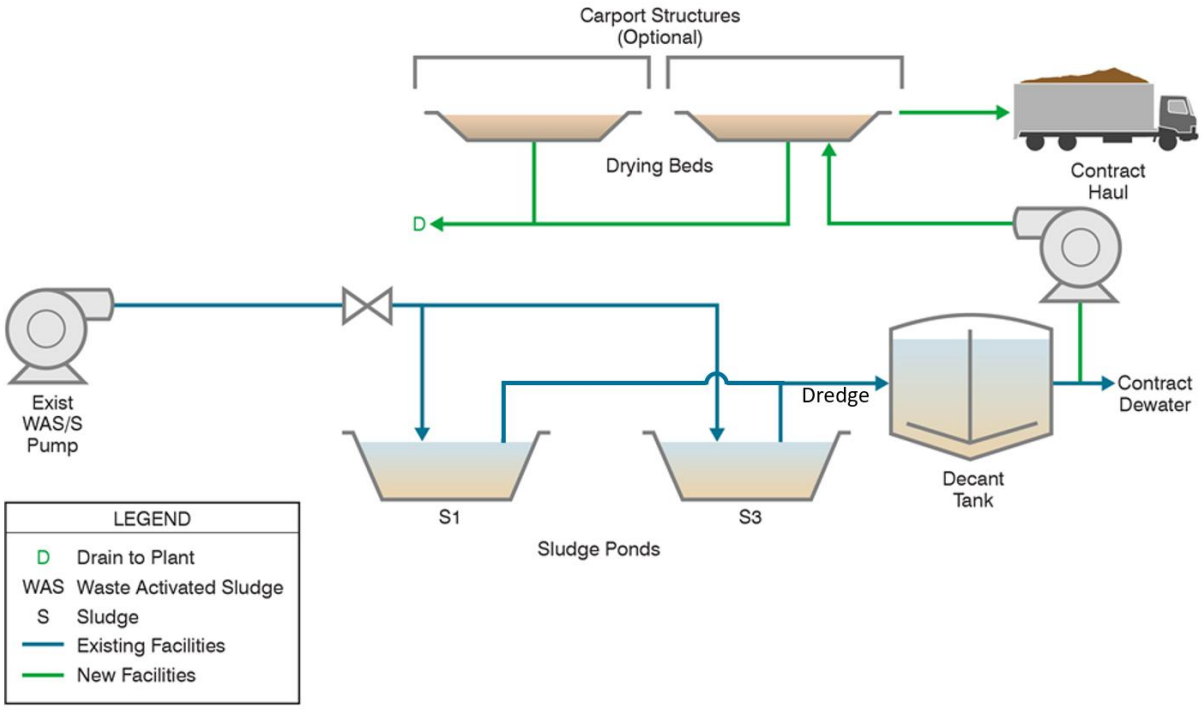


Figure 11.4: Alternative 1 - Sludge Drying Bed Schematic

The beds are typically constructed with an underdrain system overlaid by sand media or synthetic media (wedge-wire), to allow gravity drainage to occur. Loading criteria varies depending on the type of sludge, with generally higher rates for primary or primary sludge blends and lower for WAS and WAS sludge blends. For this evaluation, a general loading rate of 15 pounds of solids per year per square foot was used to develop a preliminary size. At an average sludge production of 538 dry tons per year (2013 to 2016), the size required for the WRF would approach the order of 1.5 to 2 acres. The beds would be approximately 1.5 to 2 feet deep with wet solids. Sludge drying beds are sometimes covered in wetter climates such as Northern California. **Figure 11.5** shows a covered sludge drying bed from Arcata CA. In Arcata, digested primary sludge is dewatered in the drying bed.

The sludge removal operation would be completed manually using small excavation equipment and manual labor. Typically, the sludge is placed on the bed for 3 to 6 months before removal, and essentially scraped off the bed surface. Use of drying beds would likely result in less disposal truck loads because the sludge removed from the beds would be dryer, and therefore require less trucking of entrained water. Going from the typical 16 percent solids experienced in the last 4 years of contract dewatering to 20 percent solids would reduce the number of disposal truckloads by 25 percent if the entire annual sludge volume of the WRF could be applied to sludge drying beds.



Figure 11.5: Covered Sludge Drying Bed (Arcata CA)

To process all of the WRF's annual sludge production, this alternative would require that the Town set aside 1.5 to 2 acres of property (or fill one of the existing sludge ponds for construction of the drying beds). Two ponds could remain as a backup or for other uses by the Town. The main concern with this alternative is the stabilized sludge may still create substantial odors while the sludge dries and additional stabilization takes place. Currently the slow anaerobic stabilization process takes place in the sludge ponds with an oxygenated water cap to limit odors. In a sludge drying bed, the potential for odors would be increased. It is difficult to contain and treat the odors from these operations. Such odors would be unwelcomed near the residential neighborhoods in the vicinity of the sludge ponds. The Town may consider off site sludge drying on Town-owned property; however, accessible land is currently within floodplain and is needed for Town irrigation/disposal. This alternative is not considered viable for the Town due to odor and land availability concerns.

11.5.2 Alternative 2 - Mechanical Thickening

The mechanical thickening alternative could be considered to reduce the water content of the sludge before it enters the ponds. This might extend the required period between dewatering operation by increasing the amount of solids that can be stored in each pond. It will not reduce the overall quantity of sludge that needs to be dewatered and hauled off-site. Different type of thickening equipment could be used including gravity belt thickeners (GBT), or rotary drum thickeners (RDT). A GBT was used in the analysis, although an RDT could also be considered. A GBT is typically used as a first step during the Town's current contract dewatering operations.

A simple schematic of the mechanical thickening process is shown on **Figure 11.6.** The components of this system include:

- mechanical thickening equipment such as GBT or RDT,
- polymer feed system,
- thickened sludge pump,
- piping of WAS and AWT sludge to the thickener,
- sunshade or other enclosure,
- and odor control, if enclosed.

Sludge from the WAS or AWT sludge pumps would be sent to a GBT unit. Polymer might be required for thickening, although it is not always required. The thickened sludge would then be pumped to the sludge ponds for volatile solids reduction. The thickened sludge could range from 3 to 5 percent total solids, decreasing the volume of sludge flow into the sludge ponds by a factor of 5 (assuming 3 percent solids) or more. This would increase the effective sludge pond capacity by the same factor of 5, and thus reduce the frequency that pond solids would need to be removed.

As the Town grows, additional biosolids storage will be needed. The addition of mechanical thickening would reduce the volume of sludge flow and prolong the capacity of the existing sludge ponds (S1 and S3). This would defer the need to convert Pond S2 back into a sludge pond.

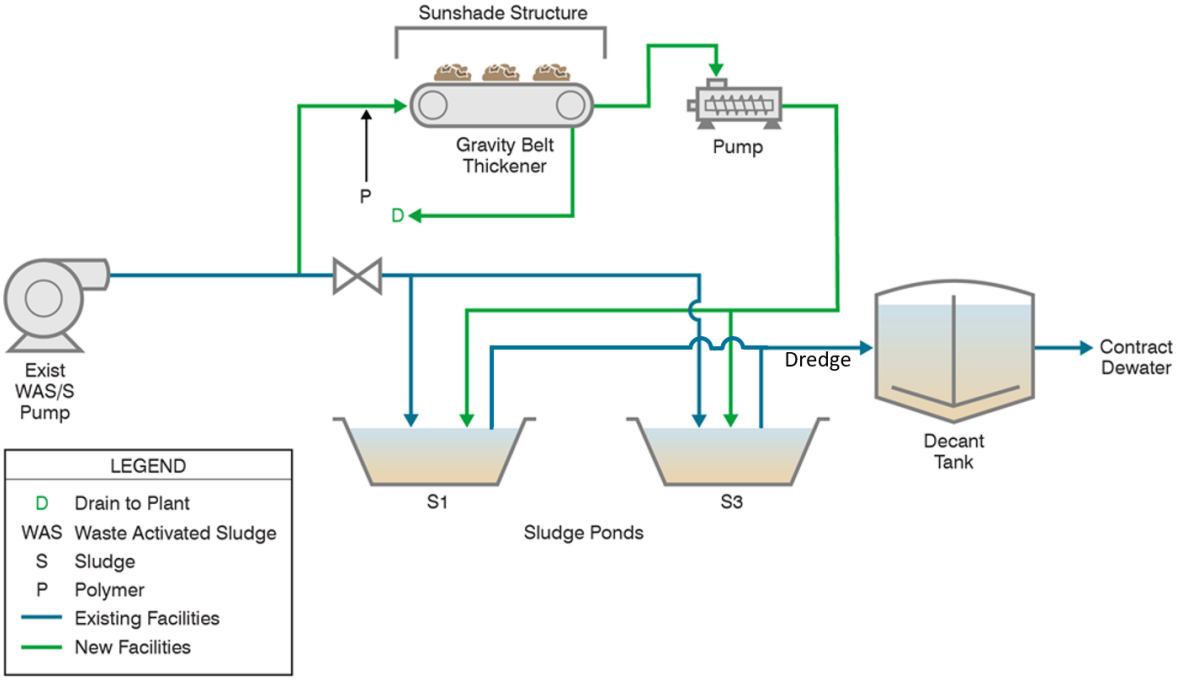


Figure 11.6: Alternative 2 - Mechanical Thickening Schematic

If the sludge ponds biosolids are currently removed and dewatered once a year, then the pond biosolids would need to be removed and dewatered approximately every other year. The alternative would not reduce the overall number of trucks, but it could increase the time between the contract dewatering operations.

This alternative is feasible for the Town and is considered further in the cost comparison section below. It is important to note that this alternative would add approximately $14,000 in annual chemical costs. As

an additional unit process, additional operator time would be required to operate and maintain the equipment. Additional breakdowns of power, chemical, operations and maintenance costs are provided in **Appendix L**.

The ponds should operate in a similar manner to the current operation with an aerated water cap to minimize odor generation. The mechanical thickening operation may also generate odors unless the unit is enclosed, and odor control is provided. An enclosure and odor control could add up to 50 percent additional construction cost of the thickening facilities. A photo of a skid mounted GBT with a sunshade is shown in **Figure 11.7**. An enclosure with odor control should be considered for this alternative if the GBT is planned to be located near the perimeter of the WRF where residential neighborhoods are located.



Figure 11.7: Gravity Belt Thickener Installation (Skid mounted units Fresno – Sized for a 48 MGD Facility)

11.5.3 Alternative 3 - Permanent Mechanical Dewatering Equipment

The third alternative considered for the Town's sludge solids is to add permanent dewatering equipment such as a belt filter press (BFP) or a screw press. This is similar to the technology that is currently being used by the Town's dewatering contractor, but the Town would be able to perform dewatering in-house. The dewatering equipment could be used to increase the solids content of the solids dredged from the ponds. The dewatering processes would result in approximately 15 to 18 percent solids, whereas mechanical thickening (as discussed in Alternative 2) would produce approximately 3 to 5 percent solids.

The Town could alternate the sludge pond that is used, and provide the same digestion time as the current operation. The Town would dewater and dredge the pond after the digestion has occurred. The Town may be able to achieve greater dewatering than at present (up to 18 percent), which would reduce truck traffic by approximately 10 percent.

With this option, the Town would also be able to bypass the ponds if needed and dewater the WAS for off-site processing at another facility such as another treatment plant with excess digester capacity, or a Lystek facility such as that in Fairfield, California. This alternative has an additional benefit in terms of phosphorus removal. Currently the WRF recycles sludge pond supernatant to the head of the plant. Because sludge undergoes digestion in the ponds, phosphorus is released, leaving high phosphorus levels in the sludge pond supernatant. By bypassing the ponds, the recycle stream should have lower phosphorus levels because little to no digestion would occur, and little to no phosphorus would be released. Thus, bypassing the ponds has the added benefit of reducing phosphorus loading to the WRF.

A simple schematic of the mechanical thickening process is shown on **Figure 11.8**.
The components of this system include:

- mechanical dewatering equipment such as belt filter press (BFP), or screw press,
- pond sludge feed sludge pump,
- dredge to remove sludge from ponds
- piping to connect to existing system,
- polymer feed system,
- sunshade or other enclosure,
- odor control,
- pond bypass, shown in green, is possible with a stabilization step such as lime stabilization,
- truck load out system such as an inclined conveyor.

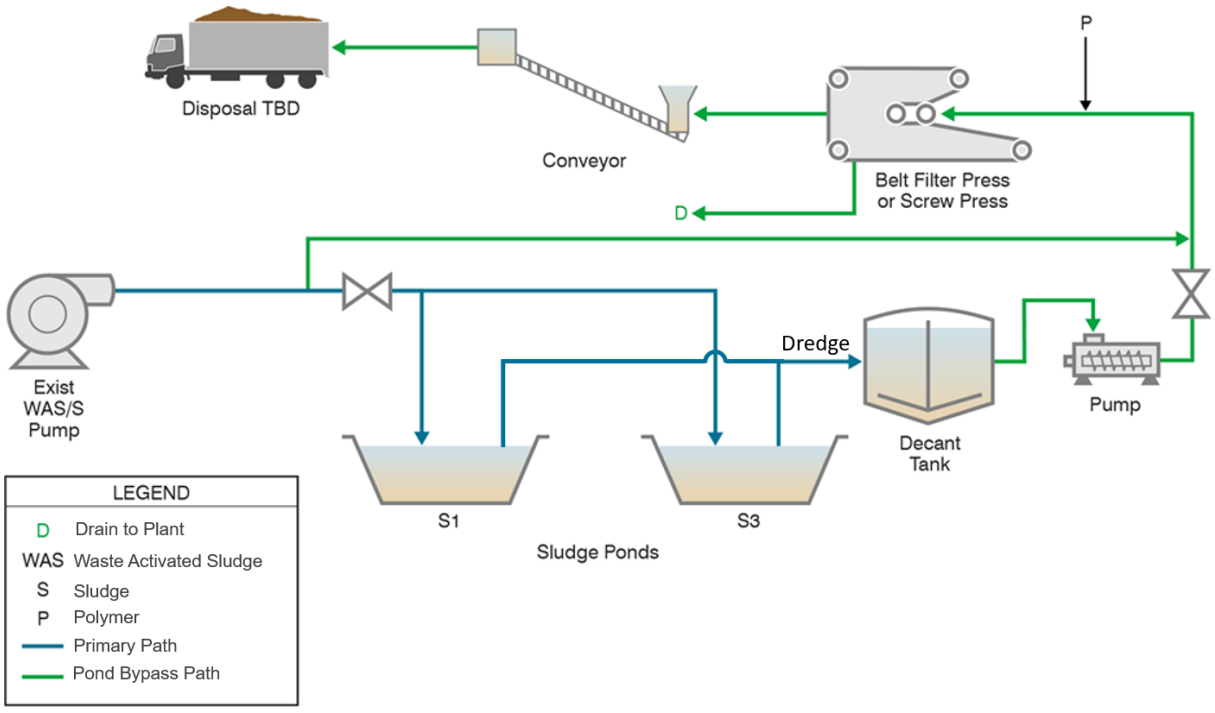


Figure 11.8: Alternative 3 - Mechanical Dewatering Schematic

This alternative is feasible for the Town and is considered further in the comparison section below. In this configuration, the Town would not need to contract with dewatering contractors if the ponds are used for sludge stabilization. If the Town were to send the sludge off-site for stabilization, the Town still would not need to contract with dredging or dewatering contractors; however, polymer costs would increase by approximately $143,000 per year. Because this is an additional treatment process, additional labor costs for the new equipment to be operated and maintained is included in the cost analysis. Other operations and maintenance costs for this alternative are provided in **Appendix L**. Since the total solids might be reduced only slightly, the total annual number of trucks for contract disposal would remain effectively the same as the current operation. If dewatering can be optimized, there is potential to reduce the total truck loads, although an increase in solids percentage of more than 3 to 5 percent may not be possible without the use of a centrifuge for dewatering. With the use of a BFP solids would be expected to peak at approximately 18 percent.

The sludge ponds would operate in a similar manner to the current operation with an aerated water cap to minimize odor generation. The mechanical dewatering operation may also generate odors unless the unit is enclosed, and odor control is provided. The enclosure and odor control could add up to 50 percent additional cost for the thickening facilities.

A photo of a BFP is shown on **Figure 11.9**. Like the GBT, a BFP can be skid mounted with only a sunshade structure or fully enclosed in a building. An enclosure can be used to contain any odors of the dewatering operation and should be considered for this alternative since the facility is located near a residential area. The enclosure is a more expensive option than a sunshade.

As previously mentioned, an alternative mode of operation is also shown on the dewatering alternative schematic (green), which shows sludge flows going directly to dewatering, bypassing the ponds. This would require additional stabilization of the raw sludge to reduce pathogens. Off-site processing at another facility such as another treatment plant with excess digester capacity, or a Lystek facility such as that in Fairfield, California could be considered. Reportedly, several Biolac-type secondary treatment systems have started to dewater sludge as it is pumped from the secondary process without digestion. This disposal option is not recommended at this time; however, it might be considered in future work after additional research.



Figure 11.9: Skid Mounted Belt Filter Press Unit

11.5.4 Alternative 4 - Anaerobic Digestion

Anaerobic digestion is the last alternative considered for the Town's solids treatment and handling options. This would be considered as an option to replace the pond system in combination with (and after both) mechanical thickening (Alternative 2) and dewatering (Alternative 3). Anaerobic digesters provide an encapsulated anaerobic environment in which microorganisms break down biosolids. Anaerobic digesters produce heat, carbon dioxide, and methane gas. Because air does not need to be applied to further the chemical breakdown of biosolids, anaerobic digesters are an efficient mechanism for solids digestion. They also provide an efficient mechanism for solids processing. It is estimated that anaerobic digestion at the WRF could result in 10 to 12 percent fewer trucks required for solids disposal. In addition, use of a digester would eliminate the need for dredging, as solids are removed from the bottom of the digester.

Anaerobic digestion could completely replace the regular use of the sludge ponds, rather than optimize the current stabilization treatment system (as in Alternatives 2 and 3). A typical digester system is sized based on hydraulic residence time (HRT). Given the WRF's feed sludge flow rate and adding

approximately 25 percent additional volume for grit and scum accumulation, the digester size would be approximately 1.5 MG to 2.25 MG, based on 20-day and 30-day HRTs, respectively. Digester cost and preliminary sizing for the WRF is outlined in **Table 11.3**.

Table 11.3: Preliminary Anaerobic Digester Sizing and Cost

Parameter	20 Day HRT	30 Day HRT
Digester Size [1] (ft^3)	160,000	240,000
Digester Size [1] (MG)	1.5	2.25
Preliminary Construction Cost [2] (in Millions)	$3.6	$5.5
Overall Projected Volatile Solids Reduction (%)	45 to 60%	

[1] *Based on a single digester with listed HRT, average sludge production and a 25% allowance for grit and scum accumulation*
[2] *Digester and ancillary equipment (e.g. boiler and exchanger) only; pre-digestion dewatering and thickening equipment would be added to digester cost.*

A simple schematic of the anaerobic sludge digestion process is shown on **Figure 11.10**. A photograph of a typical digester is shown on **Figure 11.11**. The components of this system include:

- mechanical thickening and dewatering equipment, similar to alternatives 2 and 3
- associated odor control equipment
- digester and mixing pumps
- boiler and digester heating equipment, including pumps and heat exchanger,
- gas storage tank (optional)
- digester gas flare
- monitoring and control system
- polymer feed system,
- building for boiler and electrical equipment,
- sunshade or other enclosure for dewatering and thickening equipment,
- truck load out system such as an inclined conveyor.

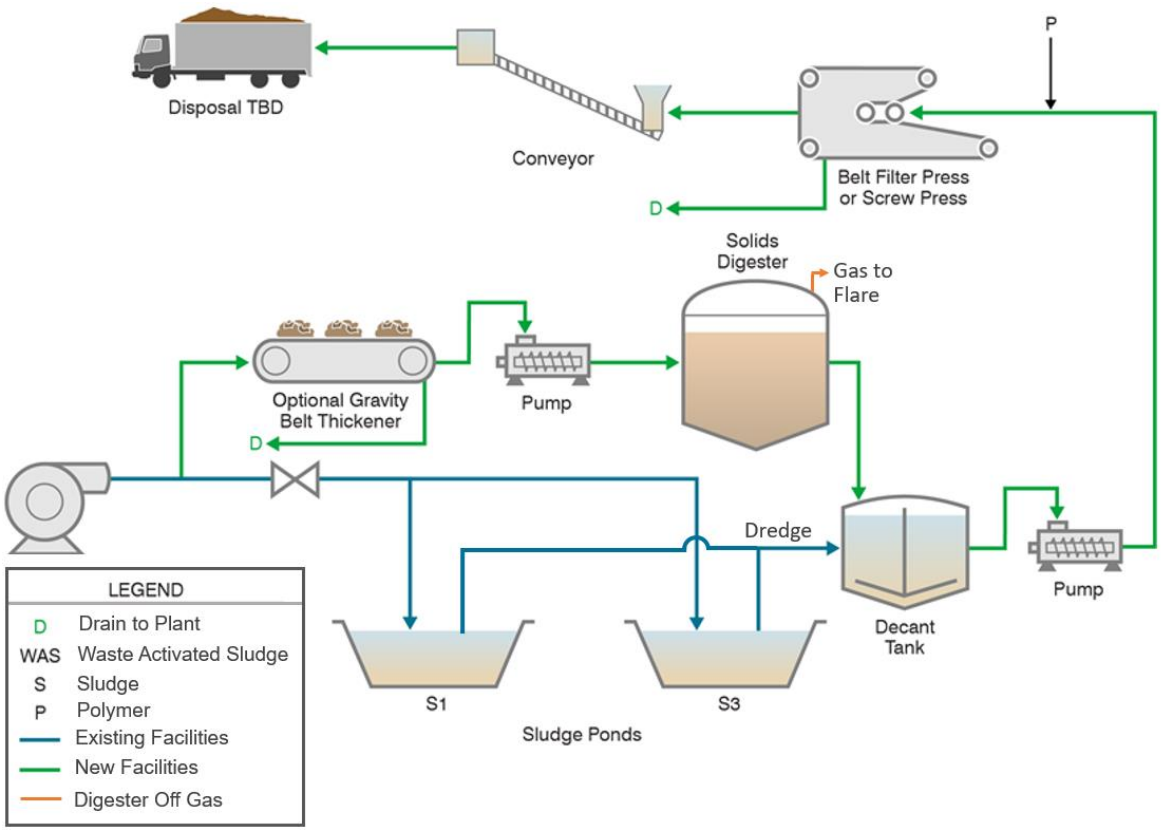


Figure 11.10: Alternative 4 - Sludge Digestion Schematic

One option is to phase the system components in over a period of time starting with a permanent thickening facility (i.e. a GBT), then adding dewatering facilities (i.e. a BFP or screw press), and finally adding the digesters. The Town could then evaluate whether to phase out the ponds. One or more ponds should be kept to provide redundancy for the digester system. As facilities are added the sludge ponds could be removed from service, cleaned and used for additional effluent storage. As shown in **Table 11.1**, keeping Pond S2 as effluent storage results in 6.8 MG for additional effluent storage. In the future scenario where digesters are construction, an additional 5.3 MG could be converted to effluent storage for a total of 12.1 MG (assuming one large sludge pond remains as a backup for the digester). There are many benefits to additional storage for the Town, including reducing discharge and phosphorus loading to Mark West Creek and providing additional capacity to re-rate the WRF. These benefits are discussed in **Section 8**.

One disadvantage of anaerobic digestion is that it would generate a more concentrated sidestream. Lysis of biomass in digestion followed by dewatering will produce a recycle stream rich in ammonia and orthophosphate. Recycle streams from dewatering of anaerobically digested sludge can have ammonia levels in the 1000's mg-N/L while orthophosphate can be as over 200 mg-P/L. These are much higher concentrations than what is currently observed in the pond supernatant. This sidestream may require further treatment before it is returned to the WRF. This may provide an opportunity to harvest nitrogen, phosphorus, and magnesium using a resource recovery process, such as Ostara. As the wastewater

industry looks to the future, resource recovery which maximizes recycling of all resources is a key consideration for plants of the future.

Using a digester as the primary solids sludge stabilization process would potentially reduce the odors compared to the current system of sludge pond dredging and dewatering. However, the mechanical thickening and dewatering operations would generate odors and would require enclosures and odor control, especially since the WRF is located in a residential area. The enclosures and odor controls will add up to 50 percent additional cost of the thickening and dewatering facilities.



