Town of Wellington Wastewater Treatment Masterplan

July 13, 2021

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1	11/23/2020	Draft incorporating Town comments	HS	KS		
2	12/11/2020	Incorporating final comments	HS	KS	RS	RS
3	7/13/2021	Incorporating Comprehensive Plan	HS	KS	RS	

Contents

Executive Summary

The Town of Wellington contracted the services of Jacobs to perform a master plan evaluation of the existing wastewater treatment plant (WWTP), with a focus on examining the remaining available plant capacity and to provide a planning document for use in determining appropriate next steps for the WWTP. This planning-level document provides an analysis of the existing WWTP using a computer process treatment model to examine existing facilities and to explore the performance and sizing requirements for proposed improvements. This report includes results of the analysis of historical treatment plant data, examines alternative solutions and makes a recommendation for phased expansions to serve the Town's anticipated build-out population of approximately 35,500.

The Town has experienced significant population growth in the past few years and this trend is expected to continue. This will lead to an increase in flow and loads at the WWTP. The population of the Town of Wellington in 2019 was 10,421. The buildout projection is currently estimated at a population of approximately 35,500. Furthermore, the existing plant operates very close to its current rated capacity, triggering the need for prompt design of an expansion and subsequent construction.

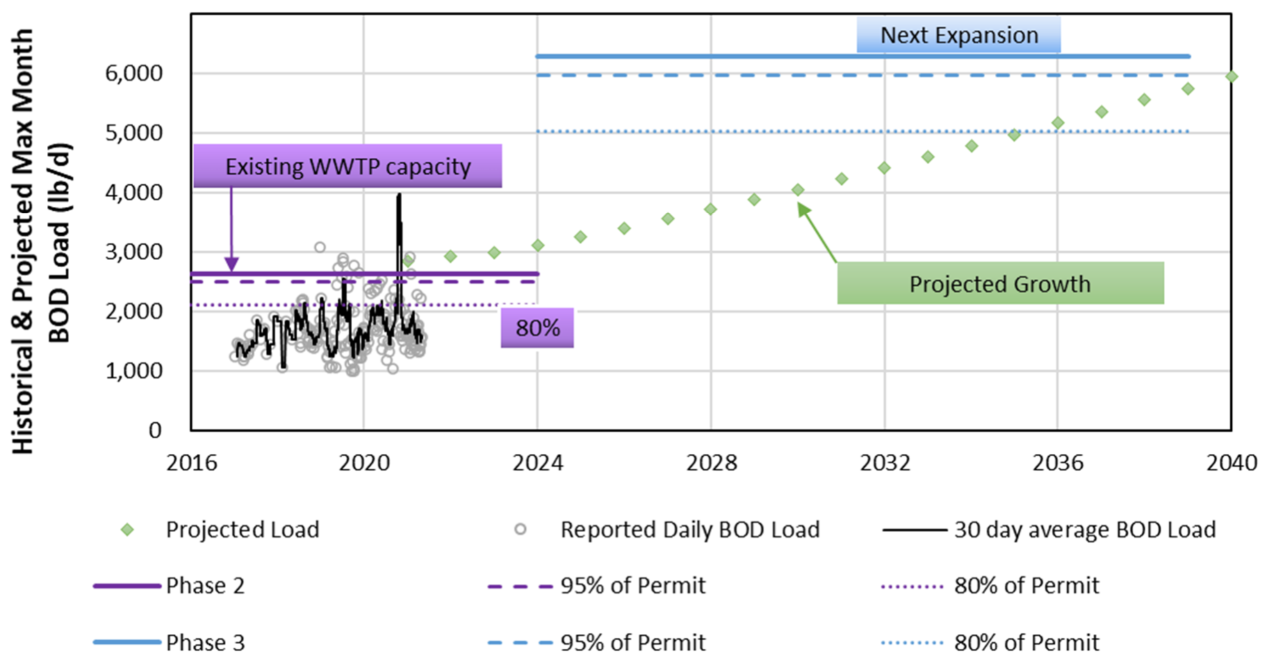


Figure ES-1. WWTP Recent and Projected Influent Load.

The Colorado Department of Public Health and Environment (CDPHE) requires that the design of an expansion to the WWTP begin at 80% of rated maximum month capacity for permitted flow and/or loads and be under construction no later than when the WWTP reaches 95% of capacity. These requirements are regulated under CDPHE regulations and polices and are included in the Town's WWTP Colorado Discharge Permit System (CDPS) permit. Dotted and dashed lines in Figure ES-1 above represent the 80% and 95% of capacity that trigger the start of design and construction respectively per CDPHE. As part of the analysis of existing plant data, it was found that in 2018 the 30-

day rolling average influent load exceeded 80% of the permit. In 2019 the 30-day rolling average exceeded 95% of the permit.

The capacity of the WWTP is limited by its ability to treat the organic loading, referred to as Biochemical Oxygen Demand (BOD). Due to an increase in indoor water use efficiency over the past few decades, the concentration of organic loading has increased, putting many WWTP's in the position of having some spare hydraulic capacity but being limited by their rated organic capacity. As illustrated in Figure 1, the treatment plant has experienced organic (BOD) loadings that have occasionally exceeded the State's trigger point of 80% capacity for commencing design of an expansion. Refer to Section 1.3.2 of this report for more detailed information.

Alternative means of meeting the increasing treatment capacity are explored in this report. An alternatives analysis is discussed in Sections 2 and 3 that used a high-level comparison of construction costs and weighed the technical advantages and disadvantages of various solutions.

The recommendation of this master plan is that a phased approach to expansion is appropriate. The main treatment system of the existing plant can remain in service, with new facilities being constructed to increase treatment capacity and improve overall operational efficiency. A preliminary layout of proposed improvements is presented in the site plan, Figure ES-2. Existing (referred to as Phase 2) facilities are highlighted in purple, Phase 3 (proposed expansion) facilities are highlighted in teal, Phase 4 (future build out) facilities are highlighted in grey, and additional future facilities are highlighted in red.

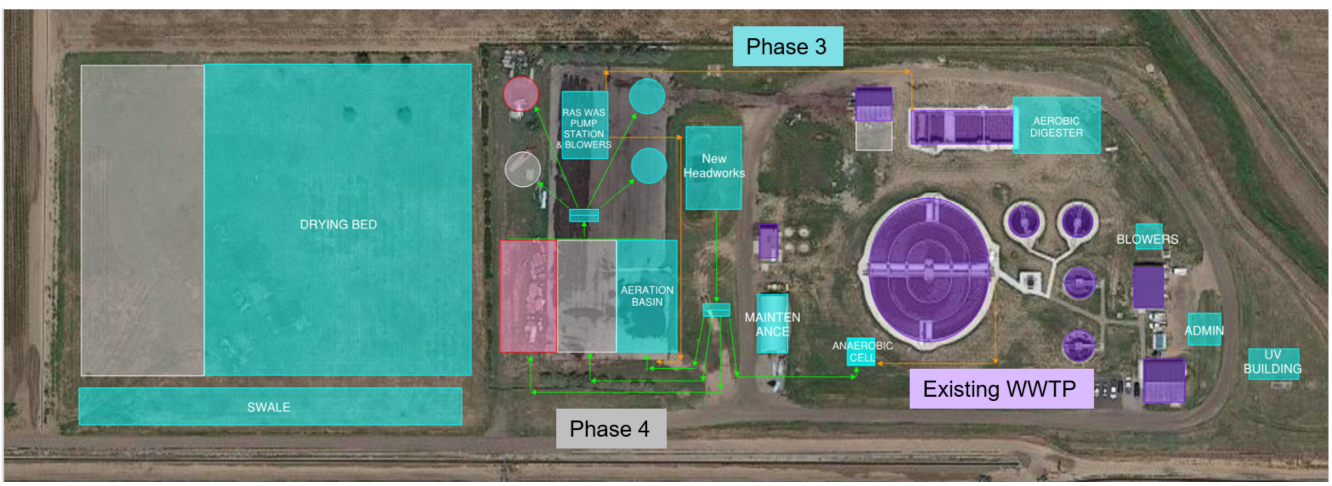


Figure ES-2. Layout of Recommended Alternatives.

An important consideration when planning for build-out capacity is to verify that the Town's existing WWTP site property is large enough and appropriately located to accommodate the planned expansions. While some additional adjacent property would be beneficial to facilitate expansion and provide buffer area with adjacent property, this compact design approach presents one option for containing the expansions on the current property owned by the Town.

The process facilities and areas colored in blue (teal) represent an approach to accomplishing the proposed expansion; the gray facilities reflect the following future expansion to accomplish build-out capacity. The red facilities are what could be installed to allow for abandonment of the existing treatment train in the future if increasingly stringent treatment regulations, system performance, or the condition of aging infrastructure became significant. Additional information on the proposed phases and capacities is described in Section 2.2 and Section 2.3.

Estimating approximate capital expenditure requirements is a key component of master planning. Section 2.4 and Section 3.3 provide an analysis of the liquids and solids alternatives and also describe the high-level master planning opinions of cost used in this evaluation. Current cost opinions for implementation of the proposed next phase (Phase 3) based on the high-level estimating performed for this master plan indicate a construction cost with Markups on the order of $39.9 M. This includes non-construction cost items at an assumed 17% for engineering design, services during construction, permitting, legal and other miscellaneous work summing to about $5.9 M. More precise opinions of cost are developed as solutions are refined in the preliminary design, and when more information is available about the level of finishes, redundancy, geotechnical conditions and the approach to construction contracting.

1. Introduction

The Town of Wellington contracted the services of Jacobs to perform a master plan evaluation of the existing wastewater treatment plant (WWTP), with a focus on examining the remaining available plant capacity and to provide a planning document for use in determining appropriate next steps for the WWTP. This planning-level document provides an analysis of the existing WWTP using a computer process treatment model to examine existing facilities and to explore the performance and sizing requirements for proposed improvements. This report includes results of the analysis of historical treatment plant data, examines alternative solutions and makes a recommendation for phased expansions to serve the Town's anticipated build-out population of approximately 35,500.

1.1 Background

The Town of Wellington owns a wastewater treatment plant (WWTP) that was built in 2002 to 2003 and expanded in 2016. The plant treats all wastewater flows that are discharged to the Town's wastewater collection system prior to discharge to Boxelder Creek. The WWTP operates under the CDPS No. CO0046451. The WWTP is rated at 0.9 million gallons per day (MGD) maximum month average flow capacity. The plant consists of the following treatment processes:

- Screening
- Grit Removal
- Influent Pumping
- Aeration
- Secondary Clarification
- UV Disinfection
- Aerobic Digestion
- Sludge Dewatering
- Sludge Air Drying

1.2 Town Population Growth Projections

The 2019 population for the Town of Wellington was 10,421. The buildout projection is currently estimated at a population of 35,500 using the Town's projected annual growth rate each year from 2019. The actual rate of growth is less critical to the planning process than the projected buildout population, although growth rate clearly impacts the timing of the phased expansions.

Table 1-1 illustrates the Town's historical and projected population estimates from the State of Colorado demographer's office for the years 2013- 2019. Based on the population projections and the flow and load projection analysis described in later sections, phasing recommendations were developed. Historical and proposed plant expansion phases are shown in Table 1-2 below.

Table 1-1. Historical and Projected Town Population

Year	Population	Change in Population Annual	Population Growth (%)
2013*	6,669		
2014*	7,090	421	6%
2015*	7,665	575	8%
2016*	8,294	629	8%
2017*	9,430	1,136	14%
2018*	9,900	470	5%
2019*	10,431	531	5%
2020	11,415	983	9%
2021	11,802	387	3%
2022	12,119	317	3%
2023	12,375	256	2%
2024	12,855	480	4%
2025	13,459	604	5%
2026	14,085	626	5%
2027	14,732	648	5%
2028	15,403	670	5%
2029	16,096	693	5%
2030	16,812	716	4%
2031	17,544	731	4%
2032	18,289	746	4%
2033	19,048	759	4%
2034	19,820	771	4%
2035	20,602	783	4%
2036	21,396	793	4%
2037	22,198	802	4%
2038	23,008	810	4%
2039	23,825	817	4%
2040	24,647	822	3%

**2013-2019 Population from the State Demographers Office, Department of Local Affairs.*

Table 1-2. Historical and Proposed Plant Phases

Phase	Year Online	Description
Phase 1	2004	Original Plant
Phase 2	2016	The current existing facilities
Phase 3	Proposed 2024	The expansion proposed in this Masterplan
Phase 4	Proposed 2039	A future expansion with additional facilities and equipment
Phase 5	TBD	A future expansion with additional facilities and equipment to allow the existing (Phase 2) plant to be decommissioned

A visualization of the impact of different population growth rates is given in Figure 1-1. Growth of influent flow and loads to the wastewater treatment plant are expected to be driven by population growth. Flow capacities for historical and proposed phases are shown by the colored horizontal lines. The portion of industrial, civic, and commercial are expected to grow in relation to the residential land use in the Town. To accommodate these increases in expected water use and wastewater production, the projected flows were increased over just the increase from population growth alone.

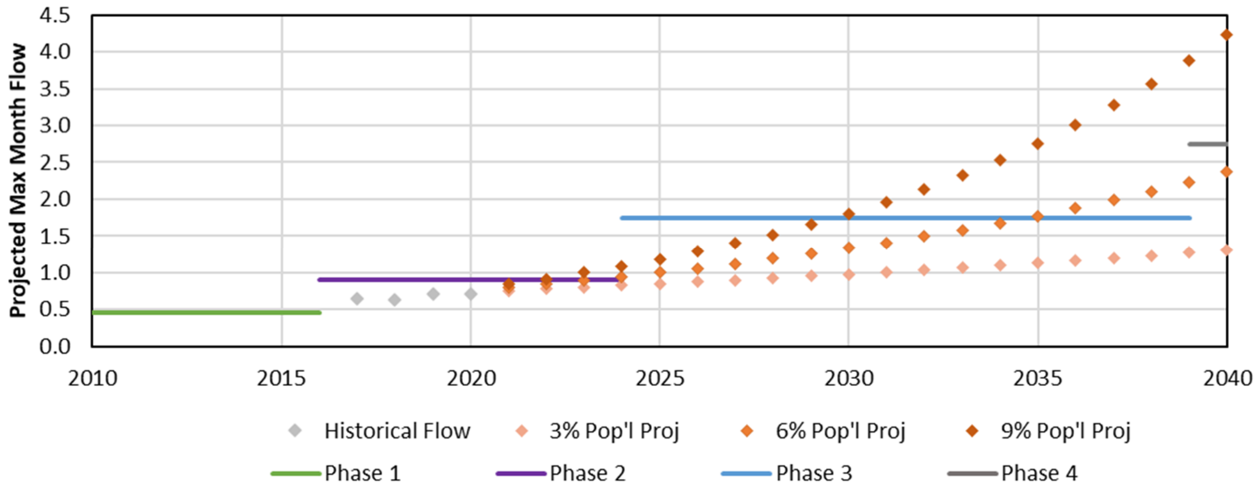


Figure 1-1. Flow Projections Based on 3%, 6%, and 9% Growth

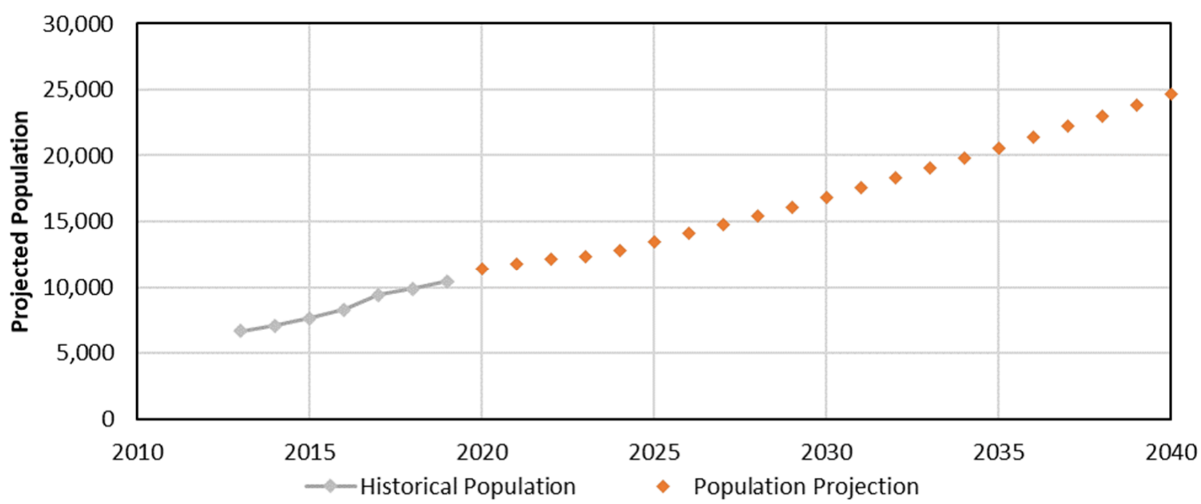


Figure 1-2 Historic and Projected Population Based on Town Growth Analysis

The projected flow and loads based on the Town's predicted annual growth rate and the capacity of historic and proposed phases are shown in Figure 1-3 and Figure 1-4. The 80% and 95% of capacity for each phase are represented by dotted and dashed lines respectively. An in-depth evaluation of current flows and loads used to project into the future is presented in the next section of the report.