Figure 11.11: Anaerobic Digester and Sludge Holding Tank in Petaluma (approx. 0.55 MG)

It should be noted that anaerobic digesters can reduce effective greenhouse gas emissions by capturing methane that would otherwise be emitted from the surface of sludge ponds. Quantification of this benefit is outside the scope of this study and should be considered during a more detailed study of solids handling options. The captured methane (natural gas) is either flared or used at cogeneration facilities, where it is combusted to generate heat and energy.

The amount of methane produced at smaller treatment plants (like the Town's WRF) is usually not sufficient to operate such cogeneration facilities. The Combined Heat and Power Partnership analyzed a number of wastewater treatment facilities that utilize anaerobic digestion and determined that facilities operating at less than 5 MGD annual average flow could not produce enough biogas from anaerobic

digestion of biosolids to make combined heat and power technically and economically feasible. Based on the WRF's average annual flowrate of 2.26 MGD and the fact that the WRF may only have waste activated sludge being sent to the digesters, the quantity of methane would likely not be sufficient to generate all of the required digester heating especially in the colder months. Supplemental natural gas may need to be purchased from a utility company to produce additional heat. A more detailed analysis would be required during the preliminary design process. Additionally, if only secondary sludge is treated (i.e. using only waste activated sludge from the secondary treatment processes such as occurs at the WRF) the methane production is lower than if primary and secondary sludge is treated. This would further reduce the likelihood that cogeneration would be economically viable. Equipment is also needed to scrub contaminants out of the digester gas before it can be used for heat or power. A more detailed economic analysis is required to determine if this investments in gas scrubbing would be feasible for the Town.

As a result, the Town may decide to flare anaerobic digestor gas if a digester is installed. It should be noted that flared gas would be released as carbon dioxide, which has a lower greenhouse gas emission factor, and thus a reduced impact on the environment compared to methane. Further analysis would be needed to quantify the greenhouse gas benefits of anaerobic digestion compared to sludge ponds.

11.5.5 Solids Handling and Treatment Alternative Comparison

The solids handling alternatives outlined above were reviewed based on cost as well as non-economic factors. A summary of the review is outlined below.

11.5.5.1 Cost Comparison

The estimated equipment costs and capital costs for the sludge treatment and handling alternatives are outlined in **Table 11.4.** These costs should be used as budgetary and comparative costs and are not detailed estimates.

Alternative 1 (Sludge Drying Beds) was eliminated from the evaluation due to infeasibility. Alternatives 2 and 3 (Mechanical Thickening and Mechanical Dewatering) only supplement the existing solids handling process and may make the existing contract dewatering and disposal more efficient. Alternative 4 (Anaerobic Digestion) will effectively replace the sludge ponds and contract dewatering operations. It should be noted that the Alternative 4 cost include the costs of mechanical thickening and dewatering prior to the anaerobic digestion step. In all cases, off-site disposal will still be required. As noted previously, the anaerobic digestion could be phased in over time.

The operation and maintenance (O&M) costs of the sludge/solids treatment alternatives are difficult to estimate, so the costs listed only estimates for comparison between alternatives and to current operation. The costs were developed based on information for the City of Petaluma operation which includes GBT thickening, anaerobic digestion, screw press dewatering, and off-site disposal. A factor of 3 was used to estimate the construction costs from the equipment costs. This is a typical factor for conceptual estimation of construction costs, based on team experience.

The main factor in O&M costs is the disposal cost which is site-specific based on trucking costs and distance to disposal or reuse sites. It was assumed that 550 dry tons of sludge solids would be produced

for all alternatives (note that wet tons differ for each alternative based on percent solids content). As previously mentioned, anaerobic digestion would likely reduce trucking by approximately 10 to 12 percent.

Table 11.4: Sludge and Solids Treatment Alternatives Cost Comparison

Alternative	Equipment Cost	Construction Cost	Construction Cost with Enclosure and Odor Control	Annual Disposal Costs [3]	Annual O&M Cost [4]	30 Year NPV [5] (in Millions)
2. Mechanical Thickening [1]	$350,000	$1,050,000	$1,575,000	$588,950	$72,700	$10.7
3. Mechanical Dewatering [1]	$650,000	$1,950,000	$2,925,000	$588,950	$232,700	$14.2
4. Anaerobic Digestion [2]	$1,500,000	$5,500,000	$7,000,000	$206,250	$425,300	$15.7
Existing						
Sludge Ponds	Not Applicable	Not Applicable	Not Applicable	$588,950 [6]	Not Applicable	$8.1

[1] *Equipment costs based on recent project quotes. Construction costs based on a factor of 3.*
[2] *Anaerobic digester costs based on cost curve data for 20 days HRT, includes mechanical thickening and dewatering costs from Alternatives 2 and 3.*
[3] *Disposal Costs include costs for existing dredging and dewatering (if applicable), and disposal for 550 dry tons of sludge solids.*
[4] *O&M Cost Estimates include costs for power, chemicals, and operation and maintenance of each alternative.*
[6] *30-year net present value assumes an interest rate of 6 percent. The estimate for capital costs assumes construction cost with enclosure and odor control.*
[6] *Based on the 2015 bid for solids removal and disposal. Does not include Town operation or maintenance costs; includes only third-party oversight.*

Based on **Table 11.4**, the most economical alternative is to keep the existing sludge ponds. However, there are non-economic factors that should be considered. These are discussed in the triple bottom line plus analysis in the following section.

11.5.5.2 Triple-Bottom-Line-Plus Analysis

A triple-bottom-line-plus analysis refers to an analysis that considers alternatives in terms of their (1) economic, (2) social, and (3) environmental criteria plus (4) technical criteria. A triple-bottom-line-plus ranking of the alternatives is outlined on **Table 11.5** and summarized below.

Alternative 1: Sludge Drying Beds was eliminated from the analysis based on concerns with land availability and odors.

Alternative 2: Mechanical Thickening will increase the effective storage volume of the existing ponds, which may allow for a contract dewatering and hauling operations every other year. The reduced traffic and neighbor impacts make this alternative more favorable in the social criteria than addition of permanent dewatering (Alternative 3). The main disadvantage could be increased odors, although the economic analysis assumes that an enclosure and odor control is provided.

Alternative 3: The advantage of Mechanical Dewatering would be to reduce the number of truck trips (by approximately 10 percent) compared to the current contract dewatering operation. The main disadvantages are increased operational expenses and potentially increased odors; however, the cost analysis assumes that an enclosure and odor control are provided. The estimated construction cost for this alternative is the second lowest cost of the alternatives considered.

Alternative 4: The social and environmental advantage of Anaerobic Digestion would be to optimize the volatile solids reduction (VSR), reducing the actual mass of the dewatered sludge that is sent off-site. Given the current VSR in the ponds, the digesters may only slightly improve the VSR which may decrease the solids volume and thus reduce truck trips by an estimated 10 to 12 percent. This would result in lower emissions due to less traffic. A second advantage is that the digestion process is always enclosed, so potential for odors would be lower compared to the existing sludge pond system. In addition, digestion system would produce methane gas which could be used to heat the digesters and may provide a reduction in greenhouse gas emissions. If this alternative is evaluated further, a greenhouse gas analysis should be performed. The main disadvantage of digestion would be the high capital costs. Because the WRF is less than 5 MGD average annual flow and would only be treating waste activated sludge, some of the heating could require supplemental natural gas (see **Section 11.5.4**). A more detailed analysis of the heating requirements will be required as part of the preliminary design process. Therefore, it is ranked the worst on the basis of capital and O&M costs. Another disadvantage would be the resulting supernatant sidestream that would be returned to the WRF for treatment. The sidestream might be more nutrient concentrated and have a higher strength than the current pond supernatant. In plants that utilize anaerobic digestion, recycle streams can represent 40 percent of the plant influent TP load. Therefore, it is ranked 3 in technical criteria. However, the sidestream could be turned into an advantage if the Town chose to recover the nutrients, at additional cost.

The existing sludge ponds present the lowest cost; however, they do not prepare the WRF for potential future regulatory decisions. Regulations determine how biosolids can be disposed. Increased concern over environmental impacts have led to more stringent regulations. For example, the sludge ponds generate Class B biosolids. Six counties in California have already banned Class B land application. Changes in regulations affecting biosolids management are expected in the future and make a flexible management program essential. Additional discussion of anticipated regulations is provided in **Section 4**. Thus, the existing sludge management regime should be re-evaluated.

Table 11.5: Sludge and Solids Treatment Alternatives – Triple Bottom Line Ranking [1]

Criteria	Economic Criteria	Social Criteria	Environmental Criteria	Technical Criteria
Alternative	Capital and O&M Cost	H&S, neighbor impacts: odors, noise and traffic	CEQA, greenhouse gases, NPDES, disposal	Proven technology, ease of operation, flexibility, constructability
2. Mechanical thickening	1	2	3	1
3. Mechanical dewatering	2	3	2	2
4. Anaerobic digestion with thickening and dewatering	3	1	1	2
Existing Sludge Ponds	1	3	3	2

[1] *Scale 1 to 3. Ranking of 1 has the least impact or the lowest cost.*

11.6 Summary

The current sludge treatment and handling system for the WRF is based on a non-mechanical, facultative sludge storage pond system. Once or twice a year the Town contracts for dredging, dewatering and disposal of the digested sludge in the pond system. The Town has noted that the operation can be disruptive and results in a significant number of truck trips (an average of 140 trucks per year) during periods of solids removal from the ponds. In addition, recent dredging and dewatering operations have resulted in temporary odor complaints.

Four alternatives were identified and reviewed to the existing system and three were determined to be viable for the WRF. The alternatives all have a potential to slightly reduce the volume of material to be disposed of off-site, although the current pond system is very efficient at volatile solids reduction. Each alternative can help to reduce the off-site odors but would require additional capital and operations and maintenance costs.

It should be noted that the alternatives all have additional construction and operations and maintenance costs compared to the existing sludge ponds with contract dredging, dewatering and disposal system. Based on the analysis, the existing sludge ponds are the most economical option. The Town should continue using the ponds until a regulatory driver requires more advanced treatment of the sludge.

Other disposal options were not reviewed in this memorandum but might include:

- Discussions with the City of Santa Rosa to utilize the City's digesters which could reduce the hauling costs and may provide the City of Santa Rosa with the benefits of additional feed for their cogeneration system.
- Trucking sludge to a regional biosolids disposal facility such as the Lystek facility in Fairfield, CA.

There are a variety of newer technologies (e.g. Lystek and PONDUS) that could provide excellent stabilization and dewatering, as well as produce a valuable by-product. These are beyond the scope of this study. The Town may want to consider a more extensive solids evaluation to address the options as well as explore any more recent technologies that are available at that time.

11.7 Conclusion

Regulations from CalRecycle or other state and local agencies and landfills are likely to put additional restrictions on the disposal of biosolids as fill or alternative daily cover. This was further discussed in **Section 4**. These restrictions are expected within the next ten years. The Town may be required to use alternative disposal sites or updated technologies. The traditional technologies reviewed above help transition from a pond-based system (land intensive) to a more mechanical system while maintaining a similar level of sludge stabilization, volatile solids reduction and disposal.

In order for the Town to dispose of biosolids in the future, it may require a technology that produces a biosolids product that has diverse disposal options. To achieve this, the Town would want to consider anaerobic digestion. Alternative 4: Anaerobic Digestion could be phased in over time, starting with permanent thickening (Alternative 2), then adding dewatering (Alternative 3), and finally adding the

digesters, and phasing out the ponds (although at least one pond should be maintained for redundancy). If this process was phased in over time, it could help address the Town's long-term wastewater treatment sludge handling infrastructure needs as well as help secure additional effluent storage.

12. Capital Improvement Plan

This section documents the cost methodology used for developing capital and O&M costs as well we provides a summary of the short-, intermediate-, and long-term capital improvements at the WRF. This section also provides a phased implementation plan for the needed improvements over the 2040 Capital Improvement Program (CIP) planning horizon. The projects presented herein have been identified in previous sections or through discussions with Town staff.

Major findings and recommendations of the Modernization Study (Study) fall into three general categories that drive all facility planning efforts:

- Future regulations related to potential phosphorus and biosolids restrictions.
- Replacement and rehabilitation of existing facilities due to aging and undersized infrastructure.
- Capacity/Improvement needs to accommodate the service area through the 2040 planning period.

12.1 Cost Methodology

Capital and Operations and Maintenance costs for projects included in the Study were developed in accordance with industry standard practices. The general methodology is summarized below.

Opportunities for optimization at the WRF were identified throughout the Modernization Study. Each optimization opportunity contained various alternatives. Capital and O&M costs were developed to evaluate and compare the various alternatives. Capital and O&M costs were developed for the WRF processes shown in **Table 12.1**.

Table 12.1: WRF Process Optimization Opportunities

Modernization Report Section	WRF Process
5	Existing Facilities
7	Phosphorus Removal
8	Effluent Storage
9	Grit and Fine Screening
10	Ultraviolet Disinfection (UV)
11	Solids Treatment and Handling

Additionally, a visual condition assessment of facility equipment was performed and projects for future replacement and rehabilitation were identified. Replacement and rehabilitation costs were estimated for the aging infrastructure at the WRF.

All projects were assumed to be performed by an outside contractor through a design-bid-build project delivery method. If the Town completes some projects in-house, project costs would be expected to be reduced from those included in this Report. However, the Town can only perform projects in-house if their total budget is less than $60,000, so the number of projects which can be performed in-house is limited.

12.1.1 Operations and Maintenance Cost Development

O&M costs were developed based upon experience with similar projects, WRF staff input, and engineering judgment. These costs include, at a minimum, electrical consumption for major equipment and WRF staff time for required maintenance. Where applicable, consumable, chemical and spare part costs were included. WRF staff wages were based upon similar wages in Sonoma County. Electrical costs were based upon the average power cost for Sonoma County.

12.1.2 Capital Cost Development

The capital cost reflects the planning level construction cost for each alternative. This cost does not include engineering, legal, administrative, construction management, environmental documentation or other costs not associated with the General Contractor's anticipated bid. These non-construction related costs are included in the Capital Improvement Plan project cost estimates.

In general, the capital costs are considered a Class 4 level Conceptual or Budget Estimate per the Association for the Advancement of Cost Engineering (AACE) estimate categorization because the estimates were based on very limited design scope and general facility information. A Class 4 level estimate is expected to have a typical accuracy of -30 percent on the low side, and +50 percent on the high side.

Depending upon the level of engineering design available, capital costs were developed based upon information from equipment manufacturers, project construction installation factors, reference projects, or broken down into general line item components and assigned unit costs. In all cases, costs were supplemented with estimator judgment to ensure they were representative of the scope at the Town's WRF.

Where line item components were used, labor, material and equipment costs were developed for each component to reflect assumed means and methods of construction, crew composition, and productivity. Budgetary quotes for major equipment were obtained from vendors, where possible, otherwise equipment costs were based on Hazen's experience with similar-type projects.

Wage rates were based on current Department of Industrial Relations prevailing wages published for Sonoma County, California, generally set to expire June 30, 2018. Craft Payroll Burdens and Benefits were carried based on Federal and California state requirements. Insurance rates were in accordance with state and craft requirements. A standard 5-day, 40-hour workweek was used; overtime, shift rates and premium time were not included. Crews, equipment and productivity used for work items are based mostly on standards specific to each trade. Some information was supplemented by RS Mean's data and modified where necessary by estimator judgment.

Equipment rates were primarily based on current published rental rates as listed in the EquipmentWatch Blue Book, but this was supplemented by RS Mean's data, the EquipmentWatch Green Book and local rental suppliers.

Electrical and instrumentation and control costs were estimated as a percentage of total project cost. Factors of 15 percent and 10 percent, respectively, were generally used based upon Hazen's experience with similar projects.

Capital costs include the Contractor's overhead and profit, the Contractor's indirect costs, the Contractor's bonding/insurance and contingency as noted in **Table 12.2**.

Table 12.2: Capital Cost Adders

No	Item	Percentage	Notes
1	Contractor Overhead/Profit	10/10	
2	Contractor Indirect Costs	15	Project Management, Safety, Permitting, Site Maintenance, Scheduling, Temporary Systems, General Site Equipment and Supervision, Submittals and Mobilization/Demobilization
3	Bonding/Insurance	3	
4	Contingency	30	Unless otherwise noted in the project cost

12.1.3 Replacement and Rehabilitation Capital Costs – Section 5

The replacement and rehabilitation project capital costs were estimated utilizing various methods depending upon the project. Each project was identified based upon either the condition assessment, or discussion with Town staff.

For smaller process equipment, pumps, blowers, tanks, mixers, etc., which might be replaced in-kind, quotes from similar type equipment on recent projects were used. These costs were escalated to 2018 dollars and adjusted where necessary to reflect the conditions at the WRF.

For projects where a larger system (comprising of multiple pieces of equipment) is to be replaced or added, such as clarifier and filter internal mechanical components a cost for the entire process was used. In the case of the clarifier internal mechanical components, vendor quotes for similar type equipment were used and adjusted to match the design criteria at the WRF. The sand filter replacement costs were estimated based upon the capacity of each unit, as measured in million-gallons per day. These costs were based upon experience with similar type projects at other plants and adjusted to reflect conditions at the WRF.

For projects with a less well-developed scope, general structural repair, or those that would require additional investigation to provide a more detailed cost, an allowance was used based upon the information available at the time of estimate and experience.

The costs were augmented with a construction installation factor to account for labor and contractor mark-ups related to the project. The construction installation factor included the items listed in **Table 12.2** as well as an installation factor of 25 percent, where applicable. The contingency was increased to 50 percent for these projects due to the longer planning horizon and additional uncertainty.

12.1.4 Phosphorous Removal Capital Costs – Section 7

The alternatives related to phosphorus removal in **Section 7** included purchasing phosphorus credits, chemical addition, modification of the existing Biolac® basins and addition of new process units. For modification of existing or addition of new process units, the costs were developed by breaking each alternative down into smaller work components as described in **Section 12.1.2** above.

12.1.5 Effluent Storage Capital Costs – Section 8

The capital cost of new ponds for effluent storage was based on costs for a similar project in Sonoma County. A reference project was used because it was beyond the scope of this project to develop detailed costs for effluent storage.

For the construction of new ponds, the capital cost was estimated based upon a similar study performed by the City of Santa Rosa from 2007 in which various locations and pond capacities were identified and evaluated. The Santa Rosa costs were adjusted for only the items expected to be included in a new storage pond for the Town. These adjusted costs were then escalated to 2018 dollars. The capital cost is inclusive of Contractor overhead and profit, indirect contractor costs and a 50 percent contingency. The contingency was increased for this alternative because of the lack of information regarding potential suitable sites.

12.1.6 Grit and Fine Screening Capital Costs – Section 9

The alternatives related to grit and fine screening in **Section 9** were developed by breaking each alternative down into smaller work components as described in **Section 12.1.2** above.

12.1.7 Ultraviolet Disinfection Capital Costs – Section 10

Capital costs for the UV alternatives were developed utilizing construction installation factors. Construction installation factors were utilized for these alternatives because the engineering design was not sufficiently developed to perform a detailed estimate. History with similar-type projects indicated that a construction installation factor approach provides a reasonable level of accuracy based upon the available information.

Vendor quotes for the major equipment were obtained and an installation factor was applied to determine the estimated capital cost. The installation factor ranged from 1.5 to 3 depending upon the equipment being installed and the nature of the work. The installation factors were developed based upon the Hazen team's experience with similar-scoped projects at representative wastewater treatment plants. **Table 12.3** shows the installation factors used for various equipment alternatives. The construction installation factor is inclusive of Contractor overhead, profit, indirect contractor costs and contingency.

Table 12.3: Ultraviolet Disinfection Construction Installation Factors

Report Section Number	Alternative Description	Construction Installation Factor
10	Ozonia40HO Open-Channel	1.5
10	ETS 45800-30 Closed Vessel	3
10	Calgon Carbon C^{3}500D Open-Channel	2

12.1.8 Solids Treatment and Handling Capital Costs – Section 11

Capital costs for the Solids Treatment and Handling alternatives were developed utilizing construction installation factors. Construction installation factors were utilized for these alternatives because a detailed engineering design was not developed to perform a detailed estimate. History with similar-type projects

indicates that a construction installation factor approach provides a reasonable level of accuracy based upon the available information.

Vendor quotes for the major equipment were obtained and an installation factor was applied to determine the estimated capital cost. The installation factor ranged from 3 to 3.7 depending upon the equipment being installed and the nature of the work. The installation factors were developed based upon the Hazen team's experience with similar-scoped projects at representative wastewater treatment plants. **Table 12.4** shows the installation factors used for various equipment. The construction installation factor is inclusive of Contractor overhead, profit, indirect contractor costs and contingency.

Table 12.4: Solids Treatment and Handling Construction Installation Factors

Report Section Number	Alternative Description	Construction Installation Factor
11	Mechanical thickening	3
11	Mechanical dewatering	3
11	Anaerobic digestion	3.7

12.2 Capital Improvement Plan

12.2.1 Summary of Needs and Opportunities

The WRF is able to treat the existing wastewater flows, meet current effluent discharge limits, and provide recycled water to users. The existing treatment processes perform well and are relatively adaptable for modification, or addition, to meet future regulatory requirements. Therefore, continued investment in the existing liquid and solids treatment processes is appropriate even though there could be a change in the regulatory requirements in the future.

One of the major recommendations of this Study is to rehabilitate and replace existing facilities that are nearing the end of their useful life. The existing liquid treatment processes should be modified to meet capacity needs until there is a regulatory driver for a change in treatment process. The WRF should also look for ways to diversify the disposal of biosolids to become more adaptable to future regulatory changes that may limit the current practice of disposal by use as alternative daily cover. Additionally, increasing and expanding use of recycled water in the service area is a policy directive that will drive the need to provide adequate treatment, storage and, eventually, distribution capacity.

In **Section 7**, alternatives were developed to meet changing regulatory requirements and to comply with more restrictive effluent phosphorus discharge limits. In **Section 11**, alternatives were developed for solids handling facilities in response to regulatory requirements and for complying with more restrictive land disposal limits anticipated in the near term future.

The major projects identified throughout this Study include the following:

- Future Regulatory Requirements
    - Phosphorus Removal – Until the formal TMDL for Mark West Creek is issued, the NCRWQCB has imposed phosphorus limits which become increasingly stringent until

zero net phosphorus loading to Mark West Creek is required. In order to meet the future net zero limit, the Town can modify its existing treatment process and create new infrastructure to deal with future phosphorus regulations. Modifications include adding a static mixer after the chemical feed pumps. Additional infrastructure can include a chemical feed facility to treat the sludge pond supernatant (sidestream treatment).

- Replacement and Rehabilitation
    - Headworks Grit Classifier Replacement – The existing headworks grit classifier is in need of replacement to ensure the continued operation of the grit washing system. This project is currently underway as part of the Influent Pump Station Screening Project.
    - Increased Standby Power – Currently the existing standby generator does not provide enough power to run the aeration blowers or the effluent pumps in a power outage. Having enough standby power to run these processes, along with the activated sludge system, will allow the WRF to keep the aeration basin biology alive and then reliably treat and discharge effluent in a power outage.
    - Headworks Screening and Washer-Compactor Replacement/Rehabilitation – The existing headworks screen and washer-compactor are in need of replacement to ensure continued optimized operation of the screening system. A new or rehabilitated Headworks screen should be designed to account for the change in solids conditions after the influent wet well screen has been installed.
    - Aeration System Improvements – Replacement of existing equipment which is nearing the end of its useful life includes:
        - The selector box in Aeration Basin A is in need of repair and it should be resized at the same time to better facilitate the growth of PAOs.).
        - Constructing a new blower building that provides adequate space for the new or existing blowers, upgraded motor controls, and better access for maintenance.
        - Replacing the blowers with more efficient units operated by VFD's.
- Capacity / Improvement Projects
    - Aeration System Improvements – Improvements to the air delivery and flow control systems will help better control the dissolved oxygen in the Biolac® basins and provide better oxygen transfer.
        - Flow meters should be installed on the blower discharge lines to better measure oxygen transfer efficiency. This project is currently being performed.
        - Pursue the ability to control amount of air delivered separately to each basin. Due to variable system inefficiencies that develop over time (components of each basin are replaced at different times), as well as different operation windows over time, each basin develops separate head losses. The use of the current air delivery system dictates that both aeration basins receive air at predetermined

rates with the single air manifold. Adding the ability to distinctly control the amount of air to each basin separately will help operations staff maximize blower energy efficiency and nutrient removal.

- The existing diffuser size could be modified, or a different type of diffused air system could be installed to provide better oxygen transfer.
- Reconfiguration (compartmentalization) of the secondary system would provide increased flexibility and allow the WRF to incrementally adjust to increased influent flow as the Town population increases. This type of compartmentalization would require additional engineering, but includes smaller basin sizes, flow splitting/flow control between the aeration basins, better RAS flow control, selector volume compartmentalization and flow splitting.
- Resizing the selector boxes in the aeration basins to better facilitate growth of PAOs (as mentioned previously).

- New Influent Screening – A new screen upstream of the influent pumps is required to remove materials that are resulting in the daily clogging of the influent pumps. This will increase the longevity and reliability of the influent pumps while increasing staff safety and saving time otherwise spent de-ragging pumps. The design phase of this project is currently underway as part of the Influent Pump Station Screening Project.
- Increased Pond Storage – Current effluent storage volumes limit the permitted ADWF capacity of the WRF to 1.9 MGD even though the treatment system is rated for 2.25 MGD. Increased storage will ensure that the WRF can meet the permitted AWDF treatment capacity, and expanded recycled water demand, through the 2040 planning period. Effluent storage analysis is provided in **Section 8**.
- Secondary Clarifier C – Increasing capacity by adding a third secondary clarifier provides redundancy for secondary treatment through the 2040 planning period. While treatment capacity will not necessarily be exceeded with the current secondary clarifier capacity, the WRF will need to operate both existing clarifiers to treat future flows. This means that the WRF will lack future redundancy unless Clarifier C is added. An additional flow control structure will be required to allow flow from either aeration basin to be sent to any of the secondary clarifiers which are online.
- AWT Clarifier 4 – Increasing the overall clarifier capacity provides redundancy for tertiary treatment process through the 2040 planning period. While treatment capacity will not be exceeded, the WRF will need three clarifiers to treat future flows. This means that the WRF will lack future redundancy unless a fourth AWT clarifier is added. The existing AWT layout includes space for a future fourth AWT clarifier.

12.2.2 Project Definition and Key Features

There are improvements that have been identified throughout the course of the Study to maintain reliable wastewater treatment at the WRF. This section provides a summary of the recommended projects

organized into the three general categories of regulatory projects, replacement and rehabilitation projects and capacity projects. The projects were prioritized based on condition and criticality of need. Timing assumptions for the projects were developed in conjunction with Town staff, recognizing the need for minimizing concurrent projects and funding limitations.

12.2.3 Future Regulatory Requirement Projects

Future regulations for phosphorus are included in the current NPDES permit for the WRF. **Section 7** documents treatment alternatives to meet the anticipated regulatory requirements for effluent phosphorus. One leg of the preferred approach to meeting future phosphorus limits, is to incorporate multi-point dosing into the Town's current treatment process. The addition of multiple chemical dosing points would not, on its own, bring the Town into compliance for future zero net phosphorus limits. In order to achieve zero net phosphorus, phosphorus credits would need to be purchased. Identified projects for phosphorus removal include:

- Installing a static mixer downstream of the current chemical injection point to better mix the alumer prior to entering the flocculation tanks.
- Resize Aeration Basin Selector Boxes. Basin B's selector box should be sized for optimal PAO growth (Basin A's selector box should be resized at the same time it is rehabilitated, so the cost of resizing Basin A's selector box is included with its rehabilitation).
- Add chemical dosing to the sludge pond supernatant (sidestream treatment). This will require capital investment in a new sidestream treatment facility.
- Add thickening and dewatering facilities that can handle sludge from the WRF at a faster rate and thus remove bioaccumulated phosphorus prior to it being reintroduced to the sludge ponds.
- Pursue an additional discharge location at the Russian River to mitigate the phosphorus burden on Mark West Creek and the Laguna during certain times of the year.

In addition to decreasing phosphorus discharge limits, more stringent biosolids disposal regulations are expected for landfill or alternative daily cover; solids treatment and disposal may require alternative treatment technologies to the Town's current solids handling process. Alternative technologies could provide stabilization and dewatering, as well as produce more valuable solids that can be disposed of in a number of ways. Certain alternative technologies can also produce valuable by-products, either in the form of soil amendment or gas that can be converted to energy to off-set the energy usage at the WRF. Solids handling projects are included in **Section 11** and include treatment alternatives that will provide solutions to help the Town continue to dispose of solids even with future regulatory changes to land disposal.

The scope of this Study only included the evaluation of four options, sludge drying beds, mechanical dewatering, mechanical thickening, and anaerobic digestion. Anaerobic digestion would likely include mechanical dewatering and mechanical thickening prior to digestion.

Restrictions on biosolids disposal have become more stringent. It is expected that this trend will continue and disposal of biosolids in the future will be further restricted. Of the four options reviewed for sludge

handling in **Section 11**, digestion was the only option that provided the WRF with a biosolids product that is expected to be able to meet the more stringent future disposal requirements while potentially providing energy as a by-product. Generally, it can be difficult for small plants to generate enough digester gas for beneficial reuse; the potential for beneficial reuse at the WRF will require additional detailed engineering study before implementation.

The digestion option could also be phased in over time, starting with permanent thickening, then adding dewatering, and finally adding the digesters, while phasing out the ponds. The first phase would provide the Town with a gravity belt thickener to allow it to concentrate solids onsite, thus allowing for more efficient use of the sludge ponds. This unit would be skid-mounted and potentially require odor control and a shade structure. Similarly, a skid-mounted dewatering unit, a belt filter press, would be installed in the second phase to provide onsite dewatering of the Town's solids. This unit would also potentially require odor control and a shade structure. The final upgrade would be the construction of an anaerobic digester and the associated solids handling building. All of the proposed infrastructure would be constructed in the area of Sludge Pond 4, which is currently not used. Phasing these upgrades will spread the capital cost out over a longer time and allow WRF staff to gain familiarity with new processes in a more manageable fashion.

Table 12.5 summarizes the liquid and solids treatment alternatives which would allow the Town to meet the identified future regulations. These improvements are based upon anticipated requirements within the next NPDES renewal cycle. NPDES permits are renewed every five years, it is not possible to forecast future regulatory requirements beyond the next renewal cycle so additional projects may be warranted as the regulatory environment changes.

Table 12.5: Regulatory Requirement Projects

Project	Description	Start Year
Phosphorus Removal (Chemical Intensive Alternative)		
Chemical dosing to AWT influent	Provide additional chemical storage, dosing pumps.	2019
Chemical dosing to sludge pond supernatant	Provide additional chemical storage, dosing pumps and piping. This will allow the plant to dose alumer at the pond supernatant before it returns to the head of the plant. The alumer will assist the phosphorus to settle out in downstream processes.	2019
Static Mixer	Provide 1 static mixer.	2019
Solids Thickening	Install a gravity belt thickener and associated odor control system. This includes ancillary piping, electrical and instrumentation. This is a precursor to anaerobic digestion and will provide longer use of existing sludge ponds.	2033
Solids Dewatering	Install a belt filter press and associated odor control system. This includes ancillary piping, electrical and instrumentation.	2034
Anaerobic Digestion	Construct a new anaerobic digester and associated odor control system. Includes ancillary piping, electrical and instrumentation components. If the Town chooses to pursue reuse, a scrubber for off gas for digester heating may be required.	2035
Sludge Pond Supernatant Sidestream Treatment	Phosphorus recovery side stream treatment is a full-scale treatment process which will treat the pond supernatant and remove phosphorus prior to the sidestream returning to the head of the plant.	2036

12.2.4 Replacement and Rehabilitation Projects

Major facility replacement and rehabilitation needs were identified in **Section 5** based on the condition, remaining useful life, and age of existing facilities and discussion with Town staff. A number of unit processes with aging infrastructure will be reaching the end of their useful life over the planning horizon; these projects are listed in **Table 12.6**. Due to the extended number of projects, brief descriptions of each project are kept within the table.

Since these projects are related to maintenance and rehabilitation of existing aging infrastructure, some of the projects will need to be repeated until the end of the Study planning period in 2040. The expected recurrence period of each project is noted in the table below. If the project is expected to recur within the Study's time horizon, that additional cost is carried in future years, as appropriate.

Table 12.6: Rehabilitation and Replacement Projects

Project	Description	Start Year	Recurrence (yrs)[1]
Headworks grit classifier replacement	Replace the existing grit classifier at the Headworks and perform repair of spalled or damaged concrete.	2018	20
Polymer System pump replacement	Replace the 2 polymer system pumps and perform repair of spalled or damaged concrete.	2018	20
Sludge Pond mixer and civil rehabilitation	Replace floating mixers within various sludge ponds.	2018	25
AWT Clarifier 2 rehabilitation	Replace internal mechanical components (center column, walkway, railing, scraper system) and perform repair of corroded or damaged areas of the steel tank.	2018	30
AWT Clarifier 1 rehabilitation	Replace internal mechanical components (center column, walkway, railing, scraper system) and perform repair of corroded or damaged areas of the steel tank.	2019	30
Secondary Clarifier A rehabilitation	Replace the internal mechanical components (center column, walkway, railing, baffles, center well, scraper and sludge rake systems) and perform repair of spalled and damaged concrete at Secondary Clarifier A.	2019	30
Blower MCC and electrical feed replacement	Replace the electrical feed to the existing 7 blowers. Replace the existing 480V, 600A MCC-I. Size MCC to physically accommodate future upgraded blower controls.	2020	20
Aeration Basin A Selector Box Replace and Resize	Repair and upgrade the existing selector box with a larger selector to better facilitate PAO growth. Consider performing concrete rehabilitation and pipe replacement at the same time (see 2024).	2020	50
Secondary Clarifier B rehabilitation	Replace the internal mechanical components (center column, walkway, railing, baffles, center well, scraper and sludge rake systems) and perform repair of spalled and damaged concrete at Secondary Clarifier B.	2020	30
Aeration Basin B Selector Box Resizing	Replace the existing selector box with a larger selector to better facilitate PAO growth.	2021	50
Main Switch Board replacement	Replace the existing main WRF electrical switch board. When the MSB is replaced a larger emergency generator to power the full WRF should be considered.	2021	20
Flocculation Tanks 1 and 2 rehabilitation	Replace the 2 mixers within each tank and perform repair of spalled or damaged concrete.	2022	30
Headworks screen, washer-compactor rehabilitation	Replace the existing fine screen and washer-compactor at the Headworks and perform repair of spalled or damaged concrete.	2022	20
High Flow Storage Pond 1 concrete rehabilitation	Perform repair of spalled or damaged concrete.	2022	50
AWT Sludge piping and pneumatic valve rehab	Replace the pneumatic valves on the AWT sludge piping and perform repair of any spalled or damaged concrete.	2023	40
AWT Sludge pump replacement	Replace 2 progressive cavity AWT sludge pumps.	2023	25
UV System quartz sleeves replacement	Replace the quartz sleeves, ballasts and lamps for the UV system. This is an ongoing item.	2024	7
AWT Filters 1 and 2 rehabilitation	Replace the mechanical internals of the existing filters.	2024	30
Flocculation Tanks 3 and 4 rehabilitation	Replace the 2 mixers within each tank and perform repair of spalled or damaged concrete.	2024	30
Laboratory Building Roof Replacement	Remove existing roof, install new roof (concrete tile).	2024	20

Project	Description	Start Year	Recurrence (yrs)[1]
RAS Pump Station A and B pump replacement	Replace the 2 RAS pumps at RAS PS A and modify the existing pipe to add a standpipe and tee. Replace 2 RAS pumps at RAS PS B.	2024	20
Aeration Basin B minor concrete rehabilitation and pipe replacement	Replace corroded piping and valves within the aeration basin.	2024	50
Secondary Clarifier flow splitting structure	Construct flow splitting structure upstream of the existing secondary clarifiers. The structure will be a buried concrete vault with associated gates and piping to allow flow to be diverted to any of the operational secondary clarifiers.	2024	50
AWT Pump Station pump replacement	Replace the 2 AWT pumps.	2027	20
Flocculation Tanks 5 and 6 rehabilitation	Replace the 2 mixers within each tank and perform repair of spalled or damaged concrete.	2027	30
WAS Pump Station pump replacement	Replace the 2 WAS pumps in the WAS PS.	2027	20
Alumer System pump replacement	Replace the 2 alumer pumps and perform repair of spalled and damaged concrete at the pump area.	2027	20
Influent Pump Station influent pumps replacement	Replace existing influent pump station pumps. Allow for minor electrical and mechanical work to install new pumps.	2029	20
Maintenance Building Roof Replacement	Remove existing roof, install new roof (concrete tile).	2029	20
UV Area Roof Replacement	Remove existing roof, install new roof (metal).	2029	20
Filters 5, 6 and 7 rehabilitation	Replace the mechanical internals of the existing filters.	2029	30
Misc. Sitewide Roof Replacement	Remove existing roof, install new roof at Generator, Influent Pump Station, Compressor, AWT Control, AWT Chemical, Effluent Pump Station, Dechlorination, Geysers Pump Station and Chlorination Buildings.	2029	20
Effluent Pump Station pump replacement and concrete rehabilitation	Replace the 4 effluent pump station pumps and perform repair of spalled or damaged concrete.	2030	20
High Flow Storage Pond 2 concrete rehabilitation and pipe replacement	Perform repair of spalled or damaged concrete and replace corroded pipes within the pond.	2030	50
No 3 Water System rehabilitation	Replace the No 3 water filters, the hydropneumatic tank, 2 pumps and associated piping, electrical and instrumentation.	2030	25
AWT Clarifier 3 rehabilitation	Replace internal mechanical components (center column, walkway, railing, scraper system) and perform repair of corroded or damaged areas of the steel tank.	2030	30
Hypochlorite System rehabilitation	Replace 3 storage tanks, 2 pumps, and associated piping and electrical and instrumentation equipment for the hypochlorite generation system. Perform repair of spalled or damaged concrete.	2039	20
Geyser Pump Station replacement of pumps	Replace the 4 Geysers PS pumps and perform repair of spalled or damaged concrete.	2039	20

[1] Recurrence periods for assets that are greater than 22 years will not fall within the timeframe of this CIP. Assets with recurrence periods less than 22 years are assumed to be replaced at the same cost for each recurrence interval.

12.2.5 Capacity/Improvement Projects

The build-out average dry weather flow and loads to the WRF were estimated based on population projections from the recently adopted 2040 General Plan and historical influent characteristics. While growth may occur at a slower rate than projected, it is prudent to plan for the population estimates identified for the long-term planning horizon. Upstream measures may be implemented that could reduce flow and/or loadings to the WRF, such as upstream storage in the collection system, onsite reuse or diversion, however, in this study, facilities were evaluated for the full projected flows and loads (especially because flow reduction measures would not reduce influent loadings and future reduction measures may or may not prove to be permanent solutions). The identified capacity related projects are shown in **Table 12.7**.

Table 12.7: Capacity/Improvement Projects

Project	Description	Start Year [3]
Blower Flow Meter installation	Add 2 flow meters on the existing discharge lines to each basin, allow for modification of the discharge lines as required.	2018
Influent Pump Station screen and washer-compactor installation	Construct a new buried chamber and install a fine screen upstream of the existing influent pumps. The project will include modification of the influent piping to the existing wet well to allow bypass of the screen for maintenance. Includes associated electrical and instrumentation work.	2018
Emergency Generator replacement	Replace existing generator with a 1600 kW unit or adequate size that can power the full operation of the WRF	2020
Aeration Basin DO Control (updated Biolac® control system) 1	Add dissolved oxygen and ammonia sensors, and associated appurtenances, to allow control of the aeration basins by Ammonia based DO-control	2020
Replace existing Biolac® diffusers	Remove and replace existing 5-ft diffusers with 7-ft diffusers from Parkson™	2020
Effluent Pump Station VFDs Installation	Install VFDs for the 4 existing effluent pump station pumps.	2020
Headworks flow bypass and Aeration flow splitting	Construct flow splitting structure upstream of the existing aeration basins. The structure will be a buried concrete vault with associated gates and piping to allow flow to be diverted to any of the operational aeration basins. This is optional to allow for flow control to each basin.	2022
AWT Filters 3 and 4 replacement	Replace the existing filters to match the other existing 5 filters.	2022
Effluent Pond operation changes	Adjust operation of the storage ponds to maximize capacity. Includes cost for additional water balance modeling and potential modifications to gates and pipes to allow the appropriate interconnections and isolations.	2023
Split Aeration Basins into smaller units	Divide each existing aeration basin into 2 basins, for a total of 4 aeration basins. This improvement will allow the Town to more efficiently operate the aeration basins from a Food to Mass perspective as well as blower efficiency as the Town population grows incrementally. A dividing wall will be constructed along the north-south centerline of each existing basin, associated diffusers, aeration piping and appurtenances will have to be replaced. This project may be performed one basin at a time. This project will utilize the Headworks flow bypass and aeration flow splitting structure listed above.	2024
Secondary Clarifier C	Construct a third secondary clarifier. The new clarifier would be west of the existing secondary clarifiers and would require backfilling of existing High Flow Equalization Pond 1. Includes mechanical, structural, civil and electrical work associated with the new clarifier, WAS and RAS pumps and piping.	2025
UV System replacement	Replace the existing UV system with a new 9-MGD system.	2026
Blower Building and Blower replacement	Replace the existing 7 blowers with new VFD controlled Blowers. Also includes demolition of the existing Blower Building and construction of a new Blower Building.	2027
Increased Effluent Storage [2]	Increase effluent storage to provide additional volume for the Town.	2028
AWT Clarifier 4 construction	Construct a fourth AWT clarifier in the empty space to the northeast of the existing AWT clarifiers. Includes associated mechanical, structural, civil and electrical work.	2030

[1] Improved DO control would increase the solids loading capacity of the aeration system.
[2] For costing purposes, involves construction of a new offsite 44 MG effluent storage pond
[3] Start year refers to start of design, construction will follow. For CIP purposes, the funds are assumed to be encumbered for the duration of the project.

12.2.6 Site Plan

Within this Study, space has been allocated on the WRF site for each identified project, with the exception of new effluent storage ponds since they are assumed to be offsite. It is important to reserve space not just for the facilities that are needed in the near term, but also for potential future needs. **Figure 12.1** shows the overall WRF site plan with anticipated space reserved for future projects.



Figure 12.1: Proposed Future Site Plan

12.2.7 Capital Improvement Plan

The sum of projects identified in the previous sections result in a significant investment at the WRF through 2040. However, some of these may not be implemented in the future unless required by regulatory changes or deemed necessary per policy decisions. In addition, decisions have not been made on which solids handling approach should be implemented, or how or where the Town could site a future effluent storage pond; these large costs have a substantial impact on the overall CIP costs. For the purposes of this Report, these costs have been included as long-term projects. Their inclusion will provide the Town with a more complete picture of potential future expenditures until a long-term plan has been decided on. A summary of the total costs for the recommended facilities through the 2040 planning horizon is shown in **Table 12.8**.

Table 12.8: Breakdown of CIP Costs by Category (Through 2040)

Category	Total Cost	Percent of CIP
Replacement and Rehabilitation	$22,200,000	19%
Capacity/Improvement[1]	$84,000,000	70%
Regulatory[2]	$13,700,000	11%
Total CIP Cost	**$119,900,000**	**100%**

[1] This includes $55M for a 44MG effluent storage pond. This cost will vary depending upon the eventual siting and size of the pond.
[2] This cost includes $4M for a 1.5MG anaerobic digester should the Town pursue that alternative.

12.2.8 Implementation and Phasing

The projects for this Study are spread through the 2040 planning horizon and some projects are dependent on either a policy decision or a regulatory driver. For the purposes of long-term CIP planning, a schedule for implementation of the projects was developed for this Study. This schedule was developed with the best available information at the time of this Study and input from Town staff. Although project priorities are generally established here, the Town may defer projects based upon budgetary constraints, regulatory drivers or other future considerations.

The projects were grouped into short-term (within 5 years), intermediate-term (6 to 10 years) and long-term (greater than 10 years) categories based upon their expected start of construction.

The short-term projects are those deemed most critical to maintaining the WRF's immediate compliance with existing permit requirements and anticipated requirements in the next NPDES renewal cycle. These projects include a mix of regulatory driven, replacement and rehabilitation and capacity upgrades. A listing of these short-term projects, in general order of priority, is included in **Table 12.9** below.

Table 12.9: Short-Term Projects (1 to 5-year Horizon)

Project	Study Section	2018 Year 0	2019 Year 1	2020 Year 2	2021 Year 3	2022 Year 4	2023 Year 5	Total[1,2]
Blower Flow Meter installation	5,6	$10,500	$94,500					$105,000
Headworks grit classifier replacement	5,9	$44,900	$404,100					$449,000
Polymer System pump replacement	5	$45,000						$45,000
Sludge Pond mixer and civil rehabilitation	5	$30,000						$30,000
AWT Clarifier 1 rehabilitation	5	$435,000						$435,000
Influent Pump Station screen and washer-compactor installation	5,9	$296,200	$2,665,800					$2,962,000
AWT Clarifier 2 rehabilitation	5		$539,000					$539,000
Secondary Clarifier A rehabilitation	5		$748,000					$748,000
Phosphorus Removal (Static mixer, chemical dosing at AWT influent and pond supernatant)	7		$99,300	$893,700				$993,000
Emergency Generator replacement	5			$416,000	$3,744,000			$4,160,000
Aeration Basin DO Control (updated Biolac® control system)	6,7			$58,297	$262,336	$262,336		$582,969
Replace existing Biolac diffusers	5,6			$106,000	$477,000	$477,000		$1,060,000
Blower MCC and electrical feed replacement	5,6			$905,000				$905,000
Aeration Basin A Selector Box Replace and Resize	5,6			$60,100	$540,900			$601,000
Secondary Clarifier B rehabilitation	5			$748,000				$748,000
Effluent Pump Station VFDs replacement	5			$158,000				$158,000
Aeration Basin B Selector Box Resizing	5,6				$60,100	$540,900		$601,000
Main Switch Board replacement	5				$172,100	$1,548,900		$1,721,000
Headworks flow bypass and Aeration flow splitting	5,6					$74,600	$671,400	$746,000
Flocculation Tanks 1 and 2 rehabilitation	5					$431,000		$431,000
Headworks screen, washer-compactor rehabilitation	5,9					$897,000		$897,000
High Flow Storage Pond 1 concrete rehabilitation	5					$57,360		$57,360
AWT Filters 3 and 4 replacement	5					$174,500	$1,570,500	$1,745,000
Effluent Pond operation changes	8						$334,000	$334,000
AWT Sludge piping and pneumatic valve rehab	5						$73,000	$73,000
AWT Sludge pump replacement	5						$159,000	$159,000
TOTAL		**$900,000**	**$4,600,000**	**$3,300,000**	**$5,300,000**	**$4,500,000**	**$2,800,000**	**$21,300,000**

[1] Project costs include construction costs, engineering, construction management, permitting, administration, legal and contingency.
[2] Project costs are AACE Class 4 and are intended to provide planning level estimates of project cost. The expected accuracy of the costs is -30% to +50%. More detailed costs should be determined during the preliminary and final design phase of each project.

Intermediate-term projects are those projects which have been identified as necessary to maintaining the WRF's level of service but are not immediately critical. These projects are mostly replacement and rehabilitation projects that are either on non-critical infrastructure, or for assets that are not currently at the end of their useful life. These intermediate term projects are listed in **Table 12.10** below.