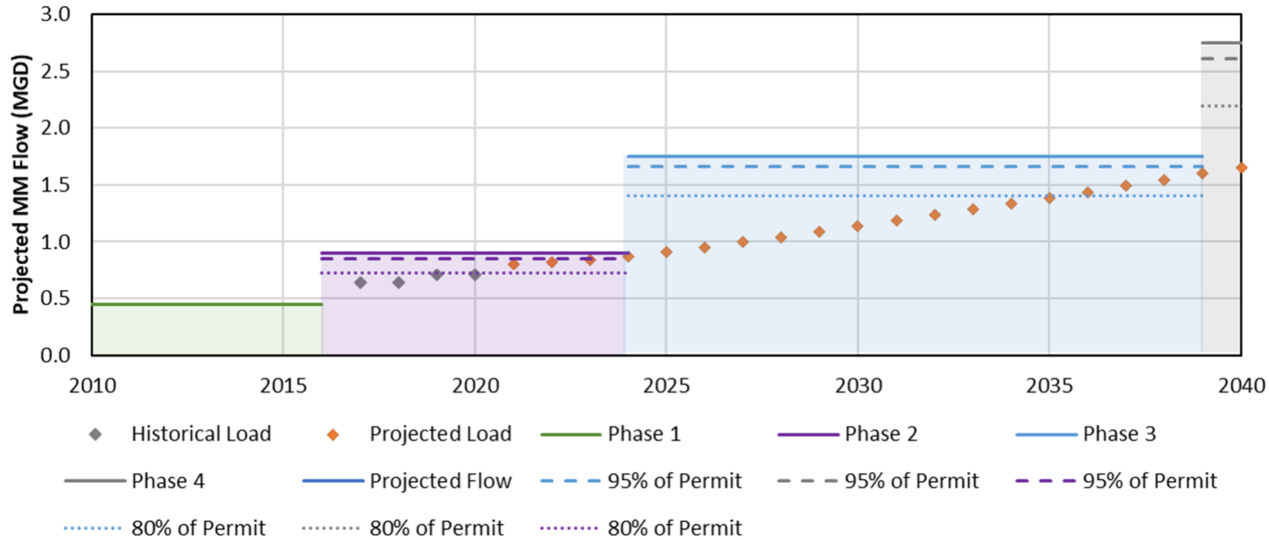


Figure 1-3. Projected Maximum Month Flow Projections based on Population Projections.

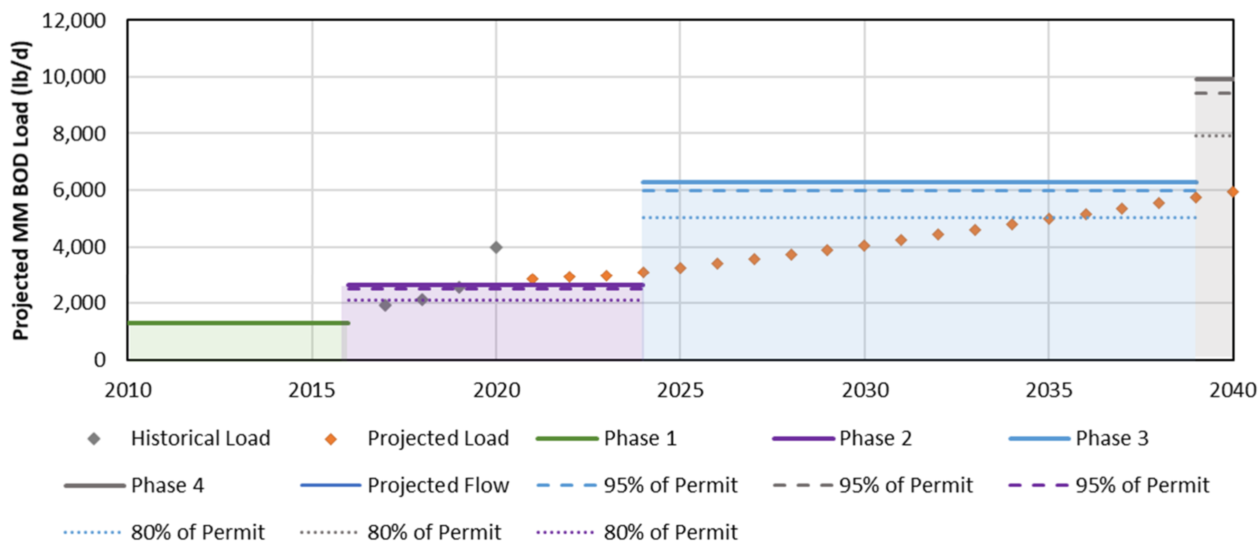


Figure 1-4. Projected Maximum Month BOD Load Projections based on Population Projections

1.3 Current Flows and Loads (influent)

As outlined in the previous section, the Town of Wellington is experiencing significant population growth. The recent historical flows and loads are presented below to assess the current conditions and project into the future.

1.3.1 Influent Flow

The daily influent flow measured at the Wellington WWTP is given in Figure 1-5 in gal/d. The rolling 30-day average is plotted alongside the daily data. The rolling average is used to evaluate trends and to calculate the current maximum month flows which are then used to project future values associated with population growth. In the Fall of 2019, there were many days that exceeded 80% of the permitted flow (720,000 gal/d). The residuals flow from the seasonally used groundwater nanofiltration water treatment plant was shut off during the winters of 2017/18 and 2019/20; the corresponding drop in the wastewater influent attributed to the nanofiltration Water Treatment Plant (WTP) residual flow can be seen in the figure. When the WTP residuals are not entering the WWTP, the WWTP staff have observed a drop in influent alkalinity and the nitrification performance is impacted.

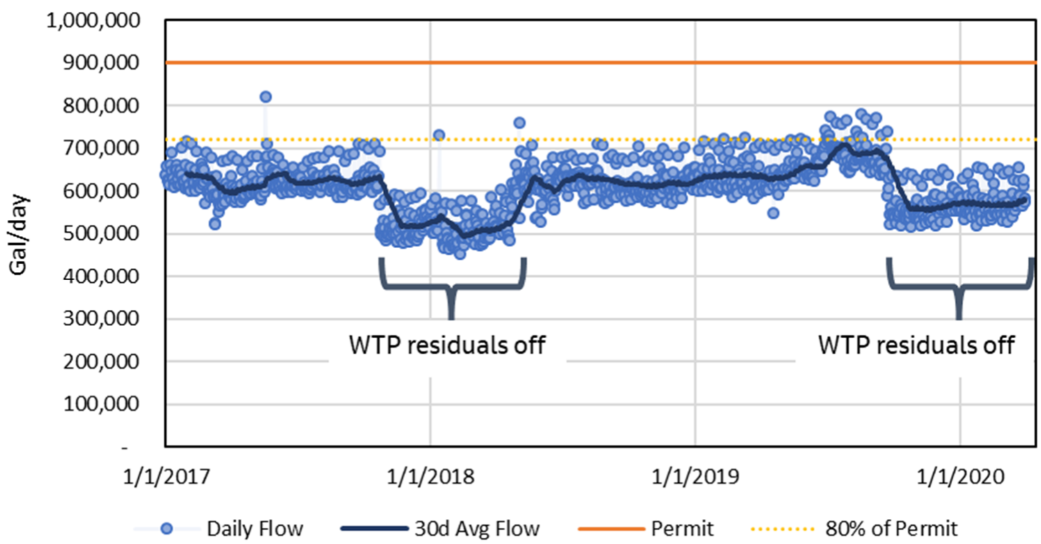


Figure 1-5. Historical Influent Flow

1.3.2 Influent BOD and TSS

The daily influent BOD and TSS loads are given in the figures below (Figure 1-5 and Figure 1-6). The BOD and TSS exhibit more variability than the influent flow to the plant. A significant number of observations in 2019 were greater than the permitted load of 2,627 lb BOD / day, particularly in the summer. The periodic trends in the TSS load correlate to that of the BOD load.

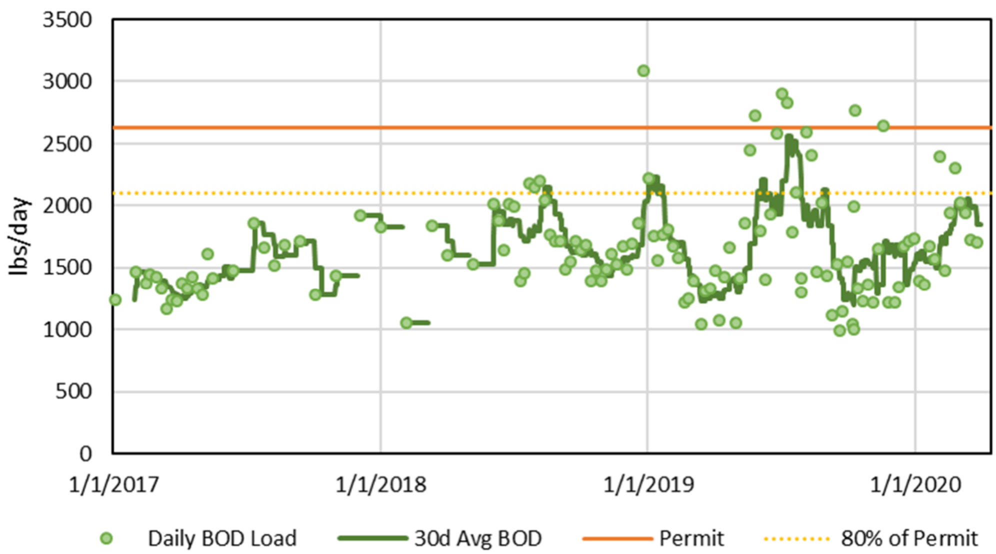


Figure 1-6. Historical Influent BOD Load

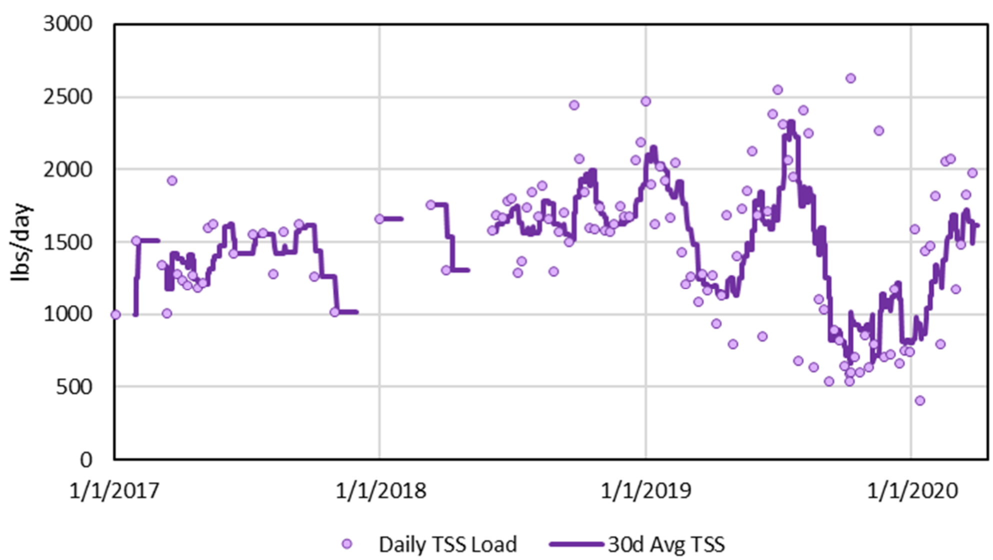


Figure 1-7. Historical Influent TSS Load

The current permit at Wellington has no TSS load limit.

1.3.3 Peaking Factors and Wet Weather

Current flows and loads at the WWTP were used to calculate the per capita flows and loads. The 2019 average daily BOD and TSS loads were divided by the population of 2019 to calculate the annual average load per capita. The ammonia and total phosphorus loads are based on the BOD loads and the ratio of ammonia to BOD and phosphorus to BOD during the influent special sampling campaign and recent historical data. The historic per capita rates are presented in Table 1-3.

Table 1-3. 2019 Per Capita Flows and Loads

	Flow*	BOD Load	TSS Load	NHx Load	TP Load	**Population**
Condition	gal/cap/day	lb/cap/day	lb/cap/day	lb/cap/day	lb/cap/day	
Annual Average	60.6	0.16	0.129	0.024	0.004	10,431
Maximum Month	68	0.245	0.223	0.037	0.007	

**This includes industrial, commercial, civic, residential, and partial year Nanofiltration plant discharges*

The Town desires additional capacity for civic and industrial water uses in the future. To accommodate this, the following projected per capita flow was developed from the Town's Comprehensive Plan during the Collection System Masterplan. The projected Maximum Month flow is based on the following calculation:

Flow = 54 gpcd * pop'l in 2019 + 66 gpcd * new pop'l since 2019 + 0.15 MGD from Nano Plant

This approach increases the annual average flow per capita in the collection system for growth post 2019, as requested by the Town. This increase in flow per capita is reasonable given that the Town currently has relatively few non-residential dischargers, whereas the Comprehensive Plan indicates an intention to encourage more civic, commercial and industrial growth. The flow discharged from the Nano/RO Plant is independent of population, and thus remains a constant in these flow projections. The future per capita flow contributions are summarized in Table 1-4.

Table 1-4 Future Per Capita Flows

	2019 Population	New Population since 2019	Nanofiltration plant discharges
Unit	gal/cap/day	gal/cap/day	MGD
Maximum Month Flow Contribution	54*	66*	0.15

*This includes industrial, commercial, civic, and residential

The projected Maximum Month flow for a given year is based on the predicted population of that year in the formula above; the Annual Average flow is calculated from the Maximum Month flow divided by the Maximum Month to Annual Average peaking factor for flow.