Table 12.10: Intermediate-Term Projects (6 to 10 -year Horizon)

Project	Study Section	2024 Year 6	2025 Year 7	2026 Year 8	2027 Year 9	2028 Year 10	Total[1,2]
Split Aeration Basins into smaller units	6	$372,900	$3,356,100				$3,729,000
UV System quartz sleeves replacement	+[3]	$94,000					$94,000
AWT Filters 1 and 2 rehabilitation	5	$974,000					$974,000
Flocculation Tanks 3 and 4 rehabilitation	5	$431,000					$431,000
Laboratory Building Roof Replacement	+[3]	$353,000					$353,000
RAS Pump Station A and B pump replacement	5	$374,000					$374,000
Aeration Basin B minor concrete rehabilitation and pipe replacement	5	$145,000					$145,000
Secondary Clarifier flow splitting structure	6	$71,800	$646,200				$718,000
Secondary Clarifier C	6		$533,700	$4,803,300			$5,337,000
UV System replacement	5,10			$266,900	$2,402,100		$2,669,000
Blower Building and Blower replacement	5,6				$461,900	$4,157,100	$4,619,000
AWT Pump Station pump replacement	5				$249,000		$249,000
Flocculation Tanks 5 and 6 rehabilitation	5				$431,000		$431,000
WAS Pump Station pump replacement	5				$109,000		$109,000
Alumer System pump replacement	5				$73,000		$73,000
Increased Effluent Storage	**8**					$5,469,600	$5,469,600[4]
TOTAL		**$2,800,000**	**$4,500,000**	**$5,100,000**	**$3,700,000**	**$9,600,000**	**$25,800,000**

[1] Project costs include construction costs, engineering, construction management, permitting, administration, legal and contingency.
[2] Project costs are AACE Class 4 and are intended to provide planning level estimates of project cost. The expected accuracy of the costs is -30% to +50%. More detailed costs should be determined during the preliminary and final design phase of each project.
[3] Project included based upon input from Town staff.
[4] Cost shown is only for portion which falls within the Intermediate Project window, total project cost is $55M. This project cost is based upon a 44MG pond, but the project cost will vary depending upon the eventual siting and size of the pond.

Long-term projects are those projects which have been identified based upon the expected end of useful life of assets, future regulatory requirements or future flow projections. These projects are not immediately critical as they deal with anticipated conditions at the WRF at a later date. A listing of these long-term projects is included in **Table 12.11** below. Due to the extended timeframe of the long-term projects, it is difficult to project the precise schedule of projects. WRF needs may dictate that a project be accelerated or delayed; however, the following table is provided as a potential pathway to implementation of the long-term projects.

Table 12.11: Long-Term Projects (11 to 22-year Horizon)

Project	Study Section	2029 Year 11	2030 Year 12	2031 Year 13	2032 Year 14	2033 Year 15	2034 Year 16	2035 Year 17	2036 Year 18	2037 Year 19	2038 Year 20	2039 Year 21	2040 Year 22	Total $M[1,2]
Increased Effluent Storage	8	$5,469,600	$10,939,200	$10,939,200	$10,939,200	$10,939,200								$50.0[4]
Influent Pump Station influent pumps replacement	5	$1,157,000												$1.16
Maintenance Building Roof Replacement	+[3]	$490,000												$0.49
UV Area Roof Replacement	+[3]	$170,000												$0.17
Filters 5, 6 and 7 rehabilitation	5	$1,461,000												$1.46
Misc. Sitewide Roof Replacement	+[3]	$150,300	$1,352,700											$1.50
AWT Clarifier 4 construction	+[3]		$286,900	$2,582,100										$2.87
Effluent Pump Station pump replacement and concrete rehabilitation	5		$421,000											$0.42
High Flow Storage Pond 2 concrete rehabilitation and pipe replacement	5		$58,000											$0.06
No 3 Water System rehabilitation	5		$590,000											$0.59
AWT Clarifier 3 rehabilitation	5		$435,000											$0.44
Solids Thickening	11					$210,200	$1,891,800							$2.10
Solids Dewatering	11						$390,300	$3,512,700						$3.90
Anaerobic Digestion	11							$333,600	$3,002,400					$3.34
Sludge Pond Supernatant Sidestream Treatment	+[3]								$333,600	$3,002,400				$3.34
Hypochlorite System rehabilitation	5											$175,000		$0.18
UV System quartz sleeves replacement				$94,000							$94,000			$0.19
Headworks grit classifier replacement											$224,500			$0.22
Polymer System pump replacement											$45,000			$0.01
Blower MCC and electrical feed replacement													$905,000	$0.90
Geyser Pump Station replacement of pumps	5											$359,000		$0.36
TOTAL		**$8,900,000**	**$14,100,000**	**$13,600,000**	**$10,900,000**	**$11,100,000**	**$2,300,000**	**$3,800,000**	**$3,300,000**	**$3,000,000**	**$400,000**	**$500,000**	**$900,000**	**$72.97**

[1] Project costs include construction costs, engineering, construction management, permitting, administration, legal and contingency.
[2] Project costs are AACE Class 4 and are intended to provide planning level estimates of project cost. The expected accuracy of the costs is -30% to +50%. More detailed costs should be determined during the preliminary and final design phase of each project.
[3] Project included based upon input from Town staff.
[4] Cost shown is only for portion which falls within the Long-term Project window, total project cost is $55M. Total project duration is 6 years and includes a study phase, design and permitting phase and construction. This project cost is based upon a 44MG pond, but the project cost will vary depending upon the eventual siting and size of the pond.

The costs for implementation of all the identified projects through the 2040 planning horizon are $120 million, assuming costs for anaerobic digestion ($3.3 million) for the solids handling approach and an additional off-site effluent storage pond of 44-million gallon capacity ($54.7 million). It should be noted that some of these projects may not be implemented if a regulatory driver or policy decision is not enacted.

Many of the identified projects are smaller rehabilitation projects that will be funded through the existing WRF ongoing budget. The Town's current CIP allocation for the WRF averages approximately $1.36 million per year over the fiscal years 2015-2019.

Larger CIP projects will require alternative funding through mechanisms such as State Revolving Fund (SRF) loans, or bonds. **Figure 12.2** shows the overall cash flow for all projects identified in the CIP program, broken down by funding source. A year-by-year breakdown of expenditure by project type is included in **Table 12.12**.

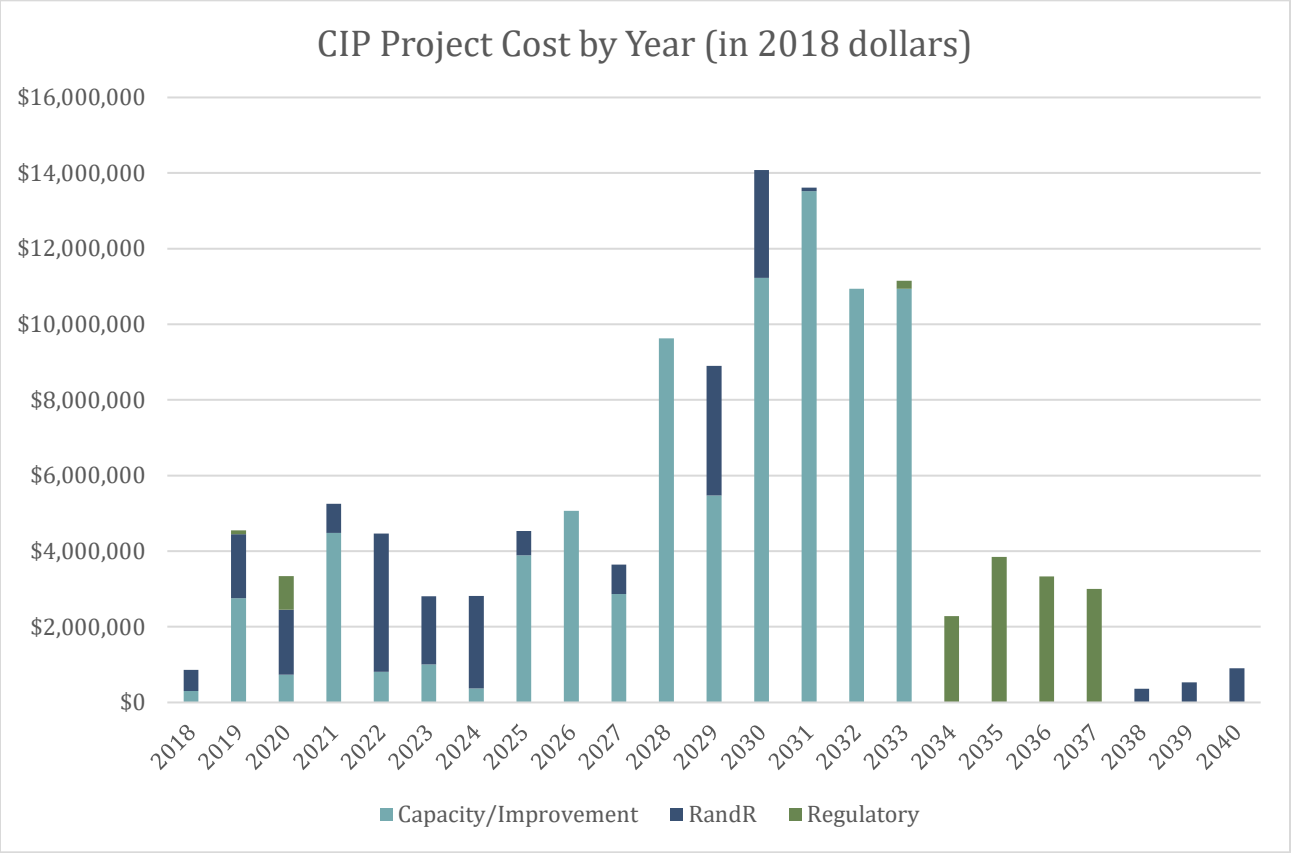


Figure 12.2: Annual CIP Expenditures by Year (in 2018 dollars)

Table 12.12: Year-by-Year Expenditure by Category

Year	R and R	Capacity/Improvement	Regulatory	Total[1,2]
2018	$600,000	$300,000	$0	$900,000
2019	$1,700,000	$2,800,000	$100,000	$4,600,000
2020	$1,700,000	$700,000	$900,000	$3,300,000
2021	$800,000	$4,500,000	$0	$5,300,000
2022	$3,600,000	$800,000	$0	$4,400,000
2023	$1,800,000	$1,000,000	$0	$2,800,000
2024	$2,400,000	$400,000	$0	$2,800,000
2025	$600,000	$3,900,000	$0	$4,500,000
2026	$0	$5,100,000	$0	$5,100,000
2027	$800,000	$2,900,000	$0	$3,700,000
2028	$0	$9,600,000	$0	$9,600,000
2029	$3,400,000	$5,500,000	$0	$8,900,000
2030	$2,900,000	$11,200,000	$0	$14,100,000
2031	$100,000	$13,500,000	$0	$13,600,000
2032	$0	$10,900,000	$0	$10,900,000
2033	$0	$10,900,000	$200,000	$11,100,000
2034	$0	$0	$2,400,000	$2,400,000
2035	$0	$0	$3,800,000	$3,800,000
2036	$0	$0	$3,300,000	$3,300,000
2037	$0	$0	$3,000,000	$3,000,000
2038	$400,000	$0	$0	$400,000
2039	$500,000	$0	$0	$500,000
2040	$900,000	$0	$0	$900,000
TOTAL	**$22,200,000**	**$84,000,000**	**$13,700,000**	**$119,900,000**

[1] Project costs include construction costs, engineering, construction management, permitting, administration, legal and contingency.
[2] Project costs are AACE Class 4 and are intended to provide planning level estimates of project cost. The expected accuracy of the costs is -30% to +50%. More detailed costs should be determined during the preliminary and final design phase of each project.

12.2.9 Funding

The adequate funding of capital projects usually is a primary constraint in the CIP implementation process. The WRF has several funding options available for financing its projects that are briefly discussed below.

Typically, an agency does not have sufficient revenue to fund large capital improvements directly from user fees, and usually needs to rely on additional funding sources. The main financing sources available to the WRF for funding the capital costs include:

- Pay-as-you-go financing
- Debt financing
- Grants and loans

Pay-as-you-go financing refers to the upfront collection of project costs from existing and new users for future capital improvement projects. Pay-as-you-go financing generally requires rate increases and can create cash flow problems. This method tends to be used to fund smaller CIP projects.

Debt financing refers to the acquisition of funds through borrowing mechanisms such as bonds. Debt financing requires the borrower to raise money for working capital or capital expenditures by selling bonds, bills, or notes to individual and/or institutional investors. In return for money, the individuals or institutions become creditors and receive a promise to repay principal and interest on the debt.

Grants and loans provide an alternate source of funds at minimal cost. Federal, State, and local grants provide funding for projects that meet select criteria. Grant funding is limited, sometimes specific to a particular type of project, and is generally not a long-term solution to meet financing needs. State and Federal loan programs provide low-cost methods of borrowing for projects that meet select criteria. Most projects receiving grant and loan funding generally will need to secure supplemental funding sources. The state has recently been providing grants for projects related to potable water offset (i.e. use of recycled water) and storm water management, these are two areas which the Town can pursue additional revenue sources to defray the cost of infrastructure, particularly recycled water storage. Teaming with other local agencies such as Sonoma Water and the City of Santa Rosa could provide a competitive advantage to grant proposals it shows the Town is seeking regional solutions to the water supply issues.

12.2.10 Sequencing

This section identifies some of the sequencing associated with the implementation of the WRF CIP.

Sequencing constraints can sometimes determine the order in with certain projects get implemented especially if there is impact on the entire WRF or portion of the WRF that must remain operational to ensure continued compliance.

In general, the smaller replacement/rehabilitation projects can be completed such that they do not impact the continuous operation of the facility. The larger projects will need to be evaluated on a case-by-case basis to determine the correct sequencing so that the WRF can provide reliable treatment, especially because bypassing of flow or high flow storage may need to be implemented to complete certain projects. However, in general most of the projects can be phased with construction occurring largely during dry weather seasons. For example, the replacement/rehabilitation of the clarifier equipment will require scheduling the work during the dry weather season where not all clarifiers are not required. Replacement of UV equipment will also require the work to be performed in phases during the dry weather season; one channel at a time to maintain continuous disinfection treatment. Sequencing around constraints can result in a longer construction and implementation period and a constructability review to be included as part of the planning and design process.

The capacity related projects also have sequencing constraints that need to be taken into account. Since one Biolac® basin must be operational at all times, the improvement projects must be coordinated to ensure the Town continues to meet all permit requirements. The first priority is to increase the reliability

of airflow to the Biolac® process. Currently, the Town lacks the ability to measure the volume of air going to each basin. Additional electrical limitations prevent all blowers from being run concurrently. By installing flow meters on the process air lines to the basins and upgrading the blower MCC feed, the Town will increase the reliability and flexibility of the existing blowers.

Once the blower reliability has been addressed, the Biolac® basins should be upgraded. The Town should take advantage of Basin A being offline to perform upgrades to the existing equipment as well as adding additional controls. The diffusers within the basin should be replaced with larger units which can provide more air to the process. This will also resolve the issue with the existing diffusers described in **Section 6**; the existing diffusers directly adjacent to the sides of the basin were set at higher elevations and air preferentially flows through the outer diffusers. New diffusers should be placed at one elevation. At the same time the existing selector box can be rebuilt with a larger unit to enhance phosphorus removal. Ammonia-based DO control should be installed at this time as well to allow the Town to more precisely control the flow of air to the system, optimizing energy use and increasing nitrogen removal. Once Basin A upgrades have been completed, Basin B can be shut down and similar modifications made to bring both basins up to the same operational standards.

After the above upgrades have been made to both basins, or concurrently with the upgrades to Basin B, a new flow splitting structure may be constructed between the headworks and the Biolac® basins. Currently, the Town has no ability to actively control the flow split to each basin and operators have observed uneven loading when both basins are operational. The flow splitting structure would allow the Town to control the volume of primary influent going to each basin to equalize loading and maintain a more uniform process.

Replacing the headworks screen should be examined by the Town after the forthcoming influent wet well screen has been installed and normally operated for at least one full year. If it is determined that the headworks screen should be replaced or refurbished, a new washer compactor system should be installed as well, but perhaps only after the above repairs to the Biolac® basins are completed. This will allow the basins to handle the additional loading of any excess organics that may be washed back into the process water.

Future regulatory projects are related to compliance with the effluent nutrient limits as well as biosolids regulations. For compliance with nutrient limits, one of the elements includes improvements to the chemical addition system. These improvements are not tied to the aeration basin improvements and can be phased in at any time. The improvements can occur with the WRF in operation, except for the final tie-in which would be coordinated to minimize significant impact to treatment reliability. If implemented, the solids handling projects (dewatering, thickening and digestion) can be phased in without significant impact since the current sludge pond treatment process can operate independently of any new processes. There will need to be coordination during the pond dredging season to minimize the impact of construction to the normal biosolids handling process as the location of the new solids handling facilities will likely be in the general area of the current annual pond dredging and hauling operations.

Increasing the effluent flowrates to the Geysers Steam Field, under contract with the City of Santa Rosa, should be sequenced along with additional storage to make up for lost irrigation water. Alternatively, the water may be sourced from an agreement with a neighboring agency.

Appendix A: Town of Windsor NPDES Permit

Refer to electronic version of the report for the complete appendix. The first page is shown here for reference.

CALIFORNIA Water Boards

Edmund G. Brown Jr. Governor

Matthew Rodriquez Secretary for Environmental Protection

North Coast Regional Water Quality Control Board

ORDER NO. R1-2013-0042
NPDES NO. CA0023345
WDID NO. 1B82037OSON

WASTE DISCHARGE REQUIREMENTS AND MASTER RECLAMATION PERMIT FOR THE TOWN OF WINDSOR
WASTEWATER TREATMENT, RECLAMATION AND DISPOSAL FACILITY

SONOMA COUNTY

The following Permittee is subject to waste discharge requirements as set forth in this Order:

Table 1. Permittee Information

Permittee	Town of Windsor
Name of Facility	Windsor Wastewater Treatment, Reclamation and Disposal Facility
Facility Address	8400 Windsor Road Windsor, California 95492 Sonoma County
Type of Facility	Publicly-Owned Treatment Works
Facility Design Flow	2.25 million gallons per day (MGD) (average dry weather design flow) 7.2 MGD (peak weekly wet weather design flow) 3.75 MGD (peak monthly wet weather design flow)

Table 2a. Discharge Locations

Discharge Point	Effluent Description	Discharge Point Latitude	Discharge Point Longitude	Receiving Water
001	Disinfected tertiary municipal effluent	---	---	Town-owned effluent storage ponds
002	Disinfected tertiary municipal effluent	38 °, 29', 39" N	122 °, 51', 05" W	Mark West Creek (at Trenton-Healdsburg Road Bridge)

Appendix B: Conditions Assessment (September 2015)

Refer to electronic version of the report for the complete appendix. The first page is shown here for reference.

Hazen *Tech Memo No. 1*

September 18, 2015

Re: Windsor Wastewater Reclamation Plant Modernization Study

TM 1 [Task 2] Condition Assessment

DRAFT

Hazen and Sawyer • 201 Mission Street, Suite 500, San Francisco, CA, 94105

Appendix C: Capital Replacement and Maintenance Plan – Brelje and Race (2013)

Refer to electronic version of the report for the complete appendix. The first page is shown here for reference.

TOWN OF WINDSOR

CAPITAL REPLACEMENT AND MAINTENANCE PROGRAM

TOWN OF WINDSOR

FEBRUARY 2013

Brelje & Race Consulting Engineers
475 Aviation Blvd., Suite 120
Santa Rosa, CA 95403
v. 707.576.1322 • f. 707.576.0469
www.brce.com

Appendix D: Process Air Calculations and Review of Design and Current Loadings

Appendix D: Process Air Calculation and Review of Design and Current Loadings

Table D– 1 summarizes the design and current loadings at the WRF. The design loads are based on design documentation, and the current loads are based on a statistical analysis of WRF influent data for 2015 through 2017. The "Current (2015 – 2017)" concentrations were calculated based on logged historical values at the WRF. The TKN for the 2015 through 2017 data was calculated from NH_3-N lab data, and assumed an NH_3-N to TKN ratio of 0.66. Current ADWF BOD_5 and TKN loads are 40% lower and 11% lower than the respective 1999 design loads.

Table D– 1: Design Criteria and Current Operation

Parameter	1999 Design*	Current (2015 - 2017)	
	ADWF	Average	Max Month
Flow (MGD)	2.25	2.3	3.6
BOD5 (mg/L)	260	203	-
BOD5 Load (lbs/day)	6,505	3,894	4,740
TKN (mg/L)	45	52	-
TKN Load (lbs/day)	1,126	1,000	1,330

**Phase IIA Stage 2 Improvements, Drawing 4 of 42*

Process Air Requirements

In order to develop accurate process air estimates, Hazen took into account the unique nature of the Biolac® process and equipment, including:

1. Standard oxygen transfer efficiency (SOTE) characteristics of Parkson Biofusers. Manufacturer's information is included in **Figure D- 1**.
2. Assumption that approximately 50% of the tankage is anoxic (unaerated) at any given time.
    a. Based on discussions with Parkson, this is the typical assumption made during airflow requirements analysis with the Biolac® *Wave-Ox* system in place.
    b. Assumption results in 50% of the available diffusers being used for process air SOTE calculations. The decrease in the number of available diffusers pushes more air through each operational diffuser which decreases oxygen transfer efficiency.
3. Assumed a dissolved oxygen set point of 1.0 mg/L for aerobic zones.
4. All calculations assume two (2) aeration basins are in operation.
    a. Parkson design calculations assume two (2) basins in operation.
    b. Operation using one (1) basin results in higher airflow requirements due to less diffusers in operation and an overall decrease in SOTE.

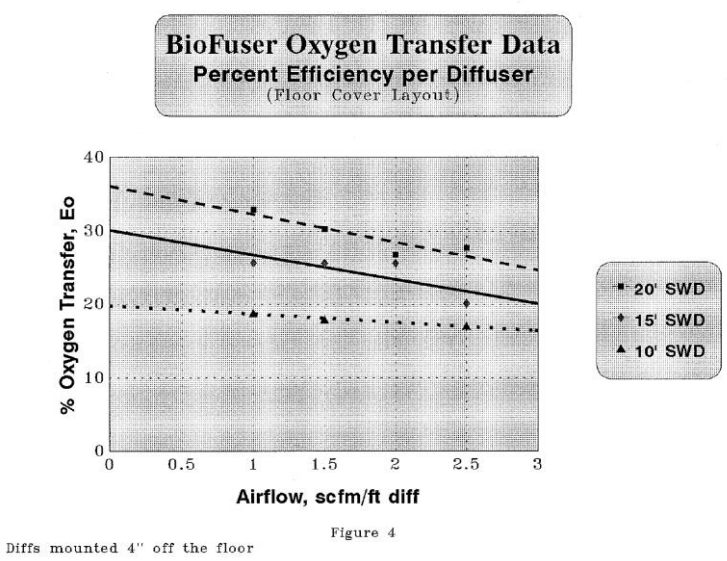


Figure D- 1: Parkson Biofuser Oxygen Transfer Efficiency Data

Table D– 2 summarizes design and operational assumptions used for the process air demand calculations. The alpha factor accounts for difference between the oxygen transfer efficiency in the wastewater versus clean water. The alpha factor can be affected by factors such as the solids retention time, the presence of surfactants, etc. It should be noted that the alpha factor is not constant, and should be chosen conservatively. If the alpha factor is over-estimated, the result will be insufficient process air capacity. Information such as diffuser submergence and lengths were derived from the design documentation and Contract Drawings. Diffuser submergence is defined as the distance between the floor and the bottom of the diffuser. The diffuser length calculations are based on 85, 3.7-foot diffusers per chain for a total diffuser length of 3,152 feet per basin. The diffuser quantities and design were verified by Parkson application specialists.

Table D– 2: Process Air Calculation Assumptions

Parameter	Value	Units	Notes
Tank Depth	14.25	Ft	Based on Contract Drawings
Diffuser Submergence	13.25	Ft	Based on Contract Drawings
Aeration Chains/Basin	10		Based on Biolac® Documentation
Length/Basin	3,152	Ft	Based on Biolac® Documentation
Alpha Factor	0.6		Parkson uses 0.7 for Biolac process. Alpha of 0.6 more conservative
Beta Factor	0.95		Typical
BOD_5 O_2 Requirement	1.4	lbs O_2/lbs BOD_5	Typical
TKN O_2 Requirement	4.57	lbs O_2/lbs TKN	Typical
Denitrification Credit	2.86	lbs O_2/lbs N Removed	Typical
TKN assimilation	0.033	lbs TKN/lbs BOD_5 Oxidized	Typical

Based on these assumptions and loads, Hazen derived the process air requirements summarized in **Table D– 3**. The final two rows in the table are the process air requirements with and without the denitrification credit. The denitrification credit accounts for carbonaceous BOD_5 removal via the denitrification pathway, which does not require additional oxygen input. Based on Biolac® operating

data, the process typically denitrifies to approximately 5 mg/L NO_3-N without the addition of external carbon.