The projected Maximum Month loads for a given year are based on the predicted population of that year, multiplied by the Annual Average load per capita, multiplied by the Maximum Month to Annual Average peaking factor for each load. The peaking factors (ratios of peak values to average values) used for this design analysis are based on the 95th percentile of the historical ratios between Maximum Month and Annual Average flows, loads, and temperature. For flow, the Peak Day and Peak Hour peaking factors were also calculated. The peaking factor definitions are given in Table 1-4 and the values are given in Table 1-6.

Table 1-5. Peaking Factor Terms

Term	Definition
AA	Annual Average
MM	Maximum Month (maximum value for 30-day running average or 11/12th percentile of annual data.
PD	Peak Day (maximum value in annual daily data or 364/365th percentile of annual data)
PH	Peak Hour (maximum value in annual hourly data)

Table 1-6. Historic and Projected Peaking Factors for Influent Flows and Loads

Peaking Factors	MM:AA	PD:AA	MM:AA	MM:AA	MM:AA
Constituent	Flow	Flow	BOD	TSS	Temp
2017	1.06	1.19	1.33	1.19	0.73
2018	1.09	1.25	1.23	1.17	0.68
2019	1.12	1.22	1.54	1.73	0.74
Recommended for Design	**1.12**	**1.25**	**1.52**	**1.68**	**0.68**

Peaking Factors	PH:AA
Constituent	Flow
2017	3.42
2018	2.28
2019	2.15
2020	2.14
Recommended for Design	**3.1**

For BOD and TSS, the historical peaking factors increased as the influent variability increased. The flow, on the other hand, is very stable aside from seasonal residual discharges from the nanofiltration plant changes. Looking at the historical influent flow in Figure 1-4, there does not appear to be significant inflow and infiltration into the collection system which create significant wet weather flow peaks. The Peak Hour flow peaking factor for design is based on the 90th percentile of the 2017-2020 hourly flow peaking factors; 2020 data was included to make use of collection system monitoring and modeling.

1.4 Future Flows and loads

Based on the current flows and loads (the average of 2019) and the population projections for the Town's service area, the projected maximum month flows and loads have been developed and are presented in Table 1-7.

Table 1-7. Projected Maximum Month Flows and Loads

Year	Flow (MGD)	BOD (lb/d)	TSS (lb/d)	NHx (lb/d)**	TP (lb/d)**
2019*	0.71	2,525	2,322	279	59
2025	0.91	3,250	2,900	380	90
2030	1.13	4,060	3,620	480	110
2035	1.38	4,980	4,440	590	140
2040	1.65	5,960	5,310	700	165
2045	1.91	6,900	6,160	810	190

**Historical values for flow, BOD, and TSS load.*
***The current and projected Ammonia and Phosphorus loads are based on ratios to the BOD load during the special sampling campaign in October 2019 and additional data collection in 2020.*

1.5 Existing and Potential Future Effluent Limits

Existing and potential future effluent limits are discussed in the following section.

1.5.1 Current Requirements and Effluent Results

The current effluent limits for the wastewater treatment plant are outlined in the Town's CDPS permit for discharge to Boxelder Creek; these limits are expected to be in effect through May 31, 2023. The permit effluent and monitoring requirements are summarized in the Table 1-8.

Table 1-8. Current Effluent Limits for Wastewater Effluent

ICIS Code	Effluent Parameter	Effluent Limitations Maximum Concentrations			Monitoring Requirements	
		30-Day Average	7-Day Average	Daily Maximum	Frequency	Sample Type
50050	Effluent Flow (MGD)	0.9		Report	Daily	Continuous
00400	pH (su)			6.5-9.0	Daily	Grab
51040	*E. coli* (#/100 ml) (5/15-9/16), until December 31, 2020	205	410		Weekly	Grab
51040	*E. coli* (#/100 ml) (5/15-9/16), starting January 1, 2021	163	410		Weekly	Grab
51040	*E. coli* (#/100 ml) (9/16-5/14), until December 31, 2020	630	1260		Weekly	Grab
51040	*E. coli* (#/100 ml) (9/16-5/14), starting January 1, 2021	163	1260		Weekly	Grab
50060	TRC (mg/l)	0.006		0.019	3 Days/Week	Grab
00610	Total Ammonia as N (mg/l)					
	January	4.1		18	Weekly	Composite
	February	3.5		16	Weekly	Composite
	March	3.7		17	Weekly	Composite
	April	3.3		17	Weekly	Composite
	May	2.8		17	Weekly	Composite
	June	2.6		16	Weekly	Composite
	July	2.3		16	Weekly	Composite
	August	1.9		18	Weekly	Composite
	September	2.1		17	Weekly	Composite
	October	2.1		17	Weekly	Composite
	November	3.2		18	Weekly	Composite
	December	3.3		18	Weekly	Composite
00310	BOD_5, effluent (mg/l)	30	45		Weekly	Composite
81010	BOD_5 (% removal)	85 (min)			Weekly	Calculated
00530	TSS, effluent (mg/l)	30	45		Weekly	Composite
81011	TSS (% removal)	85 (min)			Weekly	Calculated
84066	Oil and Grease (visual)	NA		Report	Daily	Visual
03582	Oil and Grease (mg/l)			10	Contingent	Grab
01323	Se, PD (µg/l), until December 31, 2021	Report		Report	Monthly	Composite
01323	Se, PD (µg/l), starting January 1, 2022	3.2		Report	Monthly	Composite

1.5.2 Future Effluent Limitations

1.5.2.1 Regulation 85- Nutrients Management Control Regulation (5 CCR 1002-85)

When the rated capacity of the wastewater treatment plant exceeds 1 MGD, the plant will need to comply with Regulation 85 (Nutrients Management Control Regulation 5 CCR 1002-85). The effluent limits for this regulation are summarized in the Table 1-9. Total Inorganic Nitrogen is the sum of Ammonia, Nitrite, and Nitrate nitrogen. The Town has applied to be part of the Voluntary Incentive Program where the months and degree to which the facility is able to surpass the Regulation 85 requirements is rewarded with a proportional delay in future more stringent effluent requirements under Regulation 31 (The Basic Standards and Methodologies For Surface Water 5 CCR 1002-31). It should be noted that WWTPs with a capacity of less than 2 MGD are granted delayed implementation until December 31, 2027 to meet the Regulation 85 limits.

Table 1-9. Regulation 85 Nutrient Requirements

	Effluent Total Inorganic Nitrogen mg N/L	Effluent Total Phosphorus mg P/L
Running Annual Median	15	1
95th Percentile of most recent 12 months	20	2.5

1.5.2.2 Selenium

The Town of Wellington's current permit includes a limit for Selenium of 3.2 microgram/L that will take effect January 1, 2022. The Town's selenium discharge loading appears to be driven by the use of the nanofiltration WTP and subsequent discharge of dissolved solids removed following either backwashing of the nanofiltration membranes or Reverse Osmosis (RO) treatment of the nanofiltration reject water. The Town is currently evaluating the discharge to the collection system from the WTP process and is considering a treatment unit to remove selenium at the source.

1.5.2.3 Pretreatment Requirements

The Town has consulted with the United States Environmental Protection Agency (EPA) on their pretreatment program. Town code includes limits on the discharge of metals to the collection system. In 2020 the Town is distributing a pretreatment survey to commercial users and is also developing a modification to the code which would allow surcharges based on BOD and TSS in the wastewater discharge.

Formal Pretreatment programs are not required for facilities rated below 5 MGD.

1.5.2.4 Possible EPA Regulatory Changes

The following discussion includes both proposed and recently adopted criteria from EPA. These criteria do not immediately become water quality standards for dischargers in Colorado.

- Bacteria – EPA is working on the development of water quality criteria for viruses. EPA and state agencies currently monitor E. coli and enterococci, which serve as indicators of bacterial fecal contamination. However, the new water quality criteria will be based on the presence of coliphages. Coliphages are viral indicators, which help provide a clearer picture of viral activity in water. A recent literature review by the EPA found that 5 out of 8 studies observed a statistically significant relationship between coliphage and gastrointestinal illness levels. The EPA acknowledges that coliphages are not always well correlated with the presence of human viruses in environmental waters. The EPA is working on developing more accurate methods to address these issues and concerns. Additionally, according to a May 2016 Inside EPA Water Policy Report, the 'virus' criteria will likely be used to supplement the existing water quality criteria, not replace it. The last status update by EPA was that the new criteria would be proposed in 2016 and finalized it in 2017. However, no criteria were proposed in 2016. It is unclear if or when a virus based standard will be proposed.

- Health based Standards – There are 90+ pollutants that have limits tied to fish consumption, such as Arsenic. EPA recently increased the amount of fish and water consumed per person which will result in an increase in the toxicity levels for these pollutants.
- Cadmium – In March 2016, the EPA finalized modifications to the dissolved cadmium criteria. The criteria are based on an equation and is a function of hardness of the receiving stream. The updated acute criteria are slightly more stringent than the previous 2001 criteria. However, the chronic criteria are significantly less stringent. A comparison of the values based on a hardness of 100 mg/L are provided in Table 1-10 below.

Table 1-10. Past and Current EPA Freshwater Cadmium Criteria

	2001 Criteria		2016 Criteria	
	Acute (1-day, dissolved Cd)	Chronic (4-day, dissolved Cd)	Acute (1-hour, dissolved Cd)	Chronic (4-day, dissolved Cd)
Freshwater (Total Hardness = 100 mg/L as $CaCO_3$)	2.0 µg/L	0.25 µg/L	1.8 µg/L	0.72 µg/L

1.5.2.5 PFAS

Per- and Poly-Fluoroalkyl Substances (PFAS) are a large family of organic compounds, including more than 3,000 manufactured fluorinated organic chemicals used since the 1940s. Studies indicate that exposure to some PFAS may lead to adverse health effects. When raw wastewater contains PFAS it may remain is the liquid phase and exist in the final effluent. PFAS may also accumulate in biosolids and pose health risks when land applied and contamination moves to edible crops, animal forage, and runoff to surface and groundwater.

There is currently no national or CDPHE regulatory standard for PFAS, but this may change in the future.

1.6 Existing Facility Summary

The existing facility consists of initial construction from 2003 and an expansion in 2016. The facility sits on approximately 15 acres of land and has roughly 5 acres of open area available for further expansion plus space between existing facilities. The areas of the facility that would benefit from improvement are highlighted in Figure 1-8.

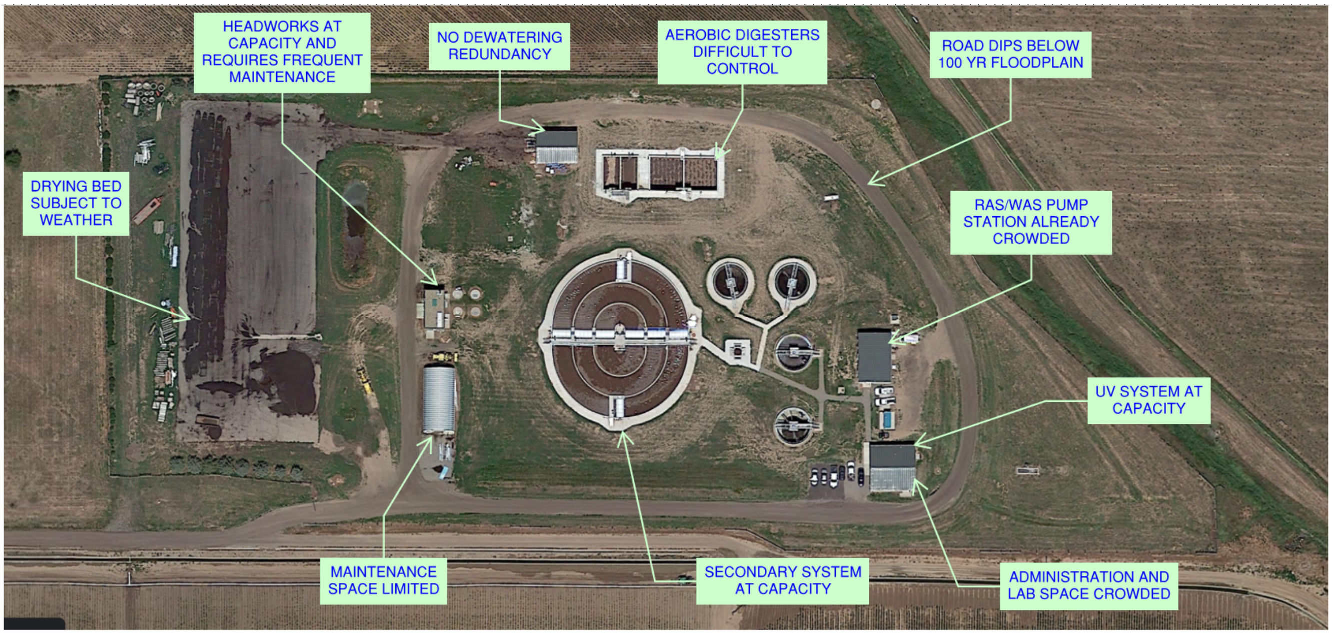


Figure 1-8. Wellington Wastewater Treatment Facility Areas for Improvement

Part of the WWTP property is encroached upon by the 100-year floodplain (see Figure 1-8) per the current Flood Insurance Rate Map (FIRM). The road in the Northeast corner of the facility may need to be raised to protect the plant from flooding. Wellington may also want to set up a buffer around the facility from future development. From an initial review of the latest proposed FIRM being developed by the Colorado Water Conservation Board's (CWCB) Colorado Hazard Mapping Program (CHAMP), the WWTP site inside the plant access ring road will be outside of the 100-year floodplain, pending any appeals and final approvals. Floodplain information will need to be confirmed during design of the next plant expansion.

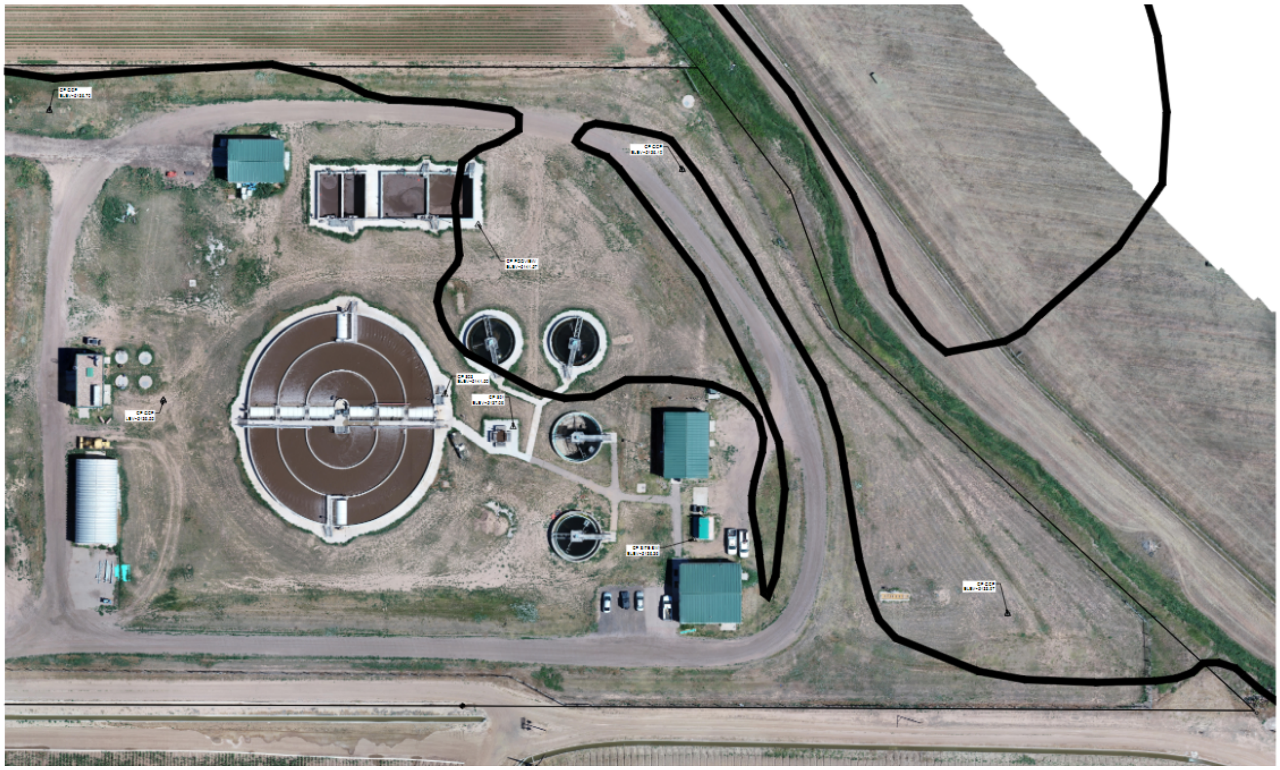


Figure 1-9. The Current 100-year Flood Plain for the Wellington Wastewater Treatment Facility

1.6.1 Headworks

The Headworks Building is a two-story structure built as part of the original construction in 2003. Screening and grit removal equipment are housed within the building. The upper level space is fairly tight for maintenance of equipment and there is not room to add additional capacity for screening and grit removal. The screening equipment has required frequent maintenance in recent years. Specific equipment in the building include:

- One 6-mm mechanical bar screen
- One screenings press
- One bypass 25-mm manual bar rack
- One vortex grit separator
- One grit pump
- One grit cyclone and classifier
- One grit and screenings dumpster
- Automatic composite sampler in a separate room.

1.6.2 Influent Pump Station

The Influent pump station consists of the following:

- Two influent wet wells with two submersible pumps each (four total)
- The four pumps include three duty pumps and one redundant pump
- Capacity 400 gpm per pump

1.6.3 Aeration Basin

The current aeration basin consists of a three-ring Orbal Oxidation Ditch with a total volume of 1 million gallons and eight disc aerators. Influent flow and RAS are conveyed to the outer ring and flow then proceeds to the middle and then the inner ring. There is an internal recycle from the inner-most ring to the outer-most ring at 1,300 gpm which provides some denitrification. The Orbal also has the capability to stepfeed a portion of influent to the internal rings. The side water depth is between 8′ and 9′, with higher levels corresponding to more oxygen transfer via the disc aerators.

Since there is only one aeration basin, and the Orbal design consists of interconnected rings without isolation capability, the basin has not been taken down for inspection or full cleaning since it went on-line in 2003. Over time there has been debris that has accumulated in the basin which cannot be easily cleaned out. The mixed liquor recirculation pump has become plugged at times with rags from the aeration basin. If the Town would like to clean the debris ahead of the Phase 3 construction, the mixed liquor could be pumped from the aeration basin a little at a time, screened, and then returned to the basin. Rain for Rent has a system that may be suitable for this purpose. If the debris in the basin is primarily suspended, a removable net could be installed across the outer channel and removed from the channel periodically to be cleaned of collected debris and returned to service. There is limited access to the inner portions of the Orbal so installation of the net could prove challenging. Another approach would be a remote controlled dredge that would move along the floor of the basin, collect settled debris, and pump it out of the basin. Following the Phase 3 construction of an additional aeration basin, the Orbal basin can be taken completely down for inspection and maintenance which would provide better access, but this would require waiting several years.

1.6.4 Secondary Clarifiers

The secondary clarifier facilities include the following:

- Fed by four-gate splitter box
- Two older 35′ diameter center-fed clarifiers
- Two newer 35′ diameter center-fed clarifiers
- A common scum pit and scum pump

The Town has completed renovations on Secondary Clarifier #2 and will complete renovations on Secondary Clarifier #1 in the near future. There is an issue with the coating on the mechanism in the new Secondary Clarifiers #3 and #4. The towbrow suction headers on #3 and #4 are showing signs of corrosion. Algae growth requires weekly cleaning which raises safety concerns for staff. Overall, the clarifiers are serviceable for future use.

1.6.5 RAS/WAS Pumping Station Building

Equipment in the building includes the following:

- Five centrifugal RAS pumps in lower level (including one redundant pump)
- Five progressing cavity WAS pumps in lower level (including one redundant pump)
- One progressing cavity scum pump in lower level interconnected to WAS pumps
- Three blowers (1,900 scfm each – two duty and one redundant) for aerobic digestion on upper level

Most of the pumps in the RAS/WAS Pumping Station Building are from the original 2003 construction. The older pumps remain in fairly good condition given their age. The newer pumps have some alarm features that could be installed (e.g. seal failure alarms) to increase the reliability of the pumping systems. The pump room has access issues due to the current layout of process piping. This has led to safety concerns for staff. The blowers for the aerobic digesters were installed in 2016 and are in good condition, but they do have some recurring maintenance issues.

1.6.6 Aerobic Digesters

The plant has a total of four aerobic digester tanks. A summary of the digester facilities includes:

- Two older digesters 63,000 MG each with course bubble diffusers
- Two newer digesters 130,200 MG each with fine bubble diffusers
- Telescoping decanting mechanisms (one per digester in two of the digesters)
- Digesters are open-top and can be run in parallel or in series via submersible pumps

Improvements to the air piping system such as additional valving are needed to add more operational flexibility. Valves and actuators freeze in the winter. New VFDs and instrumentation were added for the digester aeration in 2019/2020 and plant staff is continuing to optimize digester performance based on this new equipment. The disparate diffusers impact aeration control and should be made to match.

The digesters are typically operated in series with sludge transfer from Tank 4 to 3 to 2 to 1. The performance drops off considerably during the winter. Adding covers to the basins will trap the heat and improve performance. Concrete covers would be the best choice. The digesters were originally built to add concrete covers later. There are some modifications to the sludge piping and valving between the digesters that should be made to improve operability and potentially reduce intermediate pumping. Instrumentation improvements have been recently made and some improvements for a more permanent installation are needed. Also, modifications to the decanting system, such as adding motor-operated telescopic valves, could improve decant performance.

1.6.7 Sludge Dewatering Building and Sludge Drying Pad

Sludge dewatering is accomplished by a belt filter press located in the Sludge Dewatering Building. Dewatering facilities include:

- One progressing cavity transfer pump with shelf spare
- One DynaBlend emulsion polymer feed system with tote polymer storage
- One 1-m belt filter press

- 45,000 sq. ft. sludge air drying pad

The belt filter press has been very reliable, but there is only one unit so no redundancy exists. The belt filter press is only operated two days per week and produces solids ranging from 15 to 17 percent solids concentration. The dewatered sludge cake is discharged onto the floor of the building and then loaded with a Bobcat skid-steer loader.

The current air drying operation reduces the volume of solids that must be hauled off site and associated hauling costs. The process takes as little as 2 weeks in the summer and up to 6 months in the winter to achieve 70 to 90 percent solids concentration and Class B requirements per USEPA 40 CFR Part 503 regulations. The process is weather-dependent during all portions of the year. The existing auger attachment on the Bobcat loader is limiting and can only push through 6-12 inches windrow height.

1.6.8 Administration/UV Building

The Administration/Ultraviolet (UV) Building houses the Laboratory, Control Room, restroom, and UV disinfection process. One half of the UV system was installed in 2003 and the other half was installed a few years later. The operator interface panel has lost some functionality over the years and the technology is aging. Although the UV system meets current disinfection limits for current peak flows, there is not room for further expanding the system capacity in the building. UV cassettes must be removed manually which leads to slipping concerns for staff. The UV system consists of:

- Four horizontal UV lamp banks
- Two UV channels

The Laboratory is of sufficient size to meet current lab needs and provides for some additional countertop space if desired. The Laboratory is sometimes used as a meeting room as there is no meeting room in the building. There is no office and administrative space in the building either (just a control room), but if a new UV system is constructed to meet future flows, the current 34' x 17' space could be potentially utilized for a meeting/break room or offices. The grassy area northeast of the existing building could be utilized for a new building for other personnel space (offices, locker room, break room, etc.).

With increased development in the Town upstream of the plant and increased stormwater runoff to Boxelder Creek, consideration should be given for future rises in the floodplain when siting the new UV Disinfection Building and setting the finished floor elevation.

2. Liquid Treatment Alternatives

The purpose of this section is to summarize the results of the evaluation and screening of liquid treatment/nutrient removal alternatives.

2.1 Calibration of Wastewater Process Model

A steady-state process model of the Town of Wellington's Wastewater Treatment Plant (WWTP) was developed, calibrated and validated using Jacobs' Activated Sludge Model-based Professional Process Development and Dynamics (Pro2D$_2$) tool. Modules within Pro2D$_2$ use stoichiometric and kinetic relationships that track utilization and conversion of various compounds, nutrients, and microorganisms. This proven design tool provides an overall mass balance throughout the facility based on influent wastewater characteristics, physical unit process configuration, and operational criteria unique to the Wellington WWTP. A well-calibrated model is valuable for developing targeted alternative analyses for facility improvements. As part of this masterplan a special sampling campaign was conducted in October 2019 on the influent and process performance. Historical data and insights from the facility staff were then used to tune the model and confirm its predictive effectiveness. The process flow diagram used in the process model is given in Figure 2-1 and a sample of the calibration between modelled and observed performance is given for Mixed Liquor concentration in Figure 2-2.

Additional information on the calibration of the Wellington WWTP Pro2D2 model can be found in Appendix A.

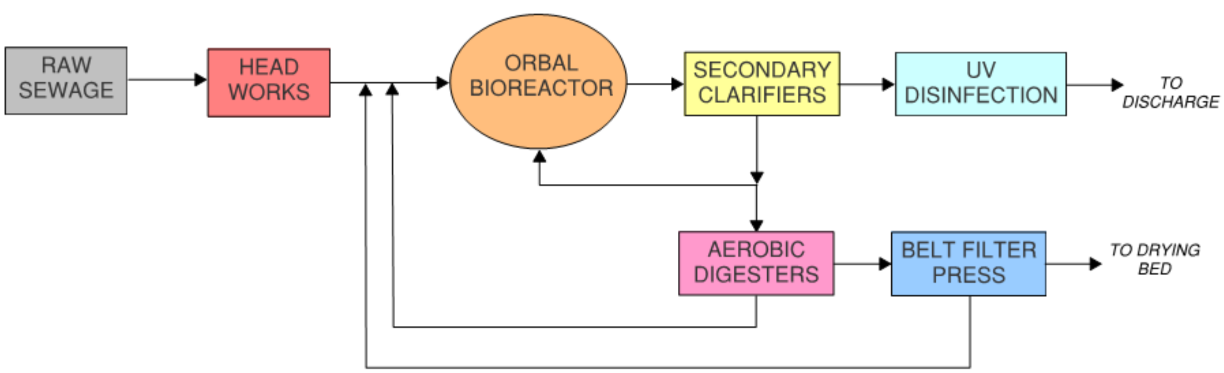


Figure 2-1. Wellington Treatment plant Process Flow Diagram.

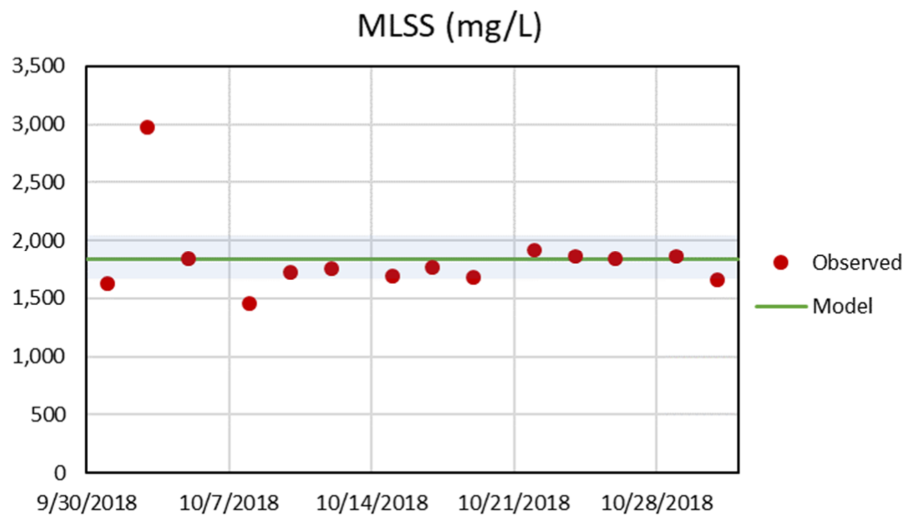


Figure 2-2. Measured vs Modelled Calibration

The measured performance (red dots) is plotted against the modeled values (green line) for Mixed Liquor concentration. The light blue band represents acceptable error (+/- 10% of the mean).

2.2 Existing Capacity

2.2.1 Headworks and Influent Pump Station

The existing Headworks Building was part of the original construction and was meant to serve an overall treatment plant capacity of 0.9 to 1.2 MGD maximum month average daily flow. The headworks is a compact facility and lacks mechanical redundancy. For the screening process there is a coarse manual bar rack in a bypass channel which, when put into use for maintenance on the single mechanical bar screen, has allowed debris to pass through the Headworks Building and into the Aeration Basin. Although the Headworks Building has served the plant through the Phase 2 doubling of plant capacity in 2016, the facility will not be of adequate size to serve future phases and will need to be replaced with a larger facility at the next expansion.

The Influent Pump Station is sized for current peak flows in terms of pumping capacity and wetwell volumes. It is unlikely that larger pumps to meet future peak flows can be installed in the existing wet wells. It is recommended that a new influent pump station be built with the new headworks for the Phase 3 expansion.

2.2.2 Secondary Treatment

The current secondary treatment system was expanded in 2016 from 2 to 3 Orbal rings and 2 additional secondary clarifiers for a total of 4. The calibrated Pro2D2 model along with assessments from staff were used to evaluate the capacity of the existing facilities in order to prioritize infrastructure improvements. The current flow and load capacity (equivalent to maximum month conditions and minimum month temperature) of the secondary system is given in Table 2-1. The BOD load is essentially equal to the rated BOD load capacity of the plant per the discharge permit. The flow capacity is slightly lower than the permit-rated flow capacity; this is merely due to the fact that the influent to the WWTP has increased in strength at a faster rate than the flow has increased. This may be due to community-wide water conservation efforts and the impact of new portions of the collection system with less infiltration and inflow. The limiting factors for the biological treatment are the oxygen transfer capacity and solids loading rate on the secondary clarifiers.

Table 2-1. Capacity of the Existing Secondary System Based on Recent Flow and Loads and Process Modeling

Influent Flow (MGD)	BOD load (lb/d)	TSS load (lb/d)	COD load (lb/d)*	Ammonia load (lb/d)*
0.75	2,677	2,389	6,524	404

**The COD and Ammonia loads are based on ratios to the BOD load during the special sampling campaign in October 2019.*

2.2.3 UV Disinfection

The existing UV disinfection system was installed in one of two UV channels built as part of the original plant construction in 2003. The system was expanded several years later as lamps were installed in the second channel. There is no space left for additional capacity. Peak Hour flow capacity is 3 MGD, which corresponds to an equivalent design (maximum month) flow of 1.08 MGD due to the diurnal nature of wastewater. The flow is projected to exceed the UV system capacity within 5 years.