Table D– 3: Process Air Demand Calculations

Parameter	Current Average	1999 Design Average
1. Input Conditions		
Aeration Basin Influent Flow (mg/L)	2.3	2.25
Number of Basins in Service	2	2
Biolac Influent BOD5 (mg/L)	203	260
Biolac Influent TKN (mg/L)	52	45
Biolac Effluent BOD5 (mg/L)	1.0	1.0
Biolac Effluent TKN (mg/L)	1.0	0.9
Biolac Effluent NO3-N (mg/L)	5.0	5.0
Design Wastewater Temperature (°C)	20	20
DO Concentration (mg/L)	1	1
2. Process Oxygen Requirements		
BOD5 Oxidized (lb/day)	3,881	4,860
TKN Oxidized (lb N/day)	851	666
BOD5 Oxygen Demand (lb O2/day)	5,433	6,804
TKN Oxygen Demand (lb O2/day)	3,887	3,041
Denitrification Credit (lb O2/day)	-2,159	-1,635
Actual Oxygen Requirement (AOR) (lb O2/day)	7,162	8,211
AOR/SOR	0.51	0.51
SOR (lbO2/hr)	583	669
3. Aeration Demands		
Air required at 100% Transfer Efficiency (scfm)	558	640
Basin Under Aeration (%)	50%	50%
Typical Length of Diffusers in Operation (feet)	3,152	3,152
SOTE at Design Submergence and Diffuser Flow	0.24	0.24
Air Required w/Denit Credit (scfm)	**2,297**	**2,677**
Air Required w/o Denit Credit (scfm)	**4,059**	**4,367**

Table D– 4 summarizes the calculations for the future maximum month airflow requirements based on estimates of population growth. According to these calculations, the capacity of the blowers (with the existing electrical limitation) should be sufficient for all future maximum month loading conditions. **Table D– 5** summarizes the calculations for future maximum day process air requirements. While the total design capacity of the system is not exceeded, the existing electrical limitation to process air capacity is anticipated to be exceeded before 2027.

Table D– 4: Future Max Month Airflow Requirements

Parameter	2017 MM (Current)	2022 MM	2027 MM	2040 MM
1. Input Conditions				
Aeration Basin Influent Flow (MGD)	3.6	3.9	4.0	4.8
Number of Basins in Service	2	2	2	2
Biolac Influent BOD5 (mg/L)	160	160	160	160
Biolac Influent TKN (mg/L)	45	45	45	45
Biolac Effluent BOD5 (mg/L)	1.0	1.0	1.0	1.0
Biolac Effluent TKN (mg/L)	1.4	1.4	1.4	0.9
Biolac Effluent NO3-N (mg/L)	5.0	5.0	5.0	5.0
Design Wastewater Temperature (°C)	20	20	20	20
DO Concentration (mg/L)	1	1	1	1
2. Process Oxygen Requirements				
BOD5 Oxidized (lb/day)	4,739	4,981	5,345	6,439
TKN Oxidized (lb N/day)	1,145	1,235	1,320	1,575
BOD5 Oxygen Demand (lb O2/day)	6,634	6,974	7,484	9,015
TKN Oxygen Demand (lb O2/day)	5,235	5,642	6,031	7,198
Denitrification Credit (lb O2/day)	-3,002	-3,233	-3,452	-4,123
Actual Oxygen Requirement (AOR) (lb O2/day)	8,867	9,383	10,062	12,090
AOR/SOR	0.51	0.51	0.51	0.51
SOR (lbO2/hr)	722	764	820	985
3. Aeration Demands				
Air required at 100% Transfer Efficiency (scfm)	691	732	785	943
Basin Under Aeration (%)	50%	50%	50%	50%
Typical Length of Diffusers in Operation (feet)	3,152	3,152	3,152	3,152
SOTE at Design Submergence and Diffuser Flow	0.24	0.23	0.23	22.37%
Air Required w/Denit Credit (scfm)	**2,923**	**3,118**	**3,385**	**4,214**
Air Required w/o Denit Credit (scfm)	**4,059**	**4,367**	**4,749**	**5,983**

Table D– 5: Future Max Day Airflow Requirements

Parameter	2017 MD	2022 MD	2027 MD	2040 MD
1. Input Conditions				
Aeration Basin Influent Flow (mg/L)	4	4	4	4
Number of Basins in Service	2	2	2	2
Biolac Influent BOD5 (mg/L)	206	216	232	280
Biolac Influent TKN (mg/L)	58	62	66	79
Biolac Effluent BOD5 (mg/L)	1.0	1.0	1.0	1.0
Biolac Effluent TKN (mg/L)	1.2	1.2	1.3	1.6
Biolac Effluent NO3-N (mg/L)	5.0	5.0	5.0	5.0
Design Wastewater Temperature (°C)	20	20	20	20
DO Concentration (mg/L)	1	1	1	1
2. Process Oxygen Requirements				
BOD5 Oxidized (lb/day)	6,831	7,183	7,711	9,295
TKN Oxidized (lb N/day)	1,664	1,793	1,917	2,287
BOD5 Oxygen Demand (lb O2/day)	9,563	10,056	10,795	13,012
TKN Oxygen Demand (lb O2/day)	7,603	8,195	8,759	10,453
Denitrification Credit (lb O2/day)	-4,281	-4,651	-5,005	-6,065
Actual Oxygen Requirement (AOR) (lb O2/day)	12,885	13,599	14,549	17,401
AOR/SOR	0.51	0.51	0.51	0.51
SOR (lbO2/hr)	1,049	1,108	1,185	1,417
3. Aeration Demands				
Air required at 100% Transfer Efficiency (scfm)	1,005	1,060	1,134	1,357
Basin Under Aeration (%)	50%	50%	50%	50%
Typical Length of Diffusers in Operation (feet)	3,152	3,152	3,152	3,152
SOTE at Design Submergence and Diffuser Flow	0.22	0.22	0.21	20.04%
Air Required w/Denit Credit (scfm)	**4,558**	**4,886**	**5,319**	**6,771**
Air Required w/o Denit Credit (scfm)	**5,896**	**8,024**	**8,653**	**10,697**

Appendix E: Secondary Clarifier State Point Analysis

Appendix E: Secondary Clarifier State Point Analysis

As part of the Windsor WRF capacity analysis, Hazen conducted a state point analysis on the secondary clarifiers. A state point analysis is a graphical method of assessing the clarifier capacity at different flow rates and MLSS values. Based on the SVI, Hazen predicts the gravity settling velocity and gravity flux curve for the WRF secondary sludge.

Figure E- 1 shows a typical state point analysis graph. Both the state point and the underflow rate line should fall below the gravity flux curve to ensure solids capture in the clarifier.

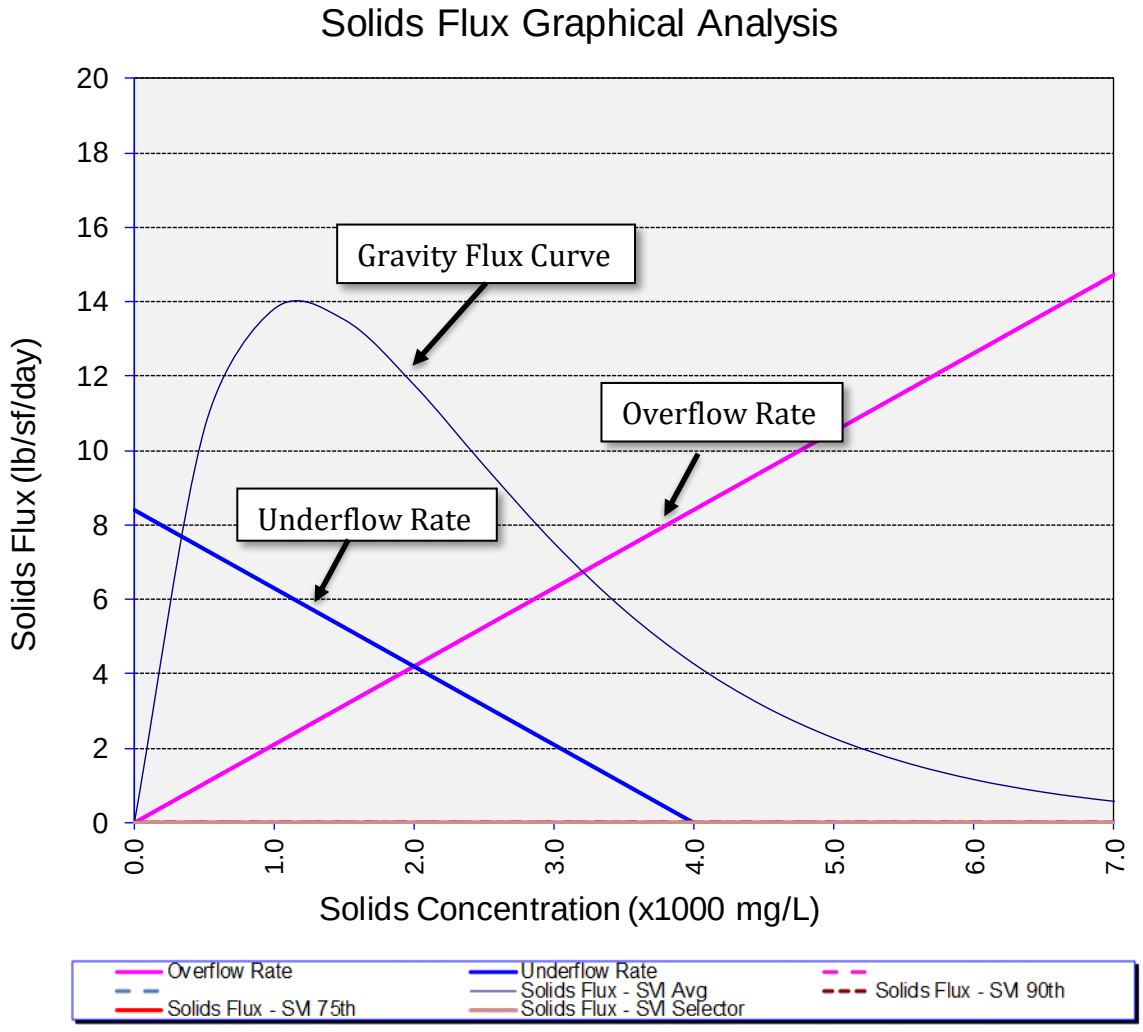


Figure E- 1: State Point Analysis Example Graph

Based on the January 2015 through November 2017 data set, Hazen utilized the following assumptions:

1. Average SVI: 258 mL/g

2. 75th Percentile SVI: 291 mL/g
3. MLSS: 2,000 – 3,000 mg/L
4. ADWF: 2.4 mgd
5. RAS:Influent Ratio: 1

As described in **Section 4.1.4**, any SVI above 150 mL/g is typically characterized as poor-settling sludge. However, due to the fact that the clarifiers are performing adequately at current flows and loads, Hazen utilized less conservative factors for gravity settling to ensure clarifier capacity results were not overly conservative.

Table E- 1 and **Table E- 2** summarize the predictions for clarifier performance at current and anticipated Future 2040 flow rates. Clarifier performance at average flow was assessed using the 75th-percentile SVI, and clarifier performance at peak flow rate was assessed using the average SVI. This approach was utilized due to the expected rarity of an overlap between peak flows and high SVI. A more conservative approach would be to assess clarifier performance at peak flow using the 75th percentile SVI.

Table E- 1: Secondary Clarifier State Point Analysis Summary Current Flows

				2 Clarifiers		
MLSS (mg/L)	**SVI (mL/g)**	**RAS Flow (MGD)/Clarifier**	**Mixed Liquor Flow Rate (MGD)/Clarifier**	**Adequate Capacity? (Y/N)**	**Overflow Rate (gal/ft²-d)**	**Solids Loading Rate (lb/ft²-d)**
2,000	291 (75th Percentile)	2.4	2.4	Y	202	6.7
2,500				Y		8.4
3,000				N		10.1
2,000	258 (Average)	4.0	4.0	Y	336	11.2
2,500				Y		14.0
3,000				N		16.8

Table E- 2: Secondary Clarifier State Point Analysis Summary Future Flows

				2 Clarifiers		
MLSS (mg/L)	**SVI (mL/g)**	**RAS Flow (MGD)/Clarifier**	**Mixed Liquor Flow Rate (MGD)/Clarifier**	**Adequate Capacity? (Y/N)**	**Overflow Rate (gal/ft²-d)**	**Solids Loading Rate (lb/ft²-d)**
2,000	291 (75th Percentile)	3.4	3.4	Y	286	9.5
2,500				N		11.9
3,000				N		14.3
2,000	258 (Average)	4.0	4.0	Y	336	9.5
2,500				Y		11.9
3,000				N		14.3

Based on discussions with Parkson and the current operation of the WRF, it was assumed that an MLSS 2,500 mg/L was required for reliable secondary treatment operation. Based on the current SVI values, clarifier settling capacity will be exceeded at the Future 2040 flow rates.

Figure E- 2 depicts the state point analysis for the Future 2040 flow rates at 2,500 mg/L MLSS. The two gravity flux curves compare the effect of reducing the SVI to 175 mL/g (75th percentile) from the current 291 mL/g.

Figure E- 3 compares the state point for two versus three clarifiers at the existing 75th percentile SVI. As shown, the effect of reducing SVI has a much greater effect on clarifier capacity than adding a third unit.

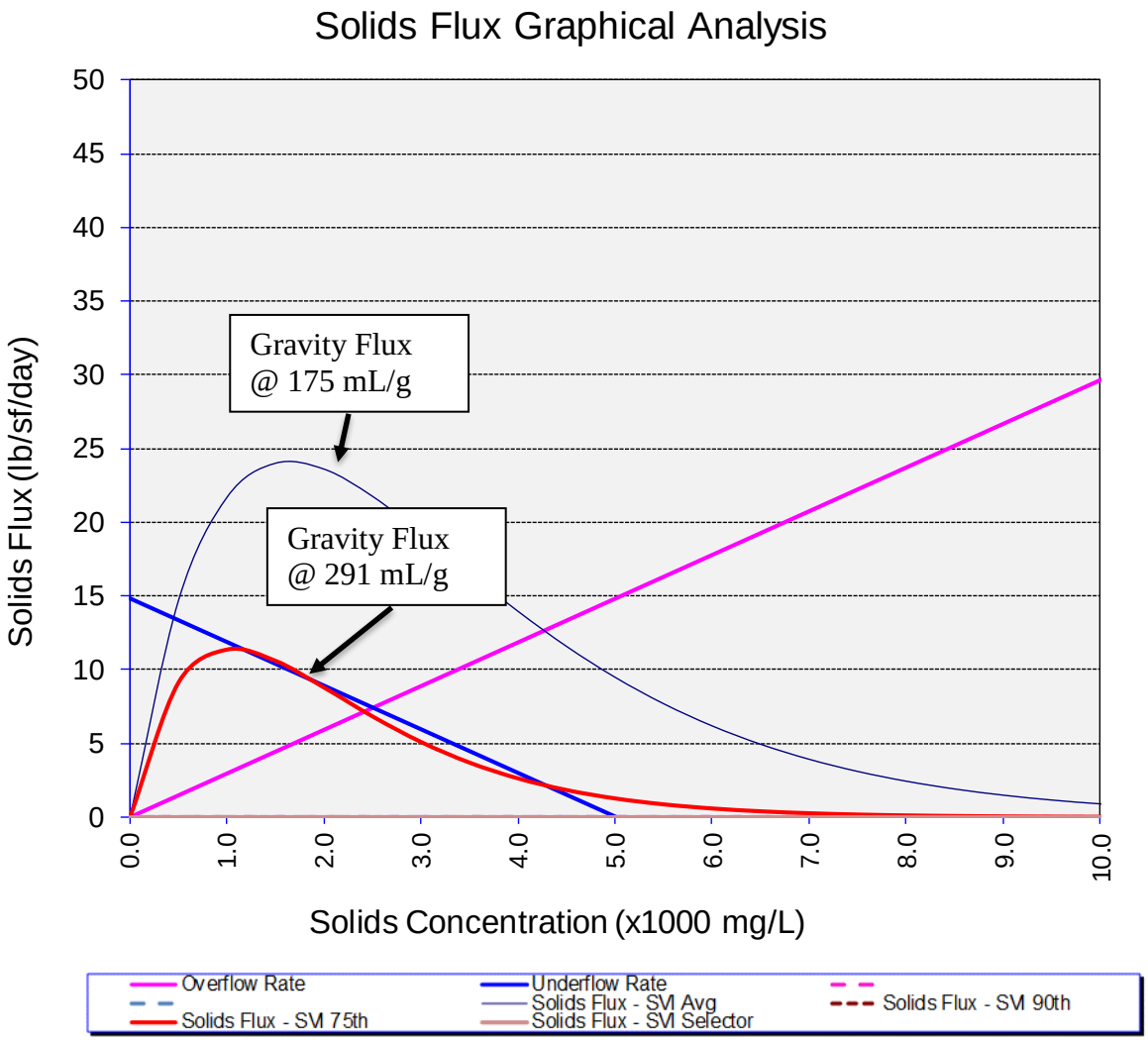


Figure E- 2: State Point Analysis Effect of Reduced SVI, MLSS 2,500 mg/L

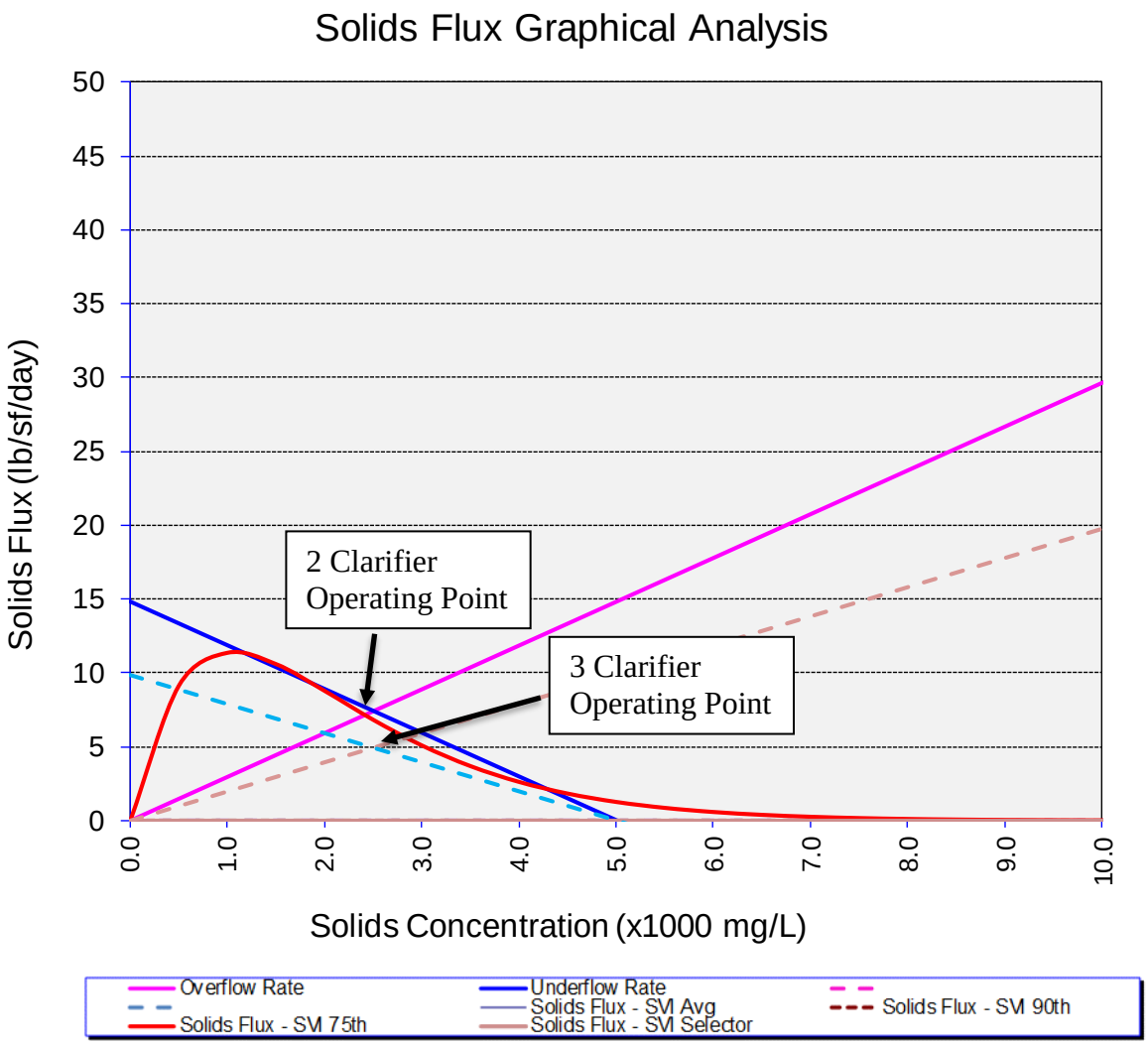
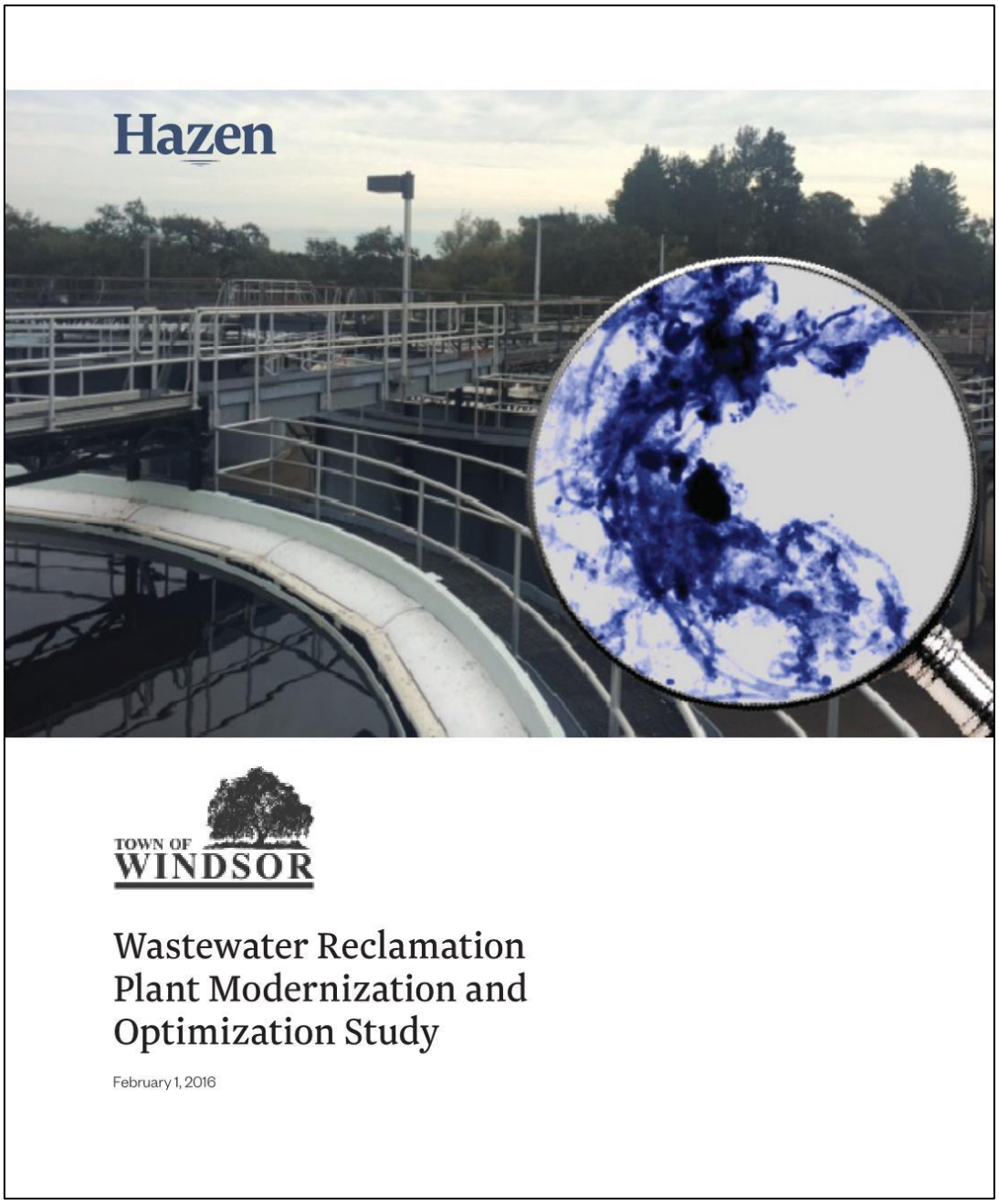


Figure E- 3: State Point Analysis Effect of Additional Clarifier, MLSS 2,500 mg/L

Appendix F: Modernization and Optimization Study (February 1, 2016)

Refer to electronic version of the report for the complete appendix. The first page is shown here for reference.