2.3 Projected Phase Flows and Loads

The flows and loads for future constructions phases were developed based on the projected flows and loads, the capacity evaluation of the existing facilities, and Town input. A balance was targeted between construction costs and expansion frequency, so that just when a construction phase is completed, the Town has breathing room before the next design must begin. The projected phase flows and loads are given in Table 2-2. Phase 3 assumes the Orbal still in operation and one rectangular aeration basin has been constructed. Phase 4 assumes the Orbal still in operation and two rectangular aeration basins have been constructed. Phase 5 assumes 3 rectangular aeration basins and the Orbal decommissioned. Peak Hour for future phases should be reevaluated at a later date as this peaking factor tends to decrease as facilities grow. For the Phase 3 design the future phase peak hour peaking factor was assumed to be constant for conservatism.

Table 2-2. Projected Flows and Loads for Future Phase Capacities

	Influent Flow (MGD)		
	AA	MM	PH
Phase 3	1.56	1.75	4.84
Phase 4	2.46	2.75	*
Phase 5	2.68	3.0	*

**Peak Hour flows of future phases should be evaluated before design of those phases.*
AA = Annual Average; MM = Maximum Month; PH = Peak Hour

Maximum Month	BOD load (lb/d)	TSS load (lb/d)	Ammonia load (lb/d)*	Total Phosphorus load (lb/d)*
Phase 3	6,300	5,700	690	150
Phase 4	9,900	9,000	1,080	230
Phase 5	10,900	9,900	1,180	250

** The current and projected Ammonia and Phosphorus loads are based on ratios to the BOD load during the special sampling campaign in October 2019 and additional data collection in 2020.*

2.4 Evaluation of Liquid Treatment Alternatives

In this section, alternatives are presented to meet the needs of the Phase 3 plant expansion.

2.4.1 Headworks

It is assumed the existing Headworks Building will be decommissioned in Phase 3 due to hydraulic and operational limitations. A new headworks facility shall be constructed which shall include an influent pump station, screening and grit removal. The influent pump station shall be large enough for future pump expansion. There will be three trains for screening: two with mechanical screens and a third with a manual bar screen. The grit removal will consist of two parallel grit separators and classifiers and grit pump redundancy. The access road to the new Headworks Building and around the facility will be paved.

2.4.2 Secondary Treatment

Secondary treatment alternatives are listed below. The existing Orbal oxidation ditch, four secondary clarifiers, and RAS/WAS Pump Station are assumed to remain in service as part of the Phase 3 facilities. Once an additional aeration basin is constructed, then the existing Orbal oxidation ditch can be taken down for maintenance and inspection.

2.4.2.1 Orbal Oxidation Ditch

Assuming similar performance as the existing Orbal oxidation ditch, a 1.4 MG Orbal oxidation ditch with either disc aerators or fine-bubble diffusers is a potential expansion alternative. The advantage of this alternative is that it would use existing technology which could streamline maintenance. The disadvantages of this alternative are that is has a larger footprint due to the shallow operating depth, does not solve any of the current operational issues of the existing Orbal such as poor settling sludge, and provides less process control than other alternatives. This option would likely require either chemical phosphorus removal to comply with Regulation 85 or a newly constructed anaerobic reactor before the existing and new Orbal with rerouted feed and RAS return at the existing. This alternative does not easily lend itself to-further expansions without building another separate basin which would limit space on site for other future construction. A layout of this alternative is presented below in Figure 2-3. Because of the numerous disadvantages, this alternative will not be considered below during the cost comparison.

Figure 2-3. Orbal Oxidation Ditch Expansion Option

The Phase 3 proposed construction is highlighted in teal.

2.4.2.2 Conventional Activated Sludge

The second treatment alternative is a 1.4 MG conventional activated sludge tank with fine-bubble diffused aeration, dedicated anoxic zones, and an internal recycle system. New blowers would be housed in a new RAS / WAS pump station. The advantages to this alternative are:

- A smaller footprint than an Orbal oxidation ditch
- Easy to expand for future phases with common wall construction
- Dedicated anoxic zones reduce the likelihood of filamentous bacteria, thus improving settleability and clarifier capacity
- More process control flexibility for staff

The disadvantage of this alternative is that it adds to the complexity of plant maintenance (additional process equipment to maintain). A layout of this alternative is presented below in Figure 2-4.

Figure 2-4. Conventional Activated Sludge Expansion Option

The Phase 3 proposed construction is highlighted in teal; Phase 4 is highlighted grey, and Phase 5 is highlighted red.

2.4.2.3 Step Feed Activated Sludge

The third alternative builds upon the benefits of the conventional activated sludge alternative with the addition of the ability to step feed influent. A step feed system introduces influent to the aeration basin at multiple points in the basin which spreads out the oxygen demand and reduces overall basin volume. The benefits of this alternative are:

- Even smaller footprint (though with more internal walls)
- Higher solids inventory possible without overloading clarifiers by storing sludge in first pass
- Potential redundancy option with double pass tanks
- Improved denitrification without supplemental carbon
- Greater nutrient removal reliability, higher inventory protects against peak loads and wet weather events

The disadvantage of this alternative is the increased operational complexity compared to the other alternatives. A layout of this alternative is presented below in Figure 2-5.

Figure 2-5. Conventional Activated Sludge with Stepfeed Expansion Option

The Phase 3 proposed construction is highlighted in teal; Phase 4 is highlighted grey, and Phase 5 is highlighted red.

2.4.2.4 Secondary Clarifiers Included in All Alternatives

All secondary treatment alternatives include two new 50' diameter secondary clarifiers and a new corresponding RAS / WAS pump station. These were sized to comply with CDPHE redundancy requirements and accessibility for staff.

2.4.2.5 Phosphorus Management (Biological with Chemical Back Up)

Each evaluation includes the construction of a new anaerobic zone for the existing Orbal to achieve biological phosphorus removal in reference to Regulation 85. This modification will require redirecting the influent and the RAS of the existing Orbal to a new unaerated zone. This could be completed at a stage after the new basins are in service.

A small, back-up chemical phosphorus system is also recommended for all alternatives, as changes in the Wellington collection system could lead to biological phosphorus upsets (e.g. a sudden industry shut down). This would be housed in the new RAS / WAS pump station and deliver ferric or alum on a short-term basis when needed.

2.4.3 Disinfection

A new disinfection system is needed for the Phase 3 expansion to replace the existing UV disinfection system. The new system can be located in a new building on the southeast corner of the site near the effluent Parshall flume. Exact location on the site should consider the floodplain location. UV disinfection has become the standard for wastewater disinfection and is recommended over chlorine disinfection. Either a horizontal or vertical oriented UV system can be designed. A vertical system has the advantage of a smaller footprint and easier lamp replacement, but deeper channel excavation. For the purposes of the master plan, a vertical system has been initially selected.

2.4.4 Administration and Maintenance Facility

Additional administration and maintenance space are necessary for the expanded facility. As previously noted, although some space is available for meeting and/or office space in the existing Administration/UV Building, a new office building could be constructed northeast or east of the existing building. The existing UV facility space in the Administration Building can be renovated for staff use.

The existing Quonset hut could be converted into a maintenance shop. Improved lighting, power, and HVAC would be required to create a more usable space.

2.4.5 Construction Cost Analysis and Noncost Discussion

The construction costs break down for each structure and major equipment systems for the liquid stream improvement alternatives are provided in Table 2-3. These individual costs incorporate the following markups

- construction markups
- contractor markups
- contingency
- non-construction costs.

Construction markups total to 12% and include sitework (which would include paving access roads), SCADA changes, and Yardwork (electrical and piping). Contractor markups total to 22% and include contractor overhead, profit, and mobilization / demobilization. The contingency used for this Level 5 estimate was 30%. Non-construction costs total to 17% and include legal, administration, permitting, engineering design and engineering services during construction.

These costs reflect systems that meet redundancy criteria required by CDPHE.

Table 2-3. Liquids Streams Alternatives Construction Costs

	Conventional Activated Sludge (CAS)	CAS with Stepfeed
Phase 3		
New Headworks building, Screening, Grit Removal	$10,350,000	$ 10,350,000
New Influent Pumps	$ 2,300,000	$ 2,300,000
Influent Splitter Structure	$ 490,000	$ 490,000
New Aeration Basin	$ 6,490,000	$ 5,400,000
Secondary Splitter Structure	$ 480,000	$ 480,000
New Secondary Clarifiers (2)	$2,220,000	$ 2,220,000
New RAS, WAS Pumps and Pump Station	$ 2,700,000	$ 2,700,000
New Blowers	$ 3,740,000	$ 3,740,000
New Chemical Systems	$ 300,000	$ 300,000
UV building	$ 1,750,000	$ 1,750,000
Office Building	$ 330,000	$ 330,000
Admin Building Renovation	$ 60,000	$ 60,000
Maintenance Repurpose	$ 100,000	$ 100,000
Modifications to existing Orbal (new Anaerobic Zone)	$ 580,000	$ 580,000
New Launder Covers for Existing Clarifiers (4)	$ 80,000	$ 80,000
Landscaping with NonPotable Water System	$ 100,000	$ 100,000
Liquids Phase 3 Total Construction Cost w/ Markups	**$32,070,000**	**$30,980,000**

In addition to the cost comparison, the operational and maintenance pros and cons of the evaluated alternative liquid stream improvements are summarized in Table 2-4.

Table 2-4. Pros and Cons of Liquids Stream Alternatives

Liquid Alternative Pros and Cons	Pros	Cons
Conventional Activated Sludge	• Simpler feed control	• 20% more expensive (just basin construction)
Conventional Activated Sludge with Stepfeed ***(Recommended)***	• Smaller basin volume, footprint • Less expensive • More effective total nitrogen removal • More robust during wet weather events	• More complex feed control • More equipment due to two passes (mixers, internal recycle pumps)

2.4.6 Recommended Liquid Treatment Alternative

Based on the lower cost estimates, smaller estimated footprint, and higher operational efficiency, the ***Conventional Activated Sludge with Stepfeed*** option is recommended for the Phase 3 project.

3. Solids Treatment Alternatives

The purpose of this section is to summarize the results of the evaluation and screening of solids treatment alternatives and offsite sludge reuse/disposal alternatives.

3.1 Existing Capacity

The current solids stabilization system (aerobic digestion and air drying pad) provides sufficient tank volume and surface area to reliably meet Class B standards. Additional solids loading beyond the current plant capacity may impact the ability to meet Class B on site. Additional solids stabilization is currently performed offsite as noted in the section below.

The existing belt filter press is only operated two days per week so additional capacity is available for processing the projected future solids for the Phase 3 loads.

3.2 Biosolids Disposal Alternatives

Currently the plant produces Class B biosolids. The air-dried solids are hauled from the plant by McDonald Farms to the private A-1 Organics composting facility near Keenesburg. A-1 Organics produces a Class A product which it markets. A-1 Organics does not require that Class B standards are met prior to hauling to their compost site nor do they charge an additional tipping fee for sub-Class B sludge. Another disposal alternative for Class B biosolids is to apply directly to private agricultural land via a contract hauler.

For future biosolids disposal, having Class A facilities on site would increase the Town's options. On-site composting or a conversion of the aerobic digesters to autothermal thermophilic aerobic digestion (ATAD) are examples of processes to produce Class A solids. Composting would be performed on land that would otherwise be needed for future air drying and would involve storage of other amendment material which would take up additional space on site. ATAD solids could still be dried on a drying pad to reduce solids volume and hauling cost. An ATAD alternative is evaluated below. Having Class A biosolids produced on site would allow the Town more flexibility in disposal options including the potential for use on nearby agricultural land, but would require additional program management by the Town.

3.3 Evaluation of Solids Treatment Alternatives

Solids treatment alternatives are evaluated in this section for digestion, dewatering, and air drying.

3.3.1 Digestion

Alternatives for digestion are presented in this section. The two alternatives reviewed were conventional aerobic digestion (current process) and autothermal thermophilic aerobic digestion (ATAD). Other alternatives that were considered, but not selected for additional analysis were lime stabilization (uncommon in Colorado and chemically intensive) and anaerobic digestion (not appropriate for this size of plant).

3.3.1.1 Conventional Aerobic Digestion

To maintain the current level of Class B solids with conventional aerobic digestion, a minimum of 20 days of digestion detention time should be provided at 20 degrees C (400 degree-days) along with the air drying process following dewatering. The existing digesters provide 250 degree-days for the current plant capacity and with uncovered digester tanks. To achieve a minimum of 400 degree-days, additional tank volume of 600,000 gallons is required. Also, the existing and new digesters should be covered with concrete covers to maintain 20 degrees C in winter. Should the combination of aerobic digestion and air drying fall short of Class B criteria, A-1 Organics will still take the solids to incorporate into their composting program. Stabilization in the aerobic digesters should be achieved to avoid excessive odor generation on the air drying pad at a minimum.

Additional blower capacity will be required for the new tank volume. The new blowers will be housed in a new blower building near the new tanks. The existing blowers for the existing tanks should be reviewed during design to determine if they are to remain in use and reside in the existing RAS/WAS Pumping Building. Telescoping decanting valves with motorized operators will be installed in both the new and existing tanks. Submersible sludge transfer pumps will be installed in the new tanks. Digester instrumentation should be reviewed for the existing and new digesters to ensure optimal performance.

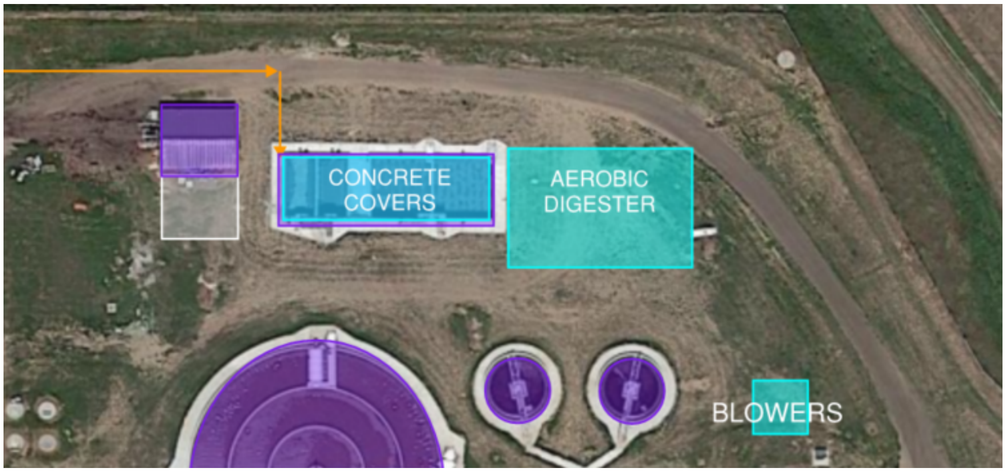


Figure 3-1. Aerobic Digestion Expansion Option

3.3.1.2 Autothermal Thermophilic Aerobic Digestion (ATAD)

ATAD systems operate at higher temperatures (50 to 60 degrees C) than conventional aerobic digestion which reduces the required SRT, produces Class A solids, and provides a greater reduction in solids. ATAD systems typically produce drier dewatered cake than conventional aerobic digestion also.

Current ATAD systems are termed “second generation” ATAD. “Second generation” ATAD systems can produce Class A biosolids. The “second generation” ATAD includes process modifications over the “first generation” ATAD systems such as better mixing and process control that have resulted in higher volatile solids reduction (VSR), lower product volume, less odor generation, and less sidestream impacts. Although ATAD has a sole provider, the “second generation” ATAD process has been installed at over 62 installations in the United States including the following Colorado installations: Edwards, Fruita, St. Vrain Sanitation District, and South Fort Collins Sanitation District. A picture of an example installation is shown in Figure 3-2.

Figure 3-2 Pump Gallery of an ATAD Installation in Colorado

The "second generation" ATAD system involves a two-stage system. The first (ThermAer™) stage is where the thermophilic activity and significant VSR occurs. Foam is controlled with hydraulic foam suppression nozzles. The second (MesoAer) stage provides sidestream treatment to remove nitrogen via nitrification and denitrification. The aerated MesoAer stage also provides additional VSR, solids cooling to reduce dewatering polymer requirements, and dewatering feed equalization. All tanks require fixed covers to retain heat and control odors. The "second generation" ATAD system typically requires a total of approximately 23 days of SRT (13 days for first stage process tanks and 10 days for second stage). As part of the "second generation" improvements, a two-stage odor control system is provided for effective odor control. The two stages include a water scrubber unit followed by a biofilter. A major purpose of the first stage scrubber, in addition to removing 70 to 80 percent of the ammonia, is to cool the hot air so the biological activity in the biofilter is not harmed.

The ATAD first and second stage reactors can be retrofit into one of the larger and one of the smaller existing digester tanks taking up half of the existing volume. The remainder of the tanks could be reserved for Phase 4. ATAD equipment (pumps, blowers, and heat exchangers) would be housed in a building adjacent to the reactors. Additionally, sludge thickening is required to approximately 5 to 6 percent prior to introduction of solids to the ATAD first stage reactor. For this evaluation, rotary drum thickeners (RDTs) are assumed as the sludge thickening technology. Odor control is also required for scrubbing the air in the ATAD reactors. A water scrubber and a biofilter are assumed for this purpose.

Costs for an ATAD system will be higher than for expanding the aerobic digesters (see cost evaluation below). A phased approach to convert to ATAD in the future could also be considered.

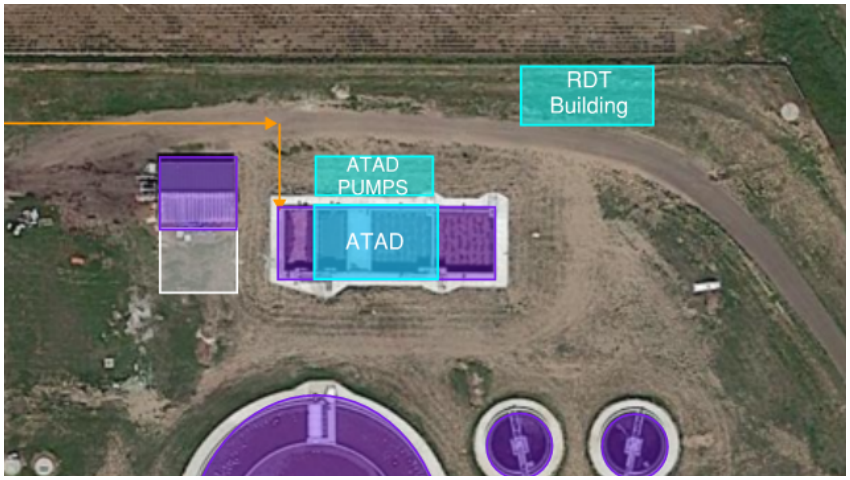


Figure 3-3. ATAD Option

3.3.2 Sludge Dewatering

As noted previously, the existing 1-meter belt filter press provides sufficient capacity for Phase 3 loads. However, the backup for the belt filter press is liquid sludge hauling which is expensive. The existing Dewatering Building is not large enough to install another belt filter press, but a centrifuge (which has a smaller footprint) could be installed in the existing building.

To alleviate the issue of sludge accumulating on the floor of the Dewatering Building, a belt conveyor could possibly be installed to carry dewatered cake through the south wall of the building to a dump truck parked outside.

3.3.3 Air Drying

The liquid treatment expansion for Phase 3 will be located where the current air drying pad is. Additional air drying pad space can be located to the west as shown on Figure 3-4. Approximately 114,000 sq ft of asphalt drying pad space will be required for windrowing the Phase 3 loads. A more durable concrete pad could be installed instead of asphalt, but at a 20% greater cost. A larger auger and front end loader would provide the ability to move taller windrows of drying solids.

Figure 3-4. Proposed Drying Bed Relocation

A cover over the air drying pad would improve the reliability of the process in inclement weather (rain). The enclosure would be open at the ends and enclosed on the sides such as the example in Figure 3-5. Due to the high cost and uncertain benefit, a cover was costed for only 50% of the proposed new drying pad area (this area is teal in Figure 3-4). Wind could be a potential concern at the Wellington WWTP site.

Figure 3-5. Example of Cover Structure

3.3.4 Construction Cost Evaluation and Non-cost Discussion

The construction costs break down for each structure and major equipment systems for the liquid stream improvement alternatives are provided in

Table 3-1. These individual costs incorporate the following markups

- construction markups
- contractor markups
- contingency
- non-construction costs.

Construction markups total to 12% and include sitework (which would include paving access roads), SCADA changes, and Yardwork (electrical and piping). Contractor markups total to 22% and include contractor overhead, profit, and mobilization / demobilization. The contingency used for this Level 5 estimate was 30%. Non-construction costs total to 17% and include legal, administration, permitting, engineering design and engineering services during construction.

These costs reflect systems that meet redundancy criteria required by CDPHE.

Table 3-1. Solids Streams Alternatives Construction Costs

	Expansion of Existing Aerobic Digesters	ATAD
Phase 3		
Add Covers to existing aerobic digesters	$400,000	$200,000
Aerobic Digester expansion & Blower Building	$6,280,000	$0
ATAD retrofit & Building	$0	$7,290,000
Rotary Drum Thickeners & Building	$0	$2,390,000
New Drying Pad	$1,100,000	$1,100,000
50% Cover for Drying Pad	$0	$1,650,000
New Centrifuge	$1,090,000	$1,090,000
Solids Phase 3 Total Construction Cost w/ Mark-ups	**$8,870,000**	**$13,720,000**

In addition to the cost comparison, the operational and maintenance pros and cons of the evaluated alternative solids stream improvements are summarized in Table 3-2.

Table 3-2. Pros and Cons of Solids Stream Alternatives

Liquid Alternative Pros and Cons	Pros	Cons
Expansion of Existing Aerobic Digesters ***(Recommended)***	• Takes advantage of stabilization via drying pad and composting facility • Simpler operation	• Larger footprint • Higher sludge production
ATAD	• Reduced volume and dryer solids production • Higher Biosolids quality (Class A) • Uses existing digester volume	• Increased operational and maintenance demands on staff • 50% more expensive

3.3.5 Recommended Solids Treatment Alternative

Although the ATAD option has several unique advantages, the capital cost is significantly higher. In the near term the ***expansion of existing aerobic digesters*** is recommended. These basins could be converted into an ATAD facility in the future such as the next expansion of the plant if a change in process was desired.

The drying bed cover adds $1.65 million to the project costs. The cover may not be worth this additional cost as composting / stabilization is completed offsite. This cost is not included in the final recommended plan in Table 4-1 of the next section.

4. Implementation Plan

4.1 Overall Plant Expansion Recommendation

A summary of the alternative combinations is given in Table 4-1 below. The overall plant expansion recommendation is to implement the Conventional Activated Sludge with Step Feed liquid treatment option along with the conventional aerobic digestion solids treatment option.

Table 4-1. Summary of Alternative Combination Construction Costs

Liquid and Solids Cost alternatives	Conventional Activated Sludge	Conventional Activated Sludge with Stepfeed
Expansion of Existing Aerobic Digesters	$40,940,000	**$39,850,000** ***(Recommended)***
ATAD	$45,790,000	$44,700,000

The Non-Construction costs include legal, administration, permitting, engineering design and engineering services during construction. The non-construction cost associated with the recommended alternative is $5.8 million.

An aerial view of the proposed Phase 3 project is shown in Figure 4-1.

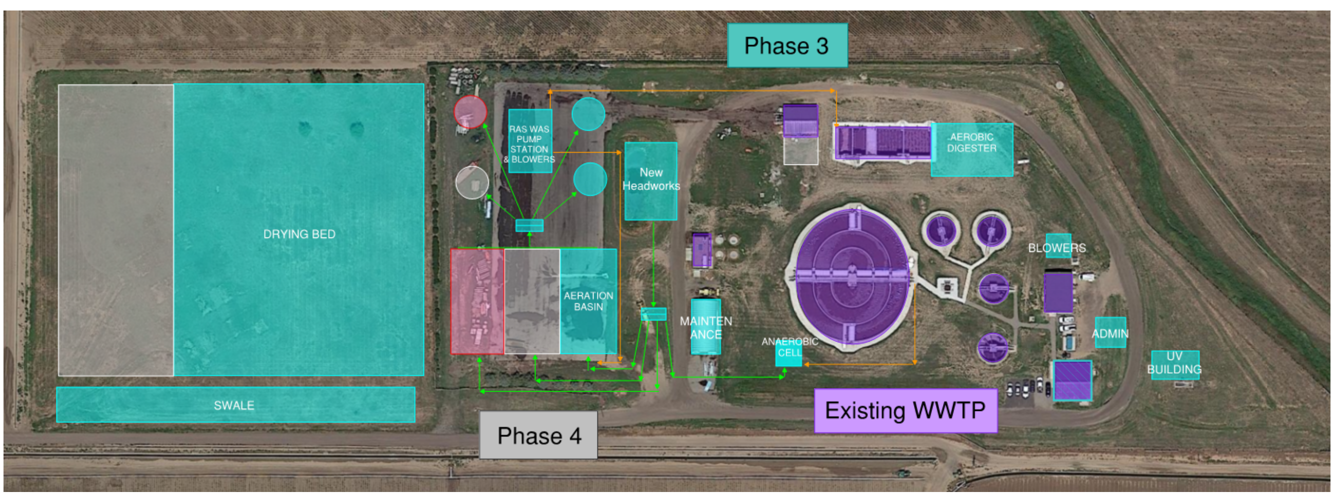


Figure 4-1. Layout of Recommended Alternatives.

The Phase 3 proposed construction is highlighted in teal; Phase 4 is highlighted grey, and Phase 5 is highlighted red. Existing facilities are highlighted purple. The administration building is purple with teal hatches to indicate an existing facility that will be upgraded in Phase 3.

4.2 Staffing Expansion

Whichever alternatives are selected, it will be important for the Town of Wellington to be diligent in operations training. It is expected that as the town grows, the staffing at the WWTP will need to increase to handle increased size and variety of processes with increased Instrumentation and Controls requirements. Pretreatment will require more attention as well.

4.3 Construction Phasing Options

The expansion of the liquid treatment facilities is the most crucial need for the plant given the projected influent flows and loads. Given that sludge stabilization is performed partially on the plant site and partially offsite at the A-1 Organics composting facility, the bulk of the solids improvements could be delayed. Installing covers for the existing digester tanks could allow for higher average operating temperatures and increased digester performance for the next 3-5 years. Any additional stabilization required due to reduction in stabilization performance at the plant could be accomplished at the A-1 Organics facility in the short term. With this approach, the solids improvements could be delayed until after the liquid treatment improvements are constructed if desired.

4.4 Project Financing and Regulatory Approval Steps

The Town is considering financing the project with a State Revolving Fund (SRF) loan through the Colorado Water Resources and Power Development Authority. There are a number of requirements for the loan program including technical, financial, and environmental reviews. Additionally, the CDPHE requires several permitting and design review steps for a wastewater treatment plant expansion. The various major agency review steps are listed below.

One alternative for financing that could be investigated is the Water Infrastructure Finance and Innovation (WIFIA) program. WIFIA features low interest rates and a deferment of payments option for up to five years after substantial completion of the project. WIFIA is typically utilized for projects of over $100M and is a competitive process.

4.4.1 CDPHE Preliminary Effluent Limit Request

Preliminary Effluent Limits (PELs) are the first step in the engineering review process with CDPHE. The Site Location Approval documents and the initial project design are based upon the PELs.

4.4.2 North Front Range Water Quality Planning Association (NFRWQPA) Utility Plan Update

The NFRWQPA is a local reviewer of wastewater treatment projects in association with CDPHE's review process. Prior to review of Site Location Approval documents by NFRWQPA, the Town's wastewater utility plan must be updated.

4.4.3 CDPHE Site Location Approval

Site Location Approval is part of the design review process by CDPHE and is typically completed during the preliminary design stage and includes an engineering report outlining overall project features and specifics of the plant site. Site Location Approval must be achieved prior to submitting an SRF loan application.

4.4.4 CDPHE Process Design Report (PDR)

The PDR is a more detailed design report typically completed at the midpoint of design. The project design is described and compared to design guidelines from CDPHE. The PDR must be submitted to CDPHE prior to submitting an SRF loan application.

4.4.5 SRF Project Needs Assessment (PNA)

The PNA describes the need and design basis for the project. The PNA is one of the SRF requirements that must be completed prior to submitting an SRF loan application.

4.4.6 SRF Environmental Determination

The Environmental Determination must be completed as part of the SRF loan process. Various agencies must be consulted in regard to environmental impacts of the project.

4.4.7 SRF Loan Application

Once the Site Location Approval is achieved with CDPHE, the PDR is submitted, and the PNA is accepted, then the loan application can be submitted.

4.4.8 CDPHE Plans and Specifications Approval

A review and approval of plans and specifications must be completed by CDPHE prior to loan execution.

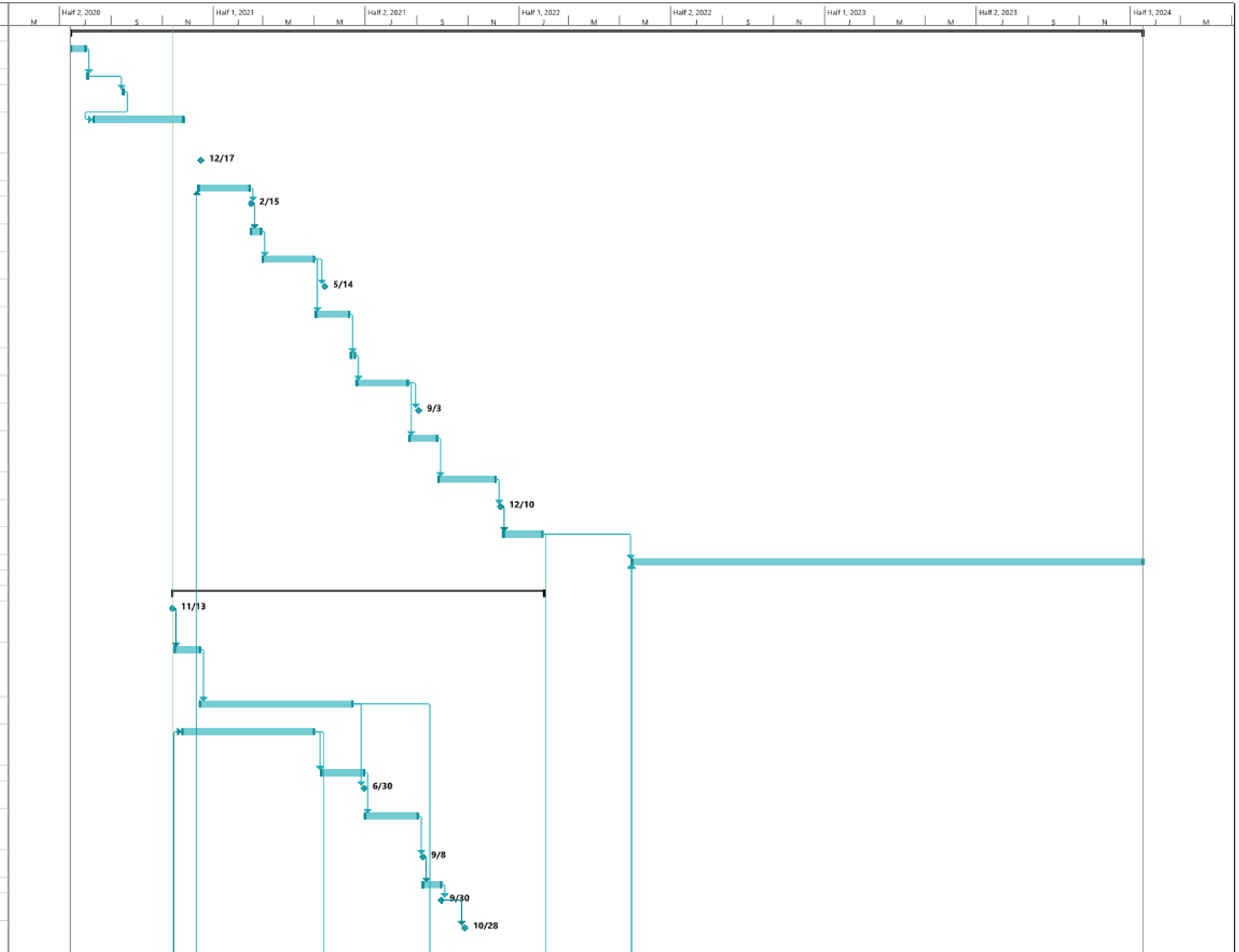
4.5 Implementation

A summary of the final recommendations and costs is given in Table 4-2. The implementation schedule for the plant expansion is presented in Figure 4-1 below with construction being completed by early 2024. To meet the schedule, design activities will need to commence in December 2020 with design, agency review, and loan execution completed by May 2022.