Appendix G: Water Balance Model TM – Brelje and Race (August 18, 2015)

Refer to electronic version of the report for the complete appendix. The first page is shown here for reference.

Brelje & Race
CONSULTING ENGINEERS

MEMORANDUM

TO: Town of Windsor - Water Reclamation Facilities Modernization Study Project Team

FROM: Sarah Yardley and Sophia Grubb

SUBJECT: Water Balance Modeling
B&R File No. 3991.01

DATE: August 18, 2015

Brelje & Race was retained by the Town of Windsor to conduct a series of calculations using the Town's water reclamation water balance. This memorandum describes the modifications made to the water balance to assist in the nutrient loading analysis for the Water Reclamation Facilities Modernization Study.

Average Dry Weather Flow (ADWF) input values were developed to reflect the projected populations in the years 2018 and 2021. An estimated population growth rate of 0.9% per year, previously established by the Town, was used. Based on the measured 2014 ADWF of 1.44 MGD, projected values for 2018 and 2021 were 1.49 and 1.53 MGD, respectively.

The irrigation distribution factors were updated to better reflect actual winter time irrigation usage in different weather year conditions. The dry year irrigation usages during the winter months were set at approximately double the average year irrigation usages, which correlated with the recycled water sales data for 2012-2014. It was assumed there would be no winter time irrigation in a wet year. The resultant total annual irrigation use per equivalent acre remained 30 inches for average weather years and became 23.6 and 36.7 inches for dry and wet years, respectively.

Nutrient loading of total phosphorous and total nitrogen in discharge to the creek was added to the water balance. It was assumed that the average nutrient concentrations in the discharge to the creek were 8.0 mg/L total nitrogen and 2.1 mg/L total phosphorous.

After these changes to the water balance were made, the initial configurations were run and recorded. The initial configurations included dry, average, and wet year weather conditions, with the standard storage curve, for both the 2018 and 2021 ADWF. In addition to varying the weather condition, the frost protection option was varied.

Next, new annual rainfall conditions were added to the original set of wet, average, and dry conditions. The rainfall conditions developed included three different mixed weather years and two historical rain data years. The mixed weather years consisted of a wet fall followed by a dry winter, a wet fall followed by an average winter, and a dry fall followed by a wet winter. The historical data utilized were the 2013-14 and the 2014-15 water years' rain data.

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Appendix H: Effluent Storage Cost Estimates

Hazen

Town of Windsor
Modernization Study
Effluent Storage Alternatives

Date: 2/9/2016

#	Alternatives	Storage Added (MG)	Capital Costs ($M)[4]
1	Operational Modifications	8	$0.25
2	New Storage	45	$41
3	Increase Existing (Permit)[2,5]	45	$13.5
4	Outside Agency		NA
5	Geysers (Permit)[1,5]	NA	NA
6b	Geysers/Increase Existing (Permit)[2,3,5]	21	$3.5

Notes:

1 For Alternative 5, current Geysers PS capacity is 1.67 mgd, current Geysers PS flow is 0.53 mgd, increased Geysers PS flow is 0.75 mgd
2 For expansion of existing storage, further study of adequate footprint is required
3 For Alternative 6b, increased Geysers PS flow sized to eliminate need for PS after UV, current Geysers PS capacity is 1.67, increased Geysers PS flow is 0.65 mgd
4 Capital costs do not include land acquisition, environmental mitigation, engineering, construction management, administrative, legal, permitting, operations and maintenar or change order reserves
5 Will require existing NPDES permit to be amended

A proportional increase in storage per mgd in the table below is assumed. Geysers Flow is assumed at 0.53 mgd, unless included as a modification.

Table 8 (From Permit)

	Geysers Diversion (mgd)	
WRP ADWF Rating	**0.53**	**0.75**
	Required Storage (MG)	
1.6	149	149
1.7	149	149
1.8	149	149
1.9	149	149
2	165	149
2.1	165	149
2.2	196	149
2.25	207	165

1 The current Geysers Flow per permit is 0.53 mgd.
2 From Table 8, at ADWF of 2.25 mgd, a 0,1 mgd increase in Geyser flow equates to a 19.1 MG decrease in storage required.
3 From Table 8, at ADWF of 2.25 mgd, a 0.22 mgd increase in Geysers flow (from 0.53 to 0.75 mgd) equates to a 42 MG reduction in storage required (207 - 165 MG).
4 To rerate the WRP to 2.25 mgd, we need to either increase available storage by 58 MG or increase Geysers Flow by a commensurate amount. If we increase Geysers Flow by 0.24 mgd, we can reduce the required additional storage needed to zero. The increased Geysers Flow to rerate the WRP to 2.25 mgd without providing additional storage is 0.77 mgd.
5 Increasing the capacity of Pond 7 alone to provide the required additional storage of 44.4 MG would result in WSE in Pond 7 higher than the WSE at the UV Channel. This would require a new pump station after the UV Channel (assuming berms are simply raised and the footprint is not expanded).
6 To eliminate the need for a pump station after the UV Channel, the Geysers Flow would have to be increased to 0.65 mgd. This would result in a WSE in Pond 7 lower than the UV Channel.

Hazen

Town of Windsor
Modernization Study
New Storage Conceptual Costs

New Storage Conceptual Costs

Additional New Storage (MG)	Using $/MG	Using Power Series Approximation	Using Reference Project
44.5	$38,668,000	$27,832,000	$38,668,000
Estimate (2015 dollars) using most conservative method ($/M	$39,000,000		
Inflated to 2017	$41,000,000		

Notes:

1 Reference project costs are from the Santa Rosa Seasonal Storage Project
2 Power series approximation is based upon a best fit curve of $/MG versus storage
3 The $/MG cost is based upon the $/MG cost of the storage pond closest in size from the Santa Rosa Seasonal Storage Project
4 Environmental costs are not included
5 Land Acquisition costs are not included
6 Operational and Maintenance costs are not included
7 Engineering, Permitting, CM, Administrative, Legal and other program costs are not included
8 0.5 miles of conveyance pipeline is included for new storage; however, no additional offsite work is assumed
9 Native fill is assumed for majority of new storage fill
10 A 50% contingency is included on all estimates
11 All costs are in 2017 dollars

Hazen

Town of Windsor
Modernization Study
Increase Existing Storage

Expand Existing Storage at Pond 7

Expand Existing (MG)	See below
44.5	$12,900,000
Total	**$12,900,000**

Geometric Calculations

	Pond 7	Needed Storage (MG)
Existing WSE	102	44.4
New WSE	109.3	Additional Storage (MG)
Perimeter (ft)	3609	44.5
Pond Surface Area (sf)	798000	
Berm Surface Area (sf)	220760	

Berm Calculations

	Pond 7
Additional Height (ft)	7.3
Slope	2.5h to 1v on water side, 2h to 1v on back side
Width at Crown (ft)	15
Width at Base (ft)	47.85

Fill required (cy)	Assuming a trapezoidal cross-section along entire length of perimeter
30670	

	$/cy	Total	Notes
Cost per cy (Assume imported at $25/cy)	$60.00	$1,840,200	Includes imported backfill and grading of embankment
Include allowance for site prep	34%	$625,668	Demolition, clear and grub, dewatering/draining pond, etc
Include allowance for foundation, drainage/erosion control	18%	$331,236	Trench Keying, over excavation, liner, drainage piping, slope stabilization
Include allowance for mechanical (PS, Pipe, etc)	67%	$1,232,934	Inbound/outbound pump station, fill/drain pipe onsite
Include allowance for site improvements	11%	$202,422	Fencing, access roads, etc
Pump Station at UV Channel and modifications	Y	$1,500,000	Assumes CIP vault, 7.2 MGD pump, can tie into existing UV header, vertical turbine pumps 75 hp each with VFDs + extra for electrical for I&C/dewatering/site work/pipe rerouting
Include GC	20%	$1,146,492	
Include OH&P	21%	$1,444,579.92	
Include Bonding	3%	$249,705.96	
Include contingency	50%	$4,286,618.94	
Total	**$12,900,000**		
Inflate to 2017 dollars	**$13,500,000**		

Notes:
1 Percentages are based upon costs from Santa Rosa Seasonal Storage Project
2 Environmental costs are not included
3 Land Acquisition costs are not included
4 Operational and Maintenance costs are not included
5 Engineering, Permitting, CM, Administrative, Legal and other program costs are not included
6 0.5 miles of conveyance pipeline is included for new storage; however, no additional offsite work is assumed
7 Imported fill is assumed for majority of fill for expanded storage, native fill is assumed for majority of new storage fill
8 A 50% contingency is included on all estimates
9 All costs are in 2015 dollars except when inflated (below the total)

Hazen

Town of Windsor
Modernization Study
Combination of Increase Geysers and Increase Existing Storage

Expand Existing Storage at Pond 7

Expand Existing (MG)	See below
21.3	$3,400,000
Total	**$3,400,000**

Geometric Calculations

Pond No 7	Pond 7	Needed Storage (MG)
Existing WSE	102	44
New WSE	105.5	Additional Storage from ponds (MG)
Perimeter (ft)	3609	21.3
Pond Surface Area (sf)	798000	
Berm Surface Area (sf)	134030	

Berm Calculations

Berm	Pond 7
Height (ft)	3.5
Slope	2.5h to 1v on water side, 2h to 1v on back side
Width at Crown (ft)	15
Width at Base (ft)	30.75

Fill required (cy)	Assuming a trapezoidal cross-section along
10710	entire length of perimeter

	$/cy	Total	Notes
Cost per cy (Assume imported at $25/cy)	$60.00	$642,600	Includes imported backfill and grading of embankment
Include allowance for site prep	34%	$218,484	Demolition, clear and grub, dewatering/draining pond, etc
Include allowance for foundation, drainage/erosion control	18%	$115,668	Trench Keying, over excavation, liner, drainage piping, slope stabilization
Include allowance for mechanical (PS, Pipe, etc)	67%	$430,542	Inbound/outbound pump station, fill/drain pipe onsite
Include allowance for site improvements	11%	$70,686	Fencing, access roads, etc
Include GC	20%	$295,596	
Include OH&P	21%	$372,451	
Include Bonding	3%	$64,380.81	
Include contingency	50%	$1,105,203.88	
Total	$3,400,000		
Inflate to 2017 dollars	$3,500,000		

Notes:
1 Percentages are based upon costs from Santa Rosa Seasonal Storage Project
2 Environmental costs are not included
3 Land Acquisition costs are not included
4 Operational and Maintenance costs are not included
5 Engineering, Permitting, CM, Administrative, Legal and other program costs are not included
6 No conveyance pipeline work is provided for expanding existing storage
7 Imported fill is assumed for majority of fill for expanded storage, native fill is assumed for majority of new storag fill
8 A 50% contingency is included on all estimates
9 All costs are in 2015 dollars except when inflated (below the total)

Appendix I: Grit and Fine Screen Cost Estimates

Hazen

Town of Windsor
Modernization Study
Headworks Alternatives Net Present Worth Evaluation

Date: 11/13/2015

#	Alternatives	Capital Costs	Annual O&M	20-year Net Present Value
			Subtotal O&M	
1	Vortex Grit Chamber w/Fine Screen and Washer	$1,500,000	$10,700	$1,659,000
2	Eutek Headcell w/Fine Screen and Washer (Reuse Flume Channel, New Screen Channel, New Grit Chamber)	$2,500,000	$10,700	$2,659,000
3	Eutek Headcell w/Fine Screen and Washer (New Flume Channel, New Screen Channel, New Grit Chamber)	$3,200,000	$10,700	$3,359,000
4	Eutek Headcell w/Fine Screen and Washer (Reuse Flume Channel, Reuse Screen Channel, New Grit Chamber)	$2,300,000	$10,700	$2,459,000

Notes:

1 All Costs are in 2015 Dollars (Escalation is to be determined & currently not included in this analysis)
2 Discount Rate is released annually by Federal Energy Management Program (FEMP) for federal energy and water conservation projects. Based on 2014 Report cited below, a discount rate of 3.0% was used. http://www1.eere.energy.gov/femp/pdfs/ashb14.pdf
3 Present Worth Factor (P/A, 3%, 20) = 14.877
4 Electricity cost is assumed at, $0.11/kW-hr per US Energy Information Administration.
5 Wastewater Operator rate is based upon latest published salaries per TransparentCalifornia, an average salary of $140400 was used.
6 Senior Wastewater Operator rate is based upon latest published salaries per TransparentCalifornia, an average salary of $168480 was used.
7 Bypass Pumping is provided for work requiring shut down of any of the screening channel, flume channel or grit chamber
8 No dewatering is assumed to be required for excavations
9 No hazardous or contaminated soil handling or disposal is provided
10 No piles are assumed required for construction of any structures

Hazen

Town of Windsor
Modernization Study
Alternative 1
Headworks Alternatives Net Present Worth Evaluation
Vortex Grit Chamber w/Fine Screen and Washer

Date: November 13, 2015

Item	Description			
1	*General Conditions (allowed factor at this time)*	*15.0%*	*$*	*113,973*
2	*Vortex Grit Chamber w/Fine Screen and Washer*		*$*	*759,818*
		Subtotal:	**$**	**873,791**
	Contractor Overhead	*10.0%*	*$*	*87,379*
		Subtotal:	**$**	**961,170**
	Contractor Profit	*10.0%*	*$*	*96,117*
		Subtotal:	**$**	**1,057,287**
	Escalation at 3% annually	*4.8%*	*$*	*50,659*
		Subtotal:	**$**	**1,107,945**
	Bond and Insurance	*3.0%*	*$*	*33,238*
		Subtotal:	**$**	**1,141,184**
	Design Contingency	*30.0%*	*$*	*342,355*
		Total:	**$**	**1,500,000**

Alternative 1

Vortex Grit Chamber w/Fine Screen and Washer

Operational and Maintenance Costs

Electricity

Equipment	hp	Number in Operation	Total kW	Days in Service each Year	Power Cost ($/kW-hr)	Annual Electricity Cost
Screen	3	1	2.24	365	$0.11	$87
Screenings Washer	5	1	3.73	365	$0.11	$145
Grit Pump	7.5	1	5.59	365	$0.11	$217
Vortex Grit	1	1	0.75	365	$0.11	$29
Classifier	1	1	0.75	365	$0.11	$29
					Subtotal Electricity Costs:	**$506**

Labor

	Annual Supervisor Man-Hours	Supervisor Wage ($/mh)	Annual Operator Man-Hours	Operator Wage ($/mh)	Annual Labor Cost
Operator Labor	26	$90	104	$75	$10,140
				Subtotal Labor Costs:	**$10,140**

Total Annual O&M Costs: **$10,700**

Hazen

Town of Windsor
Modernization Study
Alternative 2
Headworks Alternatives Net Present Worth Evaluation
Eutek Headcell w/Fine Screen and Washer (Reuse Flume Channel, New Screen Channel, New Grit Chamber)

Date: November 13, 2015

Item	Description			
1	*General Conditions (allowed factor at this time)*	*15.0%*	*$*	*192,031*
2	*Eutek Headcell w/Fine Screen and Washer (Reuse Flume Channel, New Screen Channel, New Grit Chamber)*		*$*	*1,280,207*
		Subtotal:	**$**	**1,472,238**
	Contractor Overhead	*10.0%*	*$*	*147,224*
		Subtotal:	**$**	**1,619,462**
	Contractor Profit	*10.0%*	*$*	*161,946*
		Subtotal:	**$**	**1,781,408**
	Escalation at 3% annually	*4.8%*	*$*	*85,354*
		Subtotal:	**$**	**1,866,762**
	Bond and Insurance	*3.0%*	*$*	*56,003*
		Subtotal:	**$**	**1,922,765**
	Design Contingency	*30.0%*	*$*	*576,830*
		Total:	**$**	**2,500,000**

Alternative 2

Eutek Headcell w/Fine Screen and Washer (Reuse Flume Channel, New Screen Channel, New Grit Chamber)

Operational and Maintenance Costs

Electricity

Equipment	hp	Number in Operation	Total kW	Days in Service each Year	Power Cost ($/kW-hr)	Annual Electricity Cost
SpiraSnail	1	1	0.75	365	$0.11	$29
Grit Pump	7.5	1	5.59	365	$0.11	$217
Screen	3	1	2.24	365	$0.11	$87
Screenings Washer	5	1	3.73	365	$0.11	$145
					Subtotal Electricity Costs:	**$477**

Labor

	Annual Supervisor Man-Hours	Supervisor Wage ($/mh)	Annual Operator Man-Hours	Operator Wage ($/mh)	Annual Labor Cost
Operator Labor	26	$90	104	$75	$10,140
				Subtotal Labor Costs:	**$10,140**

Total Annual O&M Costs: **$10,700**

Hazen

Town of Windsor
Modernization Study
Alternative 3
Headworks Alternatives Net Present Worth Evaluation
Eutek Headcell w/Fine Screen and Washer (New Flume Channel, New Screen Channel, New Grit Chamber)

Date: November 13, 2015

Item	Description			
1	*General Conditions (allowed factor at this time)*	*15.0%*	*$*	*243,557*
2	*Eutek Headcell w/Fine Screen and Washer (New Flume Channel, New Screen Channel, New Grit Chamber)*		*$*	*1,623,716*
		Subtotal:	**$**	**1,867,273**
	Contractor Overhead	*10.0%*	*$*	*186,727*
		Subtotal:	**$**	**2,054,001**
	Contractor Profit	*10.0%*	*$*	*205,400*
		Subtotal:	**$**	**2,259,401**
	Escalation at 3% annually	*4.8%*	*$*	*108,257*
		Subtotal:	**$**	**2,367,658**
	Bond and Insurance	*3.0%*	*$*	*71,030*
		Subtotal:	**$**	**2,438,687**
	Design Contingency	*30.0%*	*$*	*731,606*
		Total:	**$**	**3,200,000**

Alternative 3

Eutek Headcell w/Fine Screen and Washer (New Flume Channel, New Screen Channel, New Grit Chamber)

Operational and Maintenance Costs

Electricity

	hp	Number in Operation	Total kW	Days in Service each Year	Power Cost ($/kW-hr)	Annual Electricity Cost
SpiraSnail	1	1	0.75	365	$0.11	$29
Grit Pump	7.5	1	5.59	365	$0.11	$217
Screen	3	1	2.24	365	$0.11	$87
Screenings Washer	5	1	3.73	365	$0.11	$145
					Subtotal Electricity Costs:	**$477**

Labor

	Annual Supervisor Man-Hours	Supervisor Wage ($/mh)	Annual Operator Man-Hours	Operator Wage ($/mh)	Annual Labor Cost
Operator Labor	26	$90	104	$75	$10,140
				Subtotal Labor Costs:	**$10,140**

Total Annual O&M Costs: **$10,700**

Hazen

Town of Windsor
Modernization Study
Alternative 4
Headworks Alternatives Net Present Worth Evaluation
Eutek Headcell w/Fine Screen and Washer (Reuse Flume Channel, Reuse Screen Channel, New Grit Chamber)

Date: November 13, 2015

Item	Description			
1	*General Conditions (allowed factor at this time)*	*15.0%*	*$*	*174,400*
2	*Eutek Headcell w/Fine Screen and Washer (Reuse Flume Channel, Reuse Screen Channel, New Grit Chamber)*		*$*	*1,162,667*
		Subtotal:	**$**	**1,337,067**
	Contractor Overhead	*10.0%*	*$*	*133,707*
		Subtotal:	**$**	**1,470,774**
	Contractor Profit	*10.0%*	*$*	*147,077*
		Subtotal:	**$**	**1,617,851**
	Escalation at 3% annually	*4.8%*	*$*	*77,518*
		Subtotal:	**$**	**1,695,369**
	Bond and Insurance	*3.0%*	*$*	*50,861*
		Subtotal:	**$**	**1,746,230**
	Design Contingency	*30.0%*	*$*	*523,869*
		Total:	**$**	**2,300,000**

Alternative 4

Eutek Headcell w/Fine Screen and Washer (Reuse Flume Channel, Reuse Screen Channel, New Grit Chamber)

Operational and Maintenance Costs

Electricity

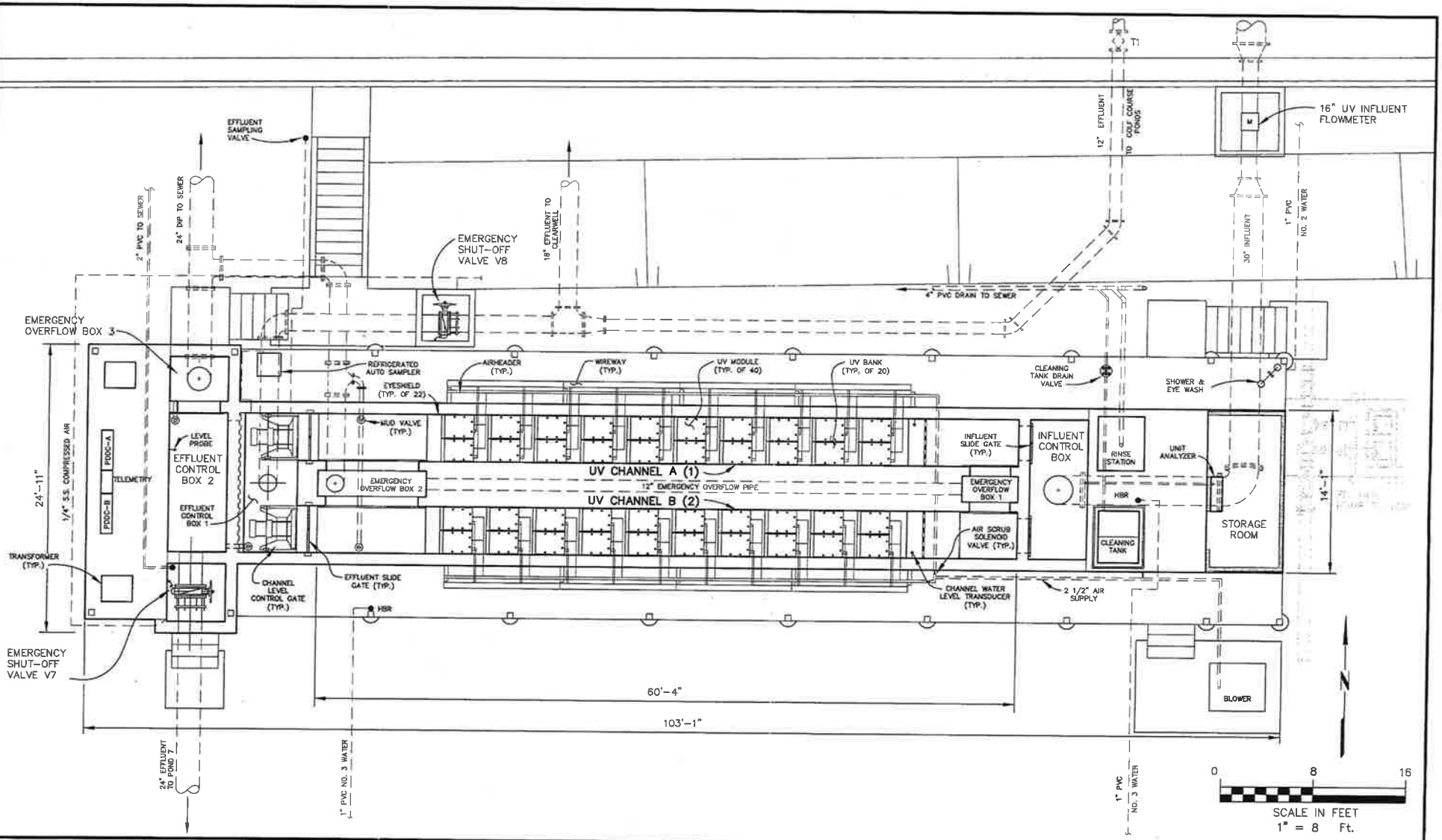
Equipment	hp	Number in Operation	Total kW	Days in Service each Year	Power Cost ($/kW-hr)	Annual Electricity Cost
SpiraSnail	1	1	0.75	365	$0.11	$29
Grit Pump	7.5	1	5.59	365	$0.11	$217
Screen	3	1	2.24	365	$0.11	$87
Screenings Washer	5	1	3.73	365	$0.11	$145
					Subtotal Electricity Costs:	**$477**