Table 4-2 Final Costs of Recommended Liquids and Solids options

Phase 3	Conventional Activated Sludge (CAS) + Expanded Aerobic Digesters
New Headworks building, Screening, Grit Removal	$10,350,000
New Influent Pumps	$ 2,300,000
Influent Splitter Structure	$ 490,000
New Aeration Basin	$ 6,490,000
Secondary Splitter Structure	$ 480,000
New Secondary Clarifiers (2)	$2,220,000
New RAS, WAS Pumps and Pump Station	$ 2,700,000
New Blowers	$ 3,740,000
New Chemical Systems	$ 300,000
UV building	$ 1,750,000
Office Building	$ 330,000
Admin Building Renovation	$ 60,000
Maintenance Repurpose	$ 100,000
Modifications to existing Orbal (new Anaerobic Zone)	$ 580,000
New Launder Covers for Existing Clarifiers (4)	$ 80,000
Landscaping with NonPotable Water System	$ 100,000
Add Covers to existing aerobic digesters	$400,000
Aerobic Digester expansion & Blower Building	$6,280,000
New Drying Pad	$1,100,000
New Centrifuge	$1,090,000
Phase 3 Total Construction Cost w/ Markups	**$39,850,000**

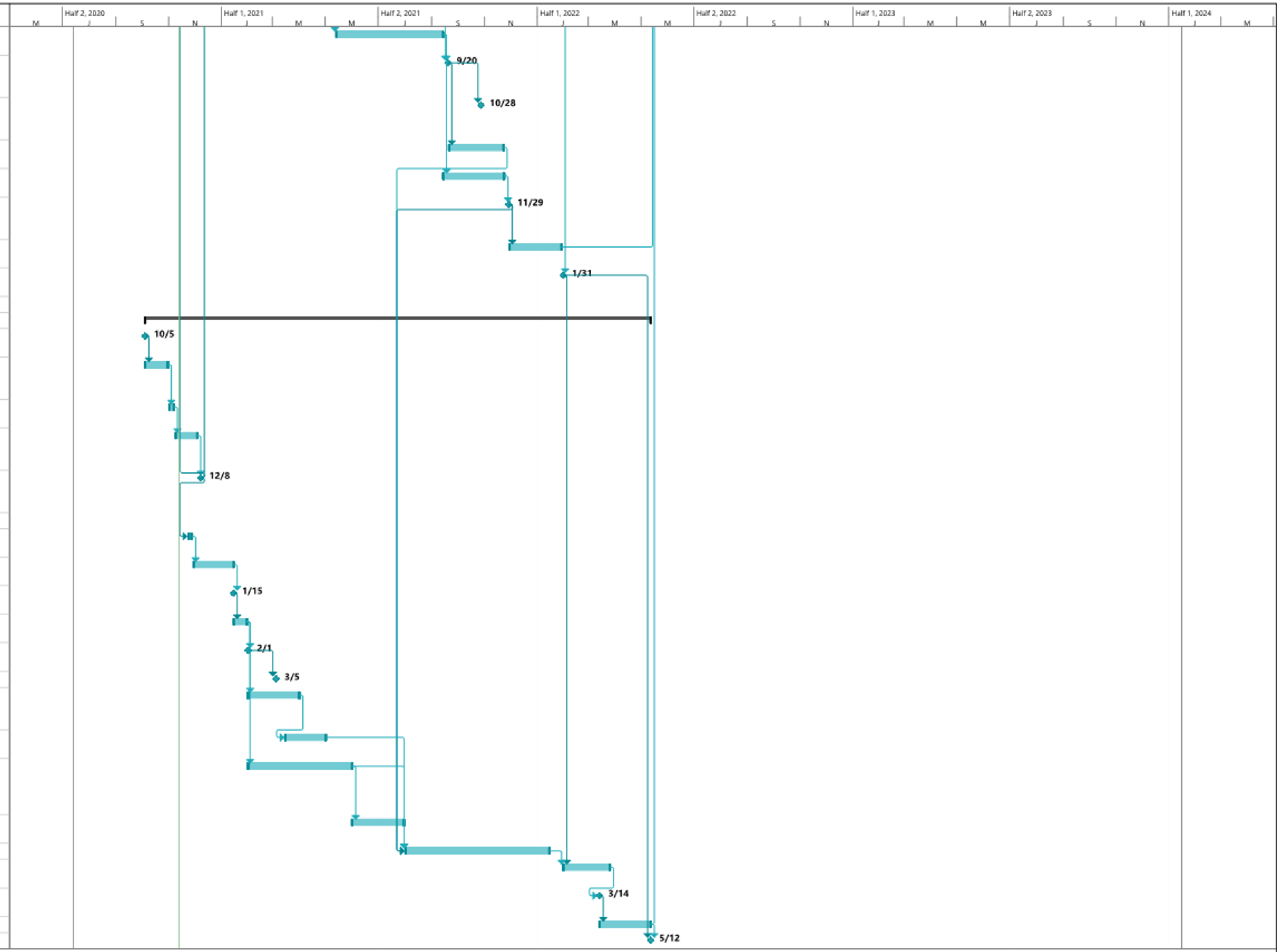
ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors
1			**Engineering/Construct**	**914 days**	**Wed 7/15/20**	**Mon 1/15/24**	
2			Deliver draft WWTP portion of Master	13 days	Wed 7/15/20	Fri 7/31/20	
3			Workshop with Tow	1 day	Mon 8/3/20	Mon 8/3/20	2
4			Town Board Presentation	1 day	Tue 9/15/20	Tue 9/15/20	3
5			Finalization of WWTP porton of Master Plan	77 days	Tue 8/11/20	Wed 11/25/20	4
6			Design Kickoff Meeting	0 days	Thu 12/17/20	Thu 12/17/20	
7			10% Design	45 days	Mon 12/14/2	Fri 2/12/21	48
8			Workshop to Present 10% Design	0 days	Mon 2/15/21	Mon 2/15/21	7
9			Town Confirmation of Design Definition	10 days	Mon 2/15/21	Fri 2/26/21	8
10			Submit 30% Design to Town and CMAR	45 days	Mon 3/1/21	Fri 4/30/21	9
11			30% Design Review Workshp	0 days	Fri 5/14/21	Fri 5/14/21	10
12			Complete Review of 30% Design and CMAR Cost	30 days	Mon 5/3/21	Fri 6/11/21	10
13			Potential Hold for Receipt of PELs	5 days	Mon 6/14/21	Fri 6/18/21	12
14			Submit 60% Design to Town and CMAR	45 days	Mon 6/21/21	Fri 8/20/21	13
15			60% Design Review Workshop	0 days	Fri 9/3/21	Fri 9/3/21	14
16			Complete Review of 60% Design and CMAR GMP Cost	25 days	Mon 8/23/21	Fri 9/24/21	14
17			Submit 90% Design to Town and CMAR	50 days	Mon 9/27/21	Fri 12/3/21	16
18			90% Design Review Workshop	0 days	Fri 12/10/21	Fri 12/10/21	17
19			Submit 100% Design to Town and	35 days	Mon 12/13/21	Fri 1/28/22	18
20			Construction Phase	436 days	Mon 5/16/22	Mon 1/15/24	19,40,63
21							
22			**Regulatory**	**316 days?**	**Fri 11/13/20**	**Mon 1/31/2**	
23			Submit Preliminary Effluent Limit Application Form	0 days	Fri 11/13/20	Fri 11/13/20	
24			Review by CDPHE and Submit Preliminary Effluent Limit and Design	22 days	Mon 11/16/20	Tue 12/15/20	23
25			CDPHE Processing of PELs	130 days	Wed 12/16/20	Tue 6/15/21	24
26			Develop and Submit Draft Utility Plan Update to	113 days	Wed 11/25/20	Fri 4/30/21	48
27			Update Draft Utility	37 days	Mon 5/10/21	Tue 6/29/21	26
28			Submit Draft Utility Plan Update to	0 days	Wed 6/30/21	Wed 6/30/21	25
29			Review of Utility Plan by NFRWQPA review committee	45 days	Thu 7/1/21	Wed 9/1/21	27
30			Review Meeting with NFRWQPA	0 days	Wed 9/8/21	Wed 9/8/21	29
31			Utility Plan Edits	16 days	Wed 9/8/21	Wed 9/29/21	30
32			Resubmit Utility Plan for Backcheck	0 days	Thu 9/30/21	Thu 9/30/21	31
33			Utility Plan Approval at Monthly Meeting	0 days	Thu 10/28/21	Thu 10/28/21	32



Project: Wellington WWTP Imp
Date: Fri 11/13/20

Task	Summary	Inactive Milestone	Duration-only	Start-only	External Milestone	Manual Progress	
Split	Project Summary	Inactive Summary	Manual Summary Rollup	Finish-only	Deadline		
Milestone	Inactive Task	Manual Task	Manual Summary	External Tasks	Progress		

ID	Indicators	Task Mode	Task Name	Duration	Start	Finish	Predecessors
34			Develop Site Application	87 days	Fri 5/14/21	Mon 9/13/21	26
35			Submit Site Application to NFRWQPA and	0 days	Mon 9/20/21	Mon 9/20/21	25,34
36			Site Application Approval at NFRWQPA Monthly	0 days	Thu 10/28/21	Thu 10/28/21	35
37			Site Application Review and	45 days	Tue 9/21/21	Mon 11/22/21	35
38			Develop Process Design Report	51 days	Tue 9/14/21	Tue 11/23/21	34
39			Submit Process Design Report to CDPHE	0 days	Mon 11/29/21	Mon 11/29/21	38
40			PDR Review and Approval by CDPHE	44 days	Tue 11/30/21	Fri 1/28/22	39
41			Plans and Specifications	0 days	Mon 1/31/22	Mon 1/31/22	19
42							
43			**Funding**	**418 days?**	**Mon 10/5/2(**	**Thu 5/12/22**	
44			Issue RFQ for Engineering	0 days	Mon 10/5/20	Mon 10/5/20	
45			Receive SOQs for Engineering Services	20 days	Mon 10/5/20	Fri 10/30/20	44
46			Engineering Firm Selection	5 days	Mon 11/2/20	Fri 11/6/20	45
47			Finalization of Contract for Design and Construction	19 days	Mon 11/9/20	Thu 12/3/20	46
48			Town Board Meeting Approval of Engineering	0 days	Tue 12/8/20	Tue 12/8/20	47
49							
50			SRF Prequalification Form	4 days	Tue 11/24/20	Fri 11/27/20	48
51			Prequalification SRF Agency Review	34 days	Mon 11/30/20	Thu 1/14/21	50
52			Preapplication Meeting	0 days	Fri 1/15/21	Fri 1/15/21	51
53			Prequalification Review Letter from	11 days	Fri 1/15/21	Fri 1/29/21	52
54			Advertisement for Public Meeting	0 days	Mon 2/1/21	Mon 2/1/21	53
55			Public Meeting	0 days	Fri 3/5/21	Fri 3/5/21	54
56			Project Needs Assessment Submission	43 days	Mon 2/1/21	Wed 3/31/21	53
57			PNA Review by CDPHE	35 days	Tue 3/16/21	Sat 5/1/21	56
58			Environmental Review including Review Agency Referrals	86 days	Mon 2/1/21	Mon 5/31/21	53
59			EA Review by WAPA, etc.	44 days	Tue 6/1/21	Fri 7/30/21	58
60			Loan Application	120 days	Mon 8/2/21	Fri 1/14/22	37,39,57,58
61			WAPA Review of Loan Application	40 days	Mon 1/31/22	Fri 3/25/22	60,41
62			WAPA Board Meeting - Loan	0 days	Mon 3/14/22	Mon 3/14/22	61
63			Loan Execution	43 days	Mon 3/14/22	Wed 5/11/22	62
64			Fund Distribution	0 days	Thu 5/12/22	Thu 5/12/22	41,63



Project: Wellington WWTP Imp
Date: Fri 11/13/20

Task	Summary	Inactive Milestone	Duration-only	Start-only	External Milestone
Split	Project Summary	Inactive Summary	Manual Summary Rollup	Finish-only	Deadline
Milestone	Inactive Task	Manual Task	Manual Summary	External Tasks	Progress
Manual Progress					

Page 2

Figure 4-2: Project Implementation Schedule

Appendix A. Process Model Calibration Memo

TECHNICAL MEMORANDUM

Wellington Sanitary Sewer System and WWTP Master Plan

Model Calibration and Validation Report

PREPARED FOR:	The Town of Wellington
PREPARED BY:	Jacobs
PROJECT:	WXXY5100
DATE	February 3, 2020

Objective

The objective of this memo is to present the quality of the calibrated and validated model for the Town of Wellington, CO Wastewater Treatment Plant to ensure confidence in the model predictions for the alternative analysis.

Approach

The approach taken for the Wellington WWTP model was to both calibrate and validate a process model. The model is calibrated first, based on wastewater data specific to the Wellington plant. Following model calibration, the model is tested on another set of actual data to see how well the predicted results match actual results (i.e. effluent values). This testing of the model is referred to as validation.

A calibrated model is valuable to developing targeted alternative analyses for facility improvements. For the purpose of model calibration, historical influent and operational data were collected, a special sampling campaign was conducted, and facility drawings were reviewed. The historical data, along with insights from Wellington staff, were analyzed for trends in influent load and operation and to determine stable periods for calibrating and validating a steady state model. The average influent flows and loads for the selected periods are given in Table 1. Historical BOD load and Mixed Liquor concentration are given in Figure 1. As the population of Wellington increases, the BOD load has also increased over time.

Table 1. Average Influent flows and Loads for Calibration and Validation Periods

	Flow	**BOD**	**TSS**	**Temperature**	***NH3****	***TP****
	MGD	lb/d	lb/d	C	*lb/d*	*lb/d*
Oct 2018 Calibration	0.61	1,518	1,767	16.6	*229*	*42.5*
Apr 2019 Validation	0.63	1,411	1,251	14.2	*213*	*39.5*

**Ammonia and Phosphorus load estimated from ratio of influent BOD to influent ammonia and phosphorus in the special sampling campaign on October 8-11, 2019.*

The special sampling data were analyzed to determine influent characteristics which are further described in the Calibration section. The process flow diagram (PFD) shown in Figure 2 was developed using facility drawing details and operational discussions with Wellington staff.

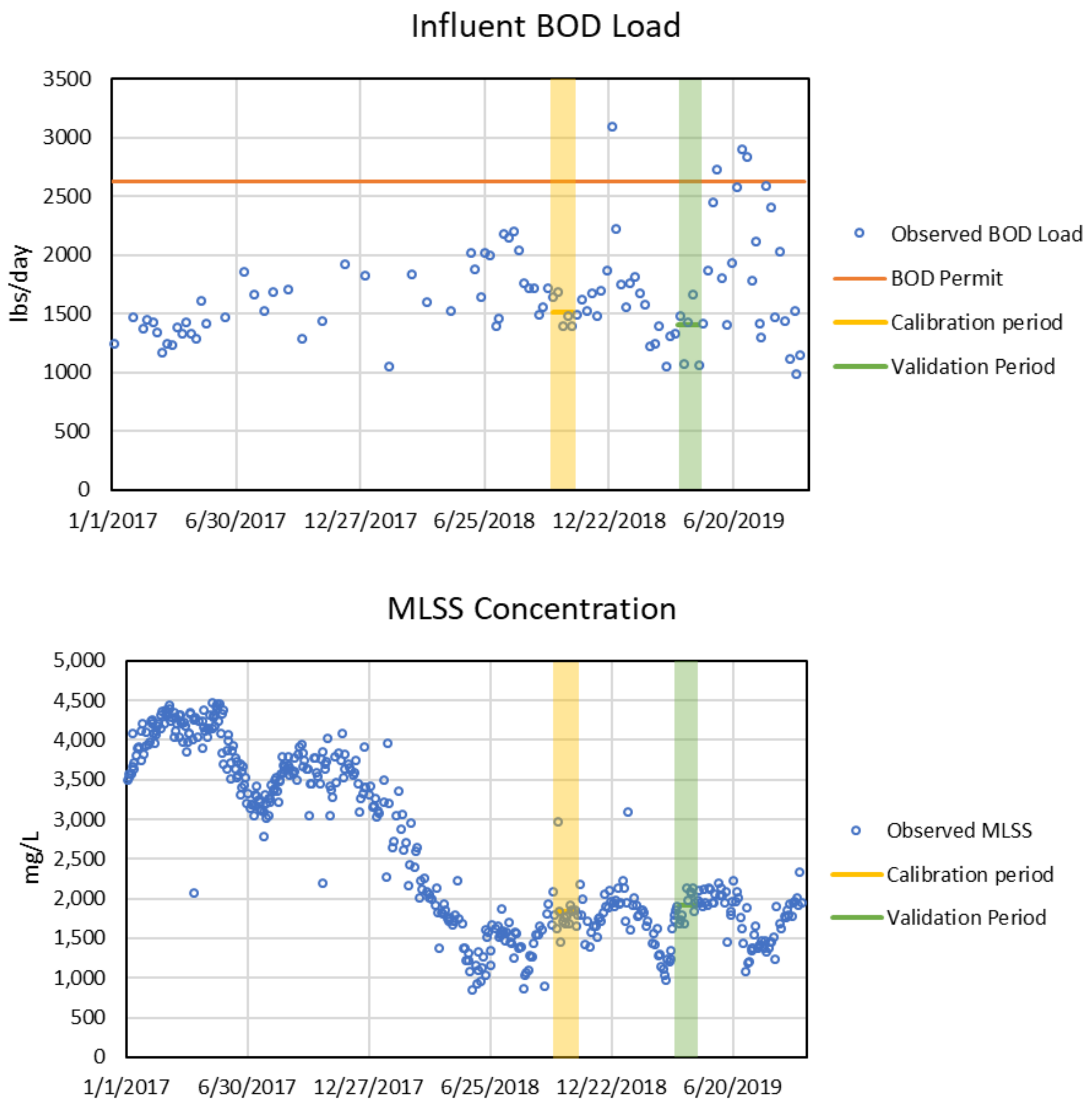


Figure 1. Historic BOD load and Mixed Liquor TSS with Calibration and Validation Periods highlighted.
The calibration month (yellow band, October 2018) and validation month (green band, April 2019) were selected for stable influent and operation. The BOD load permitted maximum month capacity of 2,627 lb/d is shown in orange.

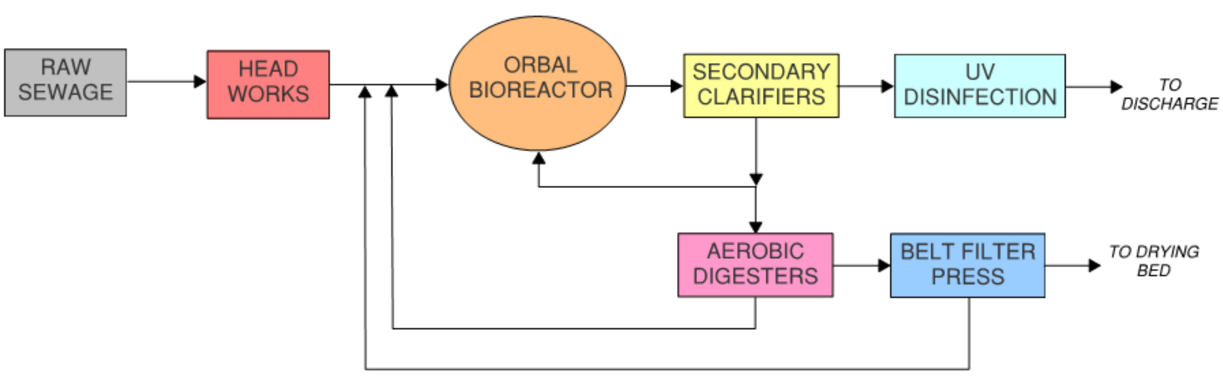


Figure 2. Wellington Treatment plant Process Flow Diagram.

Calibrated Model

The special sampling influent and performance data were used to develop the influent characteristic model inputs given in Table 2. Accurately describing the influent characteristics of a wastewater treatment plant is crucial for an accurate prediction of both the liquids and solids stream performance. Most values for Wellington fall within the typical range except for high VSS fraction of TSS, a high soluble fraction of COD, and a high ratio of ultimate BOD to BOD_5. These indicate a possible industrial load in the collection system such as brewery waste. Intermittent industrial load may explain the increased variability over time in the influent BOD load to the plant shown in Figure 1.

Table 2. Influent Fractionation

Influent Fractionation parameters	Typical range	Calibration Value
COD to BOD_5 ratio	2 - 2.5	2.44
BOD_U/BOD_5 Ratio	1.4 - 1.65	1.9
Ammonia fraction of TKN	65 - 75%	69.0%
VSS fraction of TSS	80 - 90%	94.0%
Filtered COD fraction (including colloids, VFA)	10 - 40%	43.4%
Filtered flocculated COD fraction (including VFA)	5 - 25%	31.7%
VFA fraction of filtered flocculated COD	5 - 50%	36.8%
Fraction of Unbiodegradable filtered flocculated COD of total COD	5 - 10%	4.8%
Influent particulate inert COD fraction	10 - 15%	3.5%
Influent heterotrophic fraction of COD	1 - 30%	10%
Phosphate fraction of TP	50 - 60%	53.8%
Non-biodegradable fraction of influent VSS	20 – 40%	15%

Calibration Results

The average influent flows from October 2018 were entered into the Pro2D model along with average RAS flow rates, final effluent TSS, and temperature from that period. The DO profile developed from the special sampling campaign was input to the Orbal portion of the model. The SRT was adjusted in the model (15.75 days) along with the Non-biodegradable fraction of influent VSS (in Table 2) to find the best fit between average observed performance and the model. The results of this calibration are given in Table 3. The calibration matches the average observed MLSS and WAS flow almost perfectly.

The WAS concentration does not match well, but this is not surprising given the current operation of a constant WAS and RAS flow during the day, which results in variable RAS (and WAS) concentrations. A rough dynamic simulation of the Wellington Orbal experiencing the historical average dynamic influent was performed and the results are given in Figure 3 to try and estimate actual WAS mass. As samples are typically collected in the morning during the greatest variation in concentration, it is understandable that the modeled steady state concentration does not match the grab samples. The average of the WAS concentration in the dynamic simulation does match the Pro2D2 prediction quite well.

The modeled hauled solids are much lower than the reported hauled solids. This could be due to the impact of the drying bed and precipitation which were not included in the model. In discussions with staff, they have indicated they do not have significant confidence in the values reported by the hauling company.

The modeled effluent ammonia and BOD match the observed quite well. The delta percent errors are high, but this is a function of the low values. The errors are almost within measurement error.

The simulated values for MLSS, WAS flow, effluent BOD and effluent ammonia are plotted against observed values for the month of October 2018 and given in Figure 4.

Table 3. Calibration Measured vs Model Results

Component	Measured	Model	% Deviation
Bioreactor (Orbal)			
MLSS (mg/L)	1,832	1,834	0.1%
MLVSS (mg/L)		1,543	
VSS/TSS		84%	
WAS			
Flow (MGD)	0.012	0.012	0.1%
TSS (mg/L)	7,182	9,570	33.2%
VSS (mg/L)		8,022	
VSS/TSS		84%	
Digested Sludge			
Flow (MG/day)		0.007	
TSS (mg/L)		9,968	
VSS (mg/L)		8,454	
TSS (lb/d)		283	
VSS (lb/d)		240	
VSS/TSS		85%	
Decant Flow		4,529	
Belt Filter Press Solids			
Flow (MGD)		0.0004	
TS (lb/d) Hauled wet solids	14,973	4,287	-71%
VSS/TSS		85%	
Final effluent			
BOD (mg / L)	2.60	2.40	-8%
NH_3 (mg N / L)	0.19	0.24	27%
NO_x (mg N/L)		3.45	
OP (mg P/L)		2.03	
Alkalinity (mg caco3/L)		142	
Belt Filter Press Filtrate			
Flow (MGD)		0.0207	
TSS (lb/d)		31	
NH_3 (mg N / L)		14.11	
OP (mg P/L)		8.75	

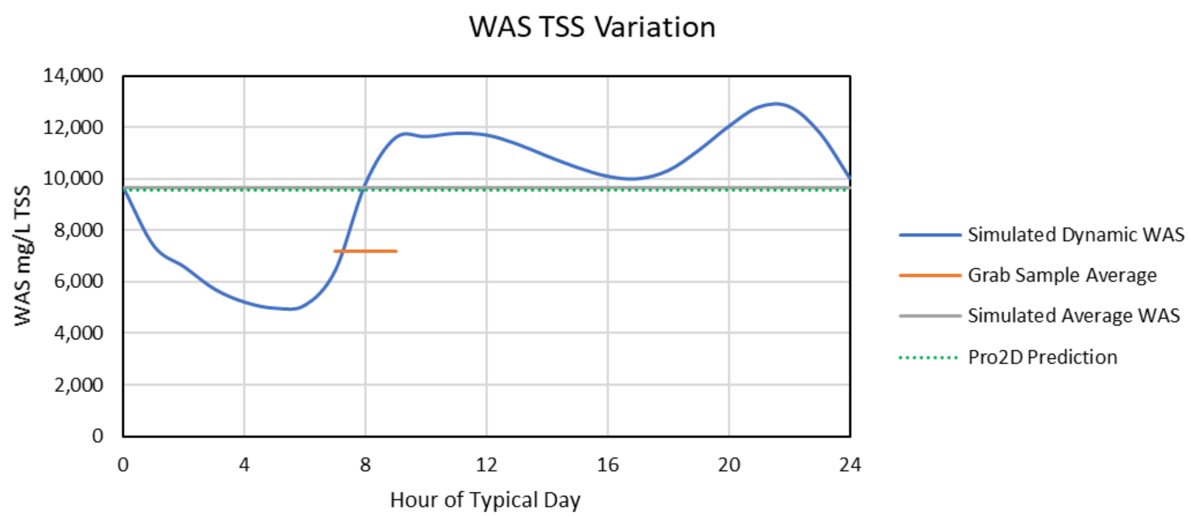
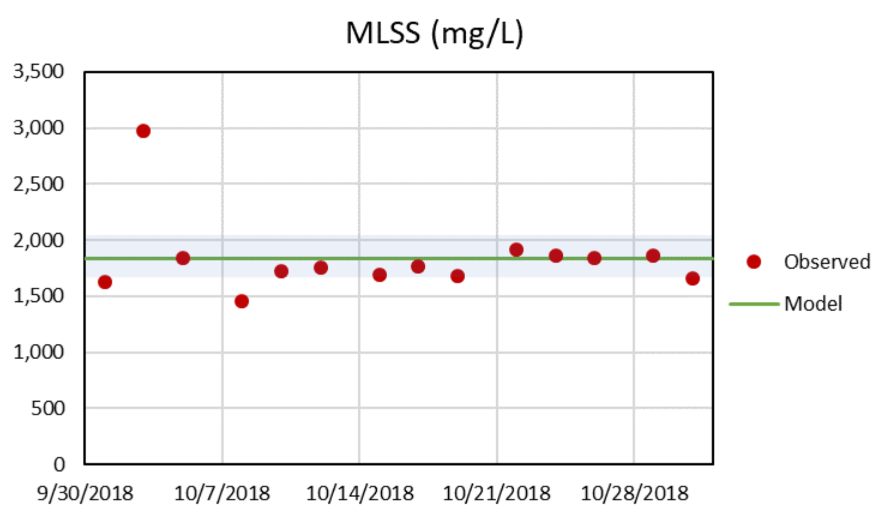
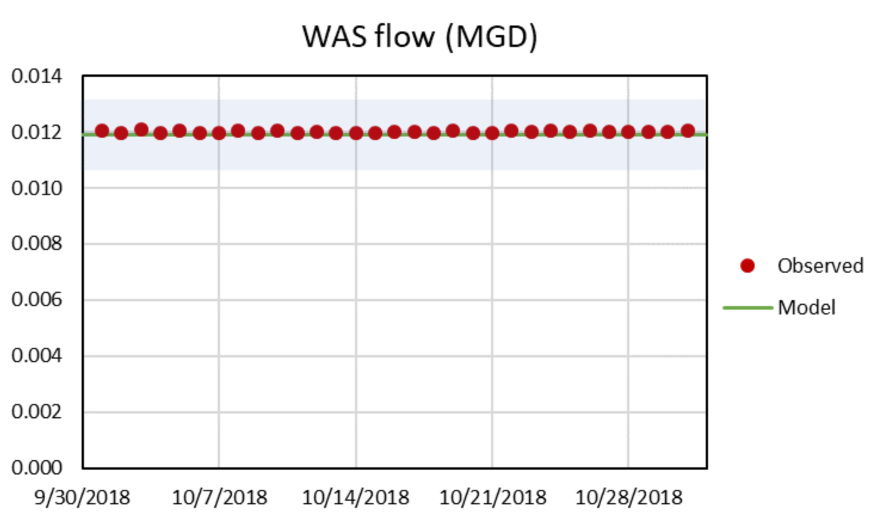