Labor

	Annual Supervisor Man-Hours	Supervisor Wage ($/mh)	Annual Operator Man-Hours	Operator Wage ($/mh)	Annual Labor Cost
Operator Labor	26	$90	104	$75	$10,140
				Subtotal Labor Costs:	**$10,140**

Total Annual O&M Costs: **$10,700**

Appendix J: UV Disinfection Additional Information



OPERATION AND MAINTENANCE MANUAL
TOWN OF WINDSOR

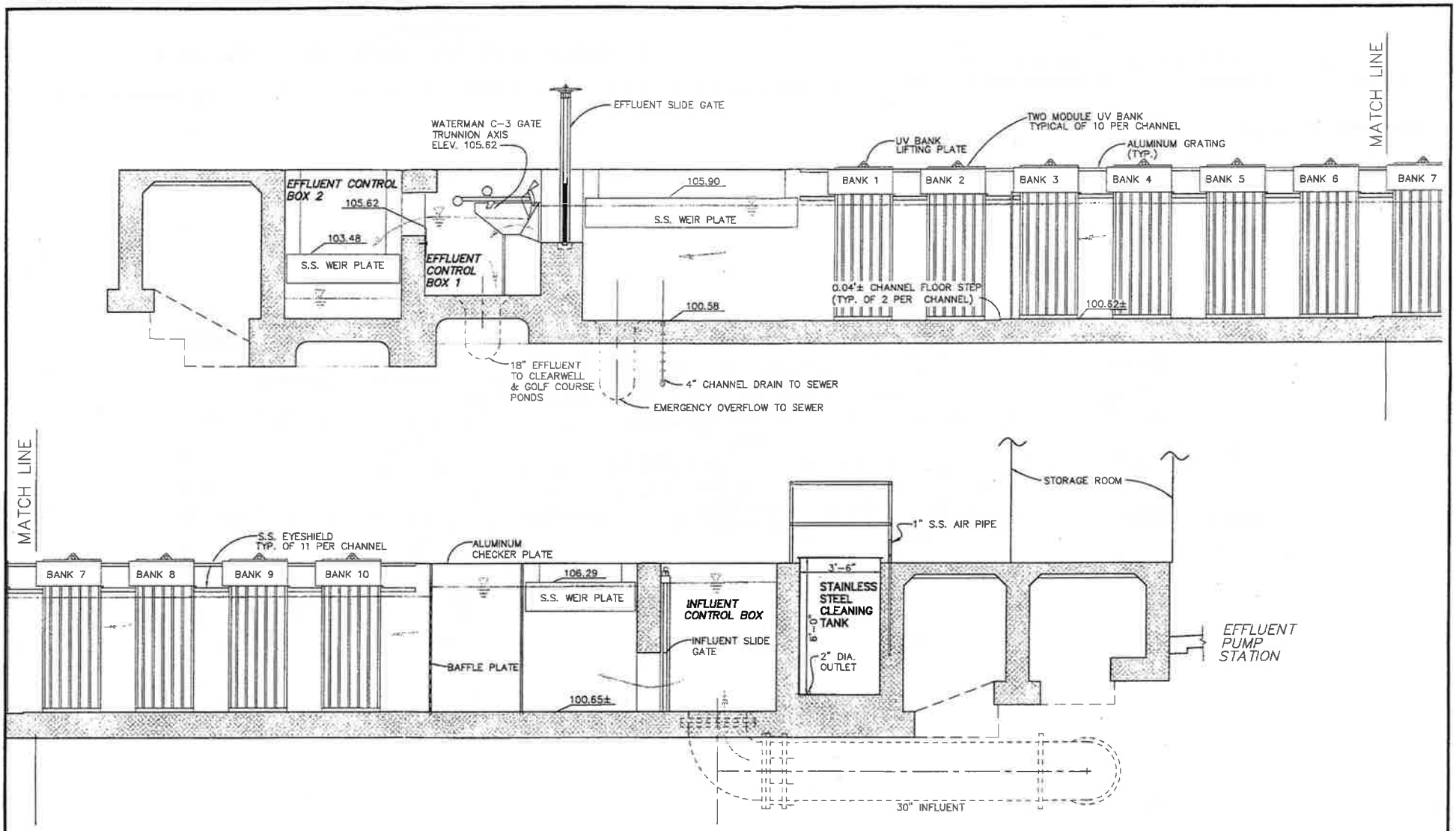
UV DISINFECTION CHANNELS PLAN

BRELJE & RACE

JULY 2002

FIGURE 25-1

07-30-02 \17763\DWG\FIGURE25-1.DWG TAB: 25-1 VIEW: PS1



OPERATION AND MAINTENANCE MANUAL
TOWN OF WINDSOR

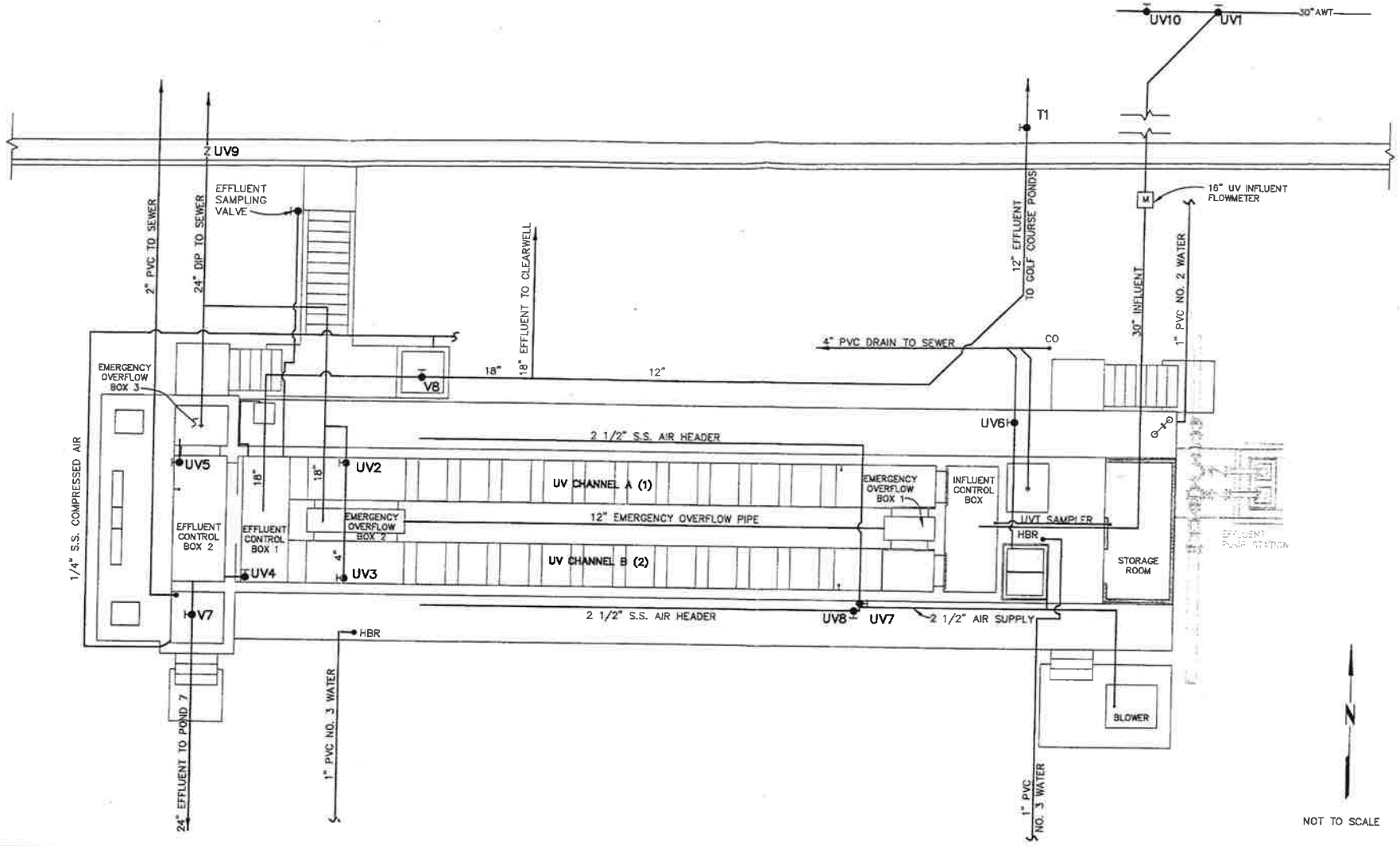
UV DISINFECTION CHANNELS SECTION

JULY 2002

BRELJE & RACE

FIGURE 25-2

07-30-02 \17763\DWG\FIGURE25-2.DWG TAB: 25-2 VIEW: PS1



OPERATION AND MAINTENANCE MANUAL
TOWN OF WINDSOR

UV DISINFECTION CHANNELS
PIPING SCHEMATIC

BRELJE & RACE

JULY 2002

FIGURE 25-3

07-30-02 \17763\DWG\FIGURE25-3.DWG TAB: 25-3 VIEW: PS1

Aquaray® 40 HO Vertical Lamp Ultraviolet Disinfection System



Preliminary Budget Proposal For
Town of Windsor, CA
UV System Replacement

August 24, 2015

OZONIA NORTH AMERICA, LLC
600 WILLOW TREE ROAD
LEONIA, NJ 07605 USA
TEL 201 676-2525 | FAX 201 346-5460

August 24, 2015

Bill Sotirakos, P.Eng.
Principal UV Technologist
Carollo Engineers, Inc.
170 Grandview Ave
Thornhill, ON L3T 1J1

Re: Aquaray® 40 HO (High Output) Vertical Lamp Ultraviolet Disinfection Equipment
Town of Windsor, CA

Ozonia is pleased to submit our proposal for the replacement Aquaray® 40 HO Vertical Lamp ultraviolet disinfection system for the above referenced project.

The Aquaray 40 “HO” system is latest generation and improvement of the previous Aquaray® VLS design which has been in use since 1986 and is currently installed at the City of Windsor WWTP. The Aquaray® 40 “HO” is based on the arrangement of the original Aquaray® 40 VLS “Type-B” design, fits in the existing channels without any modifications and includes the following additional features:

- Reduced Number of Lamps
- Longer Lamp Life
- In-Channel Automatic Wiping System
- Fully automated operation. Only requires a 4-20 mA flow signal
- Allen Bradley CompactLogix PLC with 10” Color Panelview 1000 Plus Interface
- Additional Alarms and Monitoring Features

If you have any questions or require additional information, please don't hesitate to contact the undersigned.

Sincerely,
For OZONIA NORTH AMERICA

Pedro DaCruz
Sales Director

Made in USA

OZONIA NORTH AMERICA, LLC
600 WILLOW TREE ROAD
LEONIA, NJ 07605 USA
TEL 201 676-2525 | FAX 201 346-5460

DEGREMONT TECHNOLOGIES

Degremont Technologies is a world leader in the water and wastewater treatment market and offers a full array of integrated water solutions. The group is composed of several leading equipment companies such as ***Ozonia North America***, Infilco Degremont and Anderson Water Systems and is part of the larger Degremont Group, which employs more than 3,000 people in over 70 countries, serving over 1 billion people with water and wastewater solutions. Degremont is subsidiary of Suez Environnment, a the leading global water and waste services company with sales of over $17 billion.

Degremont Technologies provides solutions in the areas of headworks, biosolids, disinfection, membrane filtration, separations and biofiltration. ***Ozonia North America*** has its headquarters in Leonia, New Jersey and is the disinfection equipment and solutions provider for the group offering a wide range of UV and ozone products. Other companies within the group offer a variety of products with longstanding market names such as the Climber Screen® Mechanical Bar Screen, ABW® Traveling Bridge Filter, and Cannon® Digester Mixing System.

AQUARAY® 40 HO (High Output) SYSTEM DESCRIPTION

The Aquaray 40 “HO” system is latest generation and improvement of the previous Aquaray® VLS design which has been in use since 1986. The Aquaray® 40 “HO” VLS is based on the arrangement of the original Aquaray® 40 VLS “Type-B” design. The vertical lamp orientation and configuration has been proven, through general use and extensive pilot studies, to be a very effective form of disinfection. The system also has many features that make it easy and safe to operate and maintain.

The low pressure, low intensity lamps of the original Aquaray® 40 VLS have been replaced with new low pressure, high output lamps - requiring fewer lamps to treat the same capacity. Fewer lamps guarantee considerable savings on capital, operation, and maintenance costs.

UV DOSAGE ENHANCEMENT:

The ultraviolet dosage is the product of the ultraviolet intensity multiplied by the time (in seconds) that the water is in contact with that UV intensity. Based on completed bioassays, the Aquaray® HO VLS system can treat more than twice the flow compared to the standard low pressure low intensity lamps in the older Aquaray® 40 configuration with the same UV dosage (uWatts-secs/cm^2) requirement. Flow deflection baffles have been added to enhance the disinfection performance capability of the Aquaray® HO VLS system.

HIGH OUTPUT LAMP ARRANGEMENT:



The ultraviolet lamps are mounted vertically so that all electrical connections are made out of the water and within the protection of a NEMA 4X stainless steel enclosure. Unlike other designs, all the lamps are easily accessed through the lid of this enclosure. Therefore, routine service such as lamp changes can be made without having to remove the lamp modules from the channel.

The lamps are also mounted in a uniform staggered array, three inch on center across the channel and five inch on center along the channel. This ensures a semi-tortuous path so that every particle of water will come into intimate contact with the most intense point of lamp output.

MODULE ARRANGEMENT:

The number and layout of the modules within the channel is determined based on the required UV dosage and a UV path for the water that eliminates any possibility of hydraulic short-circuiting.

See “DESIGN BRIEF” for details of module arrangement for this project.

CONTROL AND MONITORING:

Electronic lamp control is utilized to minimize power consumption. Electronic lamp control assemblies are conveniently mounted in the Aquaray® High Output Module’s NEMA-4X enclosure. This locates the assemblies close to the high output lamps, which minimizes the effect of outside interference such as radio waves, lightning, and voltage spikes.

With our Aquaray® High Output Module each individual lamp is monitored through the use of an on-board computer called a Data Controller Assembly (DCA). The DCA gathers and stores information relative to individual lamp hours and cycles. A non-volatile memory is included so that a possible relocation of the module will not result in a loss or misdirection of valuable lamp data.

The benefits of recording the individual lamp history may not be immediately apparent. UV lamps are guaranteed to provide a minimum operating life measured in terms of active operating hours, usually up to 13,000 hours. If a lamp fails electrically before the guarantee, our end-of-lamp life conditional warranty provides for a replacement at a cost pro-rated to the actual use achieved with the original lamp. For example, if a lamp fails at mid-life the replacement will be provided at half price.



A Power Distribution and Data Center (PDDC) included which houses the load center enclosure and GFCI Breakers for each high output module. The PDDC also includes an Allen Bradley CompactLogix PLC and Panelview 1000 Plus Operator Interface. Each Aquaray ® High Output module in the UV disinfection channel receives power from the load center locally mounted at the PDDC via a single power cable with waterproof plug-in connectors.

Each Aquaray® High Output module is fully independent and capable of automatic, fail safe operation in case of a control fault. This “default on” design ensures continuous disinfection even under emergency conditions.

FLOW PACING:

Flow Pacing is a system whereby lamp rows are switched on and off in relation to plant flow variations. The Aquaray® 40 HO System provides for very fine adjustments of the number of High Output lamps in service. Adjustments are made in direct proportion to the flow, with switching increments as low as 3%. To take full advantage of this feature we take a control signal, usually from the plant flow meter, and switch the lamps on or off as the flow changes.

The advantage of being able to switch the lamps by row is two fold:

- Energy Conservation
- Lamp Conservation

In our system each lamp requires 165 Watts. You realize immediate savings by activating only the minimum number of lamps required.

SYSTEM CLEANING:

Any UV system gradually accumulates a coating on the quartz sleeves housing the lamps. This routine fouling must be removed periodically. The Aquaray® 40 HO VLS offers a fully automatic, in-channel cleaning system which reduces maintenance. The automatic wiping system is to be operated once daily and the wipers are to be replaced once every two years. This system is included in our proposal.

SERVICE:

Every piece of equipment within a wastewater plant requires service. The Aquaray® 40 HO VLS has been developed to permit easy troubleshooting and quick replacement of components. The majority of maintenance activities can be carried out while the equipment is still located within the channel. The recommended spares included in this proposal will ensure that the system can be maintained efficiently and brought back to full operation in the shortest possible time.





DESIGN BRIEF

PLANT INFORMATION AND DESIGN BASIS;

Plant Name ..... City of Windsor WWTP
Plant Location ..... City of Windsor, CA
Peak Flow, MGD ..... 8.6 MGD
Minimum Flow, MGD ..... 2 MGD
Average UVT, %……. 75%
Minimum UVT, %……. 67%
Average TSS, mg/L..... 1.8 mg/L
Required Reuse Effluent, MPN/100ml:
Total Coliform ..... < 2.2 (7-day average)

Minimum UV Dose ..... 100,000 uWatts-secs/cm^2 (Title 22 Reuse)

SUMMARY:

Based on the information in the design table below, the system proposed will provide a minimum UV dosage of 100,000 uWatts-secs/cm^2. The dosage calculation takes into account several factors including the end of lamp life, the quartz sleeve transmittance factor, and the peak capacity.

At a peak flow of 8.6 MGD and a minimum UV transmittance of 67%, sixteen (16) Aquaray® 40 HO UV modules per modules are proposed. Each of the UV channels will include UV modules mounted two (2) across by eight (8) banks in series (7 duty + 1 redundant). The UV system will deliver a minimum UV dose of 100 mJ/cm2 at the peak flow with one (1) bank out of service per the 2012 NWRI guidelines

Each Aquaray® 40 HO module includes 40 Low Pressure High Output Lamps, arranged in five rows of eight lamps each.

PROPOSED AQUARAY® 40 HO VERTICAL LAMP SYSTEM DESIGN:

Peak Flow, MGD	8.6 MGD
% UV Transmission	67% minimum 75% average
Calculated Dosage at Peak Flow, μwatt-secs/cm^2	>100,000
Number of Channels	2
Number of Modules Across (Modules per Bank)	2
Number of Modules in Series (Number of Banks)	8 (7 service + 1 redundant)
Channel Width, in.	existing
Channel Length, ft.	existing
Channel Depth, in.	existing
Nominal Water Depth, in.	57.5 to 62 inches
Aquaray® Modules in Channel	16
Total Number of Modules	32
Number of Lamps/Module	40
Total Number of Lamps	1,280
End of Lamp Life factor, F_p	0.9
Fouling Factor, F_t	0.8
Headloss at Peak Flow (8.6 MGD) across the modules, in.	1.27 inches
Power Consumption per Lamp, W	165
Power Consumption at Average Flow, kW	58.1 kW @ 2 MGD and 75% UVT
Power Consumption at Peak (8.6 MGD), kW	192.6 kW @ 8.6 MGD and 67% UVT
Total Installed Power, kW	211.2 kW
Power Requirements	480V/3ph/60Hz

SCOPE OF SUPPLY AND BUDGET PRICE

We propose to furnish the following equipment the Aquaray® 40 HO Vertical Lamp ultraviolet disinfection system described in the previous sections

- Aquaray® 40 HO Vertical lamps with Cleaning Wipers, 316L stainless steel components
- Mounting Rail/Eye Shields, 304 stainless steel
- Power Distribution and Data Control Center(s) (PDDC) – Includes Allen Bradley CompactLogix PLC and Panelview 1000 Plus Operator Interface.
- Wireways
- Stepdown Transformers
- Interconnecting Cables between the Modules and the Data Control Station(s) and between the Modules and Power Distribution Center(s)
- Lamp Row by Row Flow Pacing
- In-Channel Cleaning System (automatic cleaning wipers)
- Lifting Spreader Bar
- Anchor Bolts
- Recommended Spare Parts
- Online UVT Analyzer

The following will also be included:

- Freight to the jobsite
- Start-up service: twenty (20) days in six (6) trips
- Four (4) O&M manuals

Note that the following items are to be provided by others (unless indicated otherwise above):

- Level Control Device (reuse existing)
- Cleaning Tank (reuse existing)
- Channel Grating
- Slide Gates
- Remote Computer System (SCADA) and Integration to SCADA
- Installation
- Embedded Conduits
- Point Check Bioassay including Sampling and Effluent Performance Testing

BUDGET PRICE: Our current budget estimating price, not including any optional adders above is:

Nine Hundred and Sixty Thousand Dollars ($960,000)

This price will be valid for one (1) year; payment terms will be as below and commercial terms and conditions are given on the following page. The price is in accordance with the Scope of Supply and terms of this proposal and any changes may require the price to be adjusted.

Payment Terms:

10% Net Cash, Payable in thirty (30) days from date of submittal of initial drawings for approval;
80% Net Cash, Payable in progress payments thirty (30) days from dates of respective shipments of the Products;
10% Net Cash, Payable in thirty (30) days from Product installation and acceptance or Ninety (90) days after date of final Product delivery, whichever occurs first.

SCHEDULE: Approval drawings and data can be submitted approximately 6-8 weeks after agreement to all terms, as evidenced by OZONIA's receipt of this proposal, fully executed; or, in the event that Purchaser issues a Purchase Order, OZONIA's receipt of fully executed letter agreement. OZONIA estimates that shipment of the Products can be made in approximately 18-20 weeks after OZONIA has received from Purchaser final approval of all submittal drawings and data.

APPENDIX B

Project: Town of Windsor, CA

Description	
Equipment Cost	
UV Disinfection System (Complete)	960000
Total Capital Costs:	**$ 960,000.00**
Operation and Maintenance Cost Information	
Power Consumption	
Power Consumption (kW) at Average Conditions[(1)]	60.5
Lamp and Ballast Power Consumption at Full Power, all Lamps (kW)	0.165
Ancillary Power Consumption per Power Distribution Center (PDC) (Power Supplies, Air Conditioners, Fans, etc.) (kW)	0.007
Total Number of PDC's	2
Power Consumption of System Control Center (SCC) (PLC, HMI, Power Supplies, Air Conditioner, Fans, etc.) (kW)	included
Maximum Power Consumption (kW)	192.6
Lamp Replacement	
Total Number of Lamps in System	1280
Number of Lamps Operating at Average Conditions[(1)]	352
Guaranteed Lamp Life (hours)	12000
Lamp Replacement Cost	$30
Ballast Replacement	
Total Number of Ballasts in System	640
Guaranteed Ballast Life (years)	5
Ballast Replacement Cost	$200
Quartz Sleeve Replacement	
Total number of Quartz Sleeves in System	1280
Guaranteed Quartz Sleeve Life (years)	5
Quartz Sleeve Replacement Cost	$30
UV Intensity Sensor Replacement	
Total Number of UV Intensity Sensors in System	16
Guaranteed UV Sensor Life (years)	5
UV Intensity Sensor Replacement Cost	$950
Annual Cost for Duty and Reference Sensors Calibration	N/A
Cleaning System - Wiper Replacement Cost	
Total Number of Automatic Cleaning Wipers in System	1280
Guaranteed Wiper Life (years)	2
Wiper Replacement Cost	$8
Annual Cost for Automatic Cleaning System Consumables (Cleaning Solution, etc.)	$250
Notes: (1) Average Conditions is defined as a flow rate of 2 MGD and a UV transmittance of 75%. Use the design End of Lamp Life and Fouling factors in your calculations.	

ETS-UV by Neptune Benson
238 Commercial Drive
PO Box 392
Beaver Dam, WI 53916
Phone: 877-885-4628
Fax: 920-885-4386

BUDGETARY QUOTE

Quote No: 15-08-PB13
Version: 1
Date: August 25, 2015

Company:	Carollo Engineers
Attention:	Bill Sotirakos
From:	Patrick Bollman, P.E. Applications Engineering Manager/Regional Sales Manager
Project:	Windsor, CA

DESIGN PARAMETERS -

Flowrate:	2 MGD	Average
	8.6 MGD	Peak
Transmittance:	75%	Average
	67%	Min
Dose:	100	mJ/cm2
Redundancy:	1	Reactor

EQUIPMENT SELECTION & DESIGN -

Model:	UVLW-45800-30	
Quantity:	7	
Configuration:	Parallel	
Connections:	20	" ANSI Flanges
Each Chamber/Train Treats:	1.44	MGD @ 67% transmittance
Lamp Type:	800 W	Low Pressure High Output Amalgam
Number of Lamps per Chamber:	45	
Lamp Configuration:	Horizontal and Parallel to Flow	

Chamber Features:	***Power & Control Features:***
Automatic Wiper	Epoxy Coated Steel Cabinets
UV Monitor	Free Standing Cabinets
Temperature Sensor	NEMA12 Rating
Drain Port	Lamp Power Level Control
Access Hatch	480V, 3-PHASE, 60 Hz Power Supply

PRICE & TERMS -

Budget Price:	$	1,905,800.00	(includes online transmittance monitor and freight to site)
Options:			
Spare Parts	$	47,775.00	(35 lamps, sleeves and seals, wiper rings, ballasts)
Startup and Training	$	17,100.00	(3 trips/15 days total on site)

Terms:

- Quote valid for 60 days
- ETS standard terms and conditions apply
- Delivery approximately 12 to 18 weeks after approved submittals

ETS UV Technology by Neptune Benson®

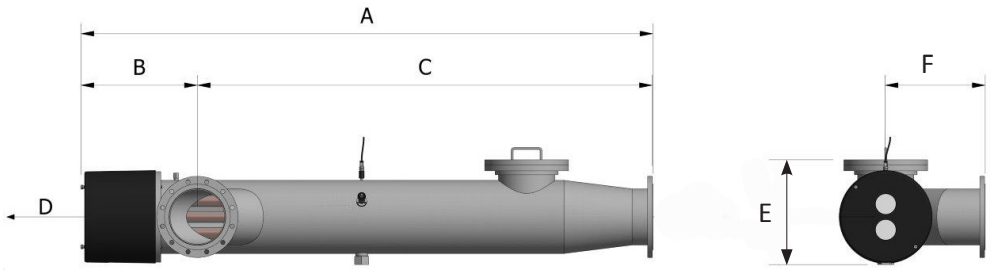
SPECIFICATION SHEET
UVLW RANGE





The UVLW is a range of 800W low pressure, high output amalgam UV systems that are validated to the 2003 and 2012 NWRI Reuse Guidelines

Model	Connection (Inches)	# of Lamps (800W)	Dimensions						Panel Dimensions		
			A	B	C	D	E	F	W	H	D
ULVW-6800-10	8	6	102	83	74	75	15	10	32	79	24
UVLW-6800-14	10	6	109	87	74	75	17	12	32	79	24
UVLW-8800-14	10	8	109	87	74	75	17	12	32	79	24
UVLW-16800-20	16	16	118	95	74	75	24	15	60	79	24
UVLW-20800-20	16	20	118	95	74	75	24	15	79	79	24
UVLW-22800-24	20	22	119	93	74	75	29	18	79	79	24
UVLW-30800-24	20	30	119	93	74	75	29	18	79	79	24
UVLW-30800-30	20	30	128	100	74	75	31	21	79	79	24
UVLW-45800-30	20	45	128	100	74	75	31	21	103	79	24



CHAMBER
316L SS
ANSI 150# flanged connections
Install inline, horizontally or vertically
Features:
Acess Hatch
Twist lock lamp connections
Dry UV intensity monitors
High purity quartz sleeves
Low voltage automatic wiper
One piece wiper ring
Temperature sensor
Drain and vent ports

CONTROL SYSTEM
NEMA 12 epoxy coated mild steel enclosure
Operational 32-113°F, RH < 90%
Features:
Panelview Plus 1000 HMI
Allen Bradley Compact Logix PLC
MODBUS over Ethernet communication
Multiple warning and alarms
Variable power lamps
480V/3-phase

SYSTEM OPTIONS
304 or 316 NEMA 4X enclosures
Effluent flange location
Skid mounted
Containerized
Internal/external polish or electropolish

INSTALLATION NOTES
Provide necessary maintenance space
Install in a dry area
Provide floor drain or sump
Lamps submerged at all times
Minimum of two conduits required
Chamber must be grounded

APPENDIX C

Project: Town of Windsor, CA

Description	
Equipment Cost	
UV Disinfection System (Complete)	1,970,000
Total Capital Costs:	**$ 1,970,000**
Operation and Maintenance Cost Information	
Power Consumption	
Power Consumption (kW) at Average Conditions[(1)]	32
Lamp and Ballast Power Consumption at Full Power, all Lamps (kW)	36
Ancillary Power Consumption per Power Distribution Center (PDC) (Power Supplies, Air Conditioners, Fans, etc.) (kW)	5
Total Number of PDC's	3
Power Consumption of System Control Center (SCC) (PLC, HMI, Power Supplies, Air Conditioner, Fans, etc.) (kW)	-
Maximum Power Consumption (kW)	41
Lamp Replacement	
Total Number of Lamps in System	45
Number of Lamps Operating at Average Conditions[(1)]	45
Guaranteed Lamp Life (hours)	12000
Lamp Replacement Cost	$440
Ballast Replacement	
Total Number of Ballasts in System	45
Guaranteed Ballast Life (years)	5
Ballast Replacement Cost	$490
Quartz Sleeve Replacement	
Total number of Quartz Sleeves in System	45
Guaranteed Quartz Sleeve Life (years)	10
Quartz Sleeve Replacement Cost	$360
UV Intensity Sensor Replacement	
Total Number of UV Intensity Sensors in System	1
Guaranteed UV Sensor Life (years)	3
UV Intensity Sensor Replacement Cost	$850
Annual Cost for Duty and Reference Sensors Calibration	$600
Cleaning System - Wiper Replacement Cost	
Total Number of Automatic Cleaning Wipers in System	45
Guaranteed Wiper Life (years)	2
Wiper Replacement Cost	$25
Annual Cost for Automatic Cleaning System Consumables (Cleaning Solution, etc.)	$100
Notes: (1) Average Conditions is defined as a flow rate of 2 MGD and a UV transmittance of 75%. Use the design End of Lamp Life and Fouling factors in your calculations.	

ETS-UV by Neptune Benson
238 Commercial Drive
PO Box 392
Beaver Dam, WI 53916
Phone: 877-885-4628
Fax: 920-885-4386

BUDGETARY QUOTE

Quote No: 15-08-PB13
Version: 1
Date: August 25, 2015

Company:	Carollo Engineers
Attention:	Bill Sotirakos
From:	Patrick Bollman, P.E. Applications Engineering Manager/Regional Sales Manager
Project:	Windsor, CA

DESIGN PARAMETERS -

Flowrate:	2 MGD	Average
	8.6 MGD	Peak
Transmittance:	75%	Average
	67%	Min
Dose:	100	mJ/cm2
Redundancy:	1	Reactor

EQUIPMENT SELECTION & DESIGN -

Model:	UVLW-45800-30	
Quantity:	7	
Configuration:	Parallel	
Connections:	20	" ANSI Flanges
Each Chamber/Train Treats:	1.44	MGD @ 67% transmittance
Lamp Type:	800 W	Low Pressure High Output Amalgam
Number of Lamps per Chamber:	45	
Lamp Configuration:	Horizontal and Parallel to Flow	

Chamber Features:	***Power & Control Features:***
Automatic Wiper	Epoxy Coated Steel Cabinets
UV Monitor	Free Standing Cabinets
Temperature Sensor	NEMA12 Rating
Drain Port	Lamp Power Level Control
Access Hatch	480V, 3-PHASE, 60 Hz Power Supply

PRICE & TERMS -

Budget Price:	$	1,905,800.00	(includes online transmittance monitor and freight to site)
Options:			
Spare Parts	$	47,775.00	(35 lamps, sleeves and seals, wiper rings, ballasts)
Startup and Training	$	17,100.00	(3 trips/15 days total on site)

Terms:

- Quote valid for 60 days
- ETS standard terms and conditions apply
- Delivery approximately 12 to 18 weeks after approved submittals

ETS UV Technology by Neptune Benson®

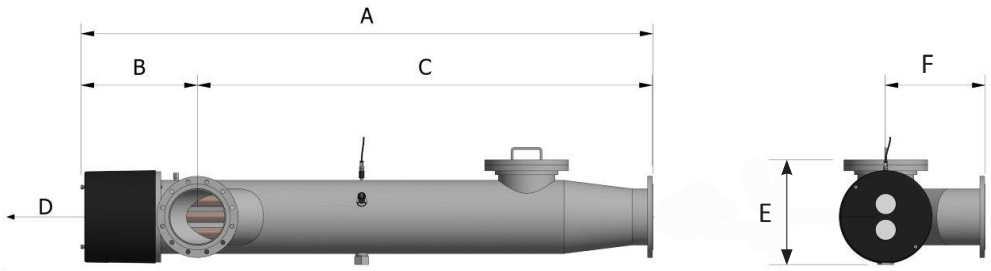
SPECIFICATION SHEET
UVLW RANGE





The UVLW is a range of 800W low pressure, high output amalgam UV systems that are validated to the 2003 and 2012 NWRI Reuse Guidelines

Model	Connection (Inches)	# of Lamps (800W)	Dimensions						Panel Dimensions		
			A	B	C	D	E	F	W	H	D
ULVW-6800-10	8	6	102	83	74	75	15	10	32	79	24
UVLW-6800-14	10	6	109	87	74	75	17	12	32	79	24
UVLW-8800-14	10	8	109	87	74	75	17	12	32	79	24
UVLW-16800-20	16	16	118	95	74	75	24	15	60	79	24
UVLW-20800-20	16	20	118	95	74	75	24	15	79	79	24
UVLW-22800-24	20	22	119	93	74	75	29	18	79	79	24
UVLW-30800-24	20	30	119	93	74	75	29	18	79	79	24
UVLW-30800-30	20	30	128	100	74	75	31	21	79	79	24
UVLW-45800-30	20	45	128	100	74	75	31	21	103	79	24



CHAMBER
316L SS
ANSI 150# flanged connections
Install inline, horizontally or vertically
Features:
Acess Hatch
Twist lock lamp connections
Dry UV intensity monitors
High purity quartz sleeves
Low voltage automatic wiper
One piece wiper ring
Temperature sensor
Drain and vent ports

CONTROL SYSTEM
NEMA 12 epoxy coated mild steel enclosure
Operational 32-113°F, RH < 90%
Features:
Panelview Plus 1000 HMI
Allen Bradley Compact Logix PLC
MODBUS over Ethernet communication
Multiple warning and alarms
Variable power lamps
480V/3-phase

SYSTEM OPTIONS
304 or 316 NEMA 4X enclosures
Effluent flange location
Skid mounted
Containerized
Internal/external polish or electropolish

INSTALLATION NOTES
Provide necessary maintenance space
Install in a dry area
Provide floor drain or sump
Lamps submerged at all times
Minimum of two conduits required
Chamber must be grounded

APPENDIX C

Project: Town of Windsor, CA

Description	
Equipment Cost	
UV Disinfection System (Complete)	1,970,000
Total Capital Costs:	**$ 1,970,000**
Operation and Maintenance Cost Information	
Power Consumption	
Power Consumption (kW) at Average Conditions[(1)]	32
Lamp and Ballast Power Consumption at Full Power, all Lamps (kW)	36
Ancillary Power Consumption per Power Distribution Center (PDC) (Power Supplies, Air Conditioners, Fans, etc.) (kW)	5
Total Number of PDC's	3
Power Consumption of System Control Center (SCC) (PLC, HMI, Power Supplies, Air Conditioner, Fans, etc.) (kW)	-
Maximum Power Consumption (kW)	41
Lamp Replacement	
Total Number of Lamps in System	45
Number of Lamps Operating at Average Conditions[(1)]	45
Guaranteed Lamp Life (hours)	12000
Lamp Replacement Cost	$440
Ballast Replacement	
Total Number of Ballasts in System	45
Guaranteed Ballast Life (years)	5
Ballast Replacement Cost	$490
Quartz Sleeve Replacement	
Total number of Quartz Sleeves in System	45
Guaranteed Quartz Sleeve Life (years)	10
Quartz Sleeve Replacement Cost	$360
UV Intensity Sensor Replacement	
Total Number of UV Intensity Sensors in System	1
Guaranteed UV Sensor Life (years)	3
UV Intensity Sensor Replacement Cost	$850
Annual Cost for Duty and Reference Sensors Calibration	$600
Cleaning System - Wiper Replacement Cost	
Total Number of Automatic Cleaning Wipers in System	45
Guaranteed Wiper Life (years)	2
Wiper Replacement Cost	$25
Annual Cost for Automatic Cleaning System Consumables (Cleaning Solution, etc.)	$100
Notes: (1) Average Conditions is defined as a flow rate of 2 MGD and a UV transmittance of 75%. Use the design End of Lamp Life and Fouling factors in your calculations.	

EXECUTIVE SUMMARY

Proposal # QW-1508-28
Revision: 0

Windsor WWTP, CA - C3500D UV Disinfection System

Calgon Carbon proposes to supply our C3500TM D Ultraviolet Disinfection System to treat effluent at the above site. This system will include 280 UV lamps to treat the peak flow of 8.6 MGD. The system will be configured into 2 channels, 4 banks per channel, 5 racks per bank each with 7 lamps. One bank per channel is provided for redundancy.

The main advantages of the C3500TMD system are as follows:

- The C3500TMD uses the highest power low pressure horizontal lamps available. The UV lamp emits 204 W of UV light at 254 nm. This means our system will have fewer lamps, resulting in less maintenance.
- The $C^3500^{TM}D$ system includes automatic, in-place cleaning as a key feature. This reduces the need for operators to remove lamp racks and manually clean them, significantly reducing maintenance. The Calgon Carbon automatic cleaner is mechanical only – no chemicals are required.
- The patented mixing devices dramatically improve the hydraulic and germicidal efficiency of the UV reactor providing unparalleled performance.

Calgon Carbon is a world leader in granular activated carbon solutions. We are also one of the world's foremost providers of ultraviolet light (UV) disinfection and oxidation technologies for water. From the initial introduction of our UV advanced oxidation systems to the continued development of drinking water and wastewater disinfection technologies, we've been delivering proven UV water treatment solutions for more than 25 years. Combined we have over 500 installations in operation or under construction.

This proposal includes system sizing and a bill of materials. If you need any further information, please feel free to contact David DesRochers at 519.824.8318.

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PROJECT SPECIFICATIONS

Proposal # QW-1508-28
Revision: 0

Windsor WWTP, CA - C3500D UV Disinfection System

1. Design Conditions

Peak Flow Rate	8.6	MGD
Average Daily Flow Rate	2.0	MGD
Minimum UV Transmittance	67 %	
Average UV Transmittance	75%	
Total Suspended Solids	1.8 mg/L	
Total Coliform Permit Limit	2.2 CFU/100mL, based on 7 day median	
	23 CFU/100 mL, based on maximum grab sample	

2. $C^3$500D Specifications

Lamp UV Radiation at 254 nm	204
Lamp Life	16,000 Hours
Lamp Life Factor	0.85
Quartz Transmission Factor	0.92
Quartz Fouling Factor	0.90
MS2 T22 RED	106 mJ/cm2 validated

3. System Configuration

Number of Channels	2	
Number of Banks/Channel	4	One bank is provided for redundancy
Number of Racks/Bank	5	
Number of Lamps/Rack	7	
Total Number of Lamps	280	
Number of UV Sensors	8	
Number of Power Distribution Centers	8	
Number of System Control Centers	2	
Number of Weirs	2	

4. Hydraulic Considerations

Peak Velocity in Channel	9.12 inches/s
Headloss per UV Bank	0.34 inches
Headloss across Level Control Device	6 inches
Total Headloss across UV System	7.36 inches
Retention Time	17.36 seconds

5. Electrical Requirements

Input Voltage	480/277 VAC, 3 Phase, 4 Wire, 60 Hz
Peak Loading per PDC	28.99 Amps
Power Consumption per PDC	22.09 kW
Total System Power Consumption	176.74 kW

6. Approximate Channel Dimensions

Length	683 inches
Width	49 inches
Width with Channel Reduction Baffle	30 inches
Width at Level Control Weir	49 inches
Depth	66 inches
Depth at Level Control Weir	44 inches
Effluent Depth in Channel	42 inches, nominal

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BILL OF MATERIALS

Proposal # QW-1508-28
Revision: 0

Windsor WWTP, CA - C3500D UV Disinfection System

Item No.	*Qty.*		*Model Number C3500D2405071WP*
1.	2	Channel Equipment	
		Qty. 1	Level Control Weir
		Qty. 4	Integrated Bank Assembly
		Qty. 16	Bank Guides
		Qty. 4	Point Ultrasonic Level Sensor with Mounting Bracket
		Qty. 4	UV Sensors with Mounting Bracket and Scrapers
		Qty. 4	Channel Reduction Baffles
2.	40	Lamp Rack Assemblies	
		Qty. 7	Low Pressure High Intensity Amalgam Lamps
		Qty. 7	Quartz Sleeves
		Qty. 2	Cable Assemblies
		Qty. 1	Cleaning System Motor
		Qty. Lot	Cleaning System Mechanism and Accessories
		Qty. 7	Scrapers
3.	8	Power Distribution Centers	
		Qty. 35	Electronic Ballasts
		Qty. 1	Main Breaker
		Qty. 5	Earth Leakage Circuit Breakers
		Qty. Lot	Control & Monitoring Equipment with Operator Station
		Qty. 1	Air Conditioner
4.	2	System Control Center	
		Qty. 1	Main Breaker
		Qty. Lot	Allen Bradley CompactLogix L32E PLC Equipment and Accessories
		Qty. 1	Allen Bradley PanelView 1000 Plus Operator Interface
5.	Lot	Spare Parts and Accessories	
		Qty. 1	UV Face Shield
		Qty. 14	Electronic Ballasts
		Qty. 28	UV Lamps
		Qty. 28	Quartz Sleeves
		Qty. 28	Lamp End Seals
		Qty. 1	Service Trolley
		Qty. 1	Rack Lifting Sling
		Qty. 1	Lifting Sling for Integrated Bank Assembly
		Qty. 1	On-line UV Transmission Analyzer
		Qty. 1	Reference UV Sensor
6.	Lot	Equipment Documentation	
7.	Lot	Start-up and Commissioning Services	
8.		One (1) Year Warranty Period	

Terms and Conditions

Payment Terms:	CCC Standard Terms
FOB:	Jobsite
Delivery:	20 to 24 weeks after receipt of approved shop drawings

CCC Standard Terms & Conditions Will Apply

Budgetary Price (USD): $986,000

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APPENDIX D

Project: Town of Windsor, CA

Description	
Equipment Cost	
UV Disinfection System (Complete)	$ 986,000.00
Total Capital Costs:	**$ 986,000.00**
Operation and Maintenance Cost Information	
Power Consumption	
Power Consumption (kW) at Average Conditions[(1)]	27.3
Lamp and Ballast Power Consumption at Full Power, all Lamps (kW)	161
Ancillary Power Consumption per Power Distribution Center (PDC) (Power Supplies, Air Conditioners, Fans, etc.) (kW)	2.45
Total Number of PDC's	8
Power Consumption of System Control Center (SCC) (PLC, HMI, Power Supplies, Air Conditioner, Fans, etc.) (kW)	2.68
Maximum Power Consumption (kW)	183.28
Lamp Replacement	
Total Number of Lamps in System	280
Number of Lamps Operating at Average Conditions[(1)]	35
Guaranteed Lamp Life (hours)	16000
Lamp Replacement Cost	$250
Ballast Replacement	
Total Number of Ballasts in System	280
Guaranteed Ballast Life (years)	10
Ballast Replacement Cost	$400
Quartz Sleeve Replacement	
Total number of Quartz Sleeves in System	280
Guaranteed Quartz Sleeve Life (years)	15
Quartz Sleeve Replacement Cost	$90
UV Intensity Sensor Replacement	
Total Number of UV Intensity Sensors in System	8
Guaranteed UV Sensor Life (years)	3
UV Intensity Sensor Replacement Cost	$1,300
Annual Cost for Duty and Reference Sensors Calibration	$933
Cleaning System - Wiper Replacement Cost	
Total Number of Automatic Cleaning Wipers in System	280
Guaranteed Wiper Life (years)	5
Wiper Replacement Cost	$70
Annual Cost for Automatic Cleaning System Consumables (Cleaning Solution, etc.)	$0

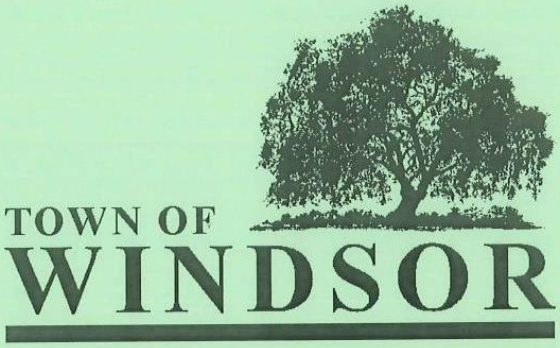
Notes: (1) Average Conditions is defined as a flow rate of 2 MGD and a UV transmittance of 75%. Use the design End of Lamp Life and Fouling factors in your calculations.

Appendix K: 2013 to 2016 Annual Reports of Solids Disposal

Refer to electronic version of the report for the complete appendix. The first page is shown here for reference.

Town of Windsor
8400 Windsor Road
P.O. Box 100
Windsor, CA 95492-0100
Public Works Department (707) 838-1006
Fax (707) 838-3137

Mayor, Bruce Okrepkie
Vise Mayor, Steve Allen
Councilmember, Debra Fudge
Councilmember, Robin Goble
Councilmember, Sam Salmon



TOWN OF
WINDSOR

2013 ANNUAL REPORT OF SOLIDS DISPOSAL

Prepared by: Brian Diamantini
Wastewater Systems Supervisor
And
Michael Carson
Management Analyst

January 2014

Appendix L: Solids O&M Cost Estimates

By **DWW** **Subject:** **SOLIDS TREATMENT O&M COSTS ESTIMATES**

Date: 10/12/2015

GBT

		Quantity	Units	Cost	Units	Operation Time		Annual Cost	
Power		10	HP	$0.12	KW-hr	50%		$ 2,800.00	
Polymer		5	GPD	$11	per day			$ 14,300.00	
Operations		2	Hr per day	$100	per hr	5	days / week	$ 52,100.00	
Maintenance		1	percent	$350,000				$ 3,500.00	
								$ 72,700.00	
Disposal		550	tons	$375				$ 588,950.00	
							TOTAL	**$ 662,000.00**	

BFP

		Quantity	Units	Cost	Units	Operation Time		Annual Cost	
Power		20	HP	$0.12	KW-hr	50%		$ 5,600.00	
Polymer		50	GPD	$11	per day			$ 143,400.00	
Operations		3	Hr per day	$100	per hr	5	days / week	$ 78,200.00	
Maintenance		1	percent	$550,000				$ 5,500.00	
								$ 232,700.00	
Disposal		550	tons	$375				$ 588,950.00	
							TOTAL	**$ 822,000.00**	

Digester

		Quantity	Units	Cost	Units	Operation Time		Annual Cost	
Power		20	HP	$0.12	KW-hr	50%		$ 5,600.00	
Polymer		0	GPD	$10	per day			$ -	
Operations		4	Hr per day	$100	per hr	5	days / week	$ 104,300.00	
Maintenance		1	percent	$1,000,000				$ 10,000.00	
GBT								$ 72,700.00	
BFP								$ 232,700.00	
Disposal		550	tons	$375				$ 206,250.00	
							TOTAL	**$ 632,000.00**	

Sludge Disposal (Based on 2015 Town of Windsor Bid and 2015 Petaluma Disposal costs)

Disposal		550	tons	$375				$ 206,250.00	
Dredging and Disposal Bid								$ 588,950.00	
Dredging and Dewatering Bid								$ 382,700.00	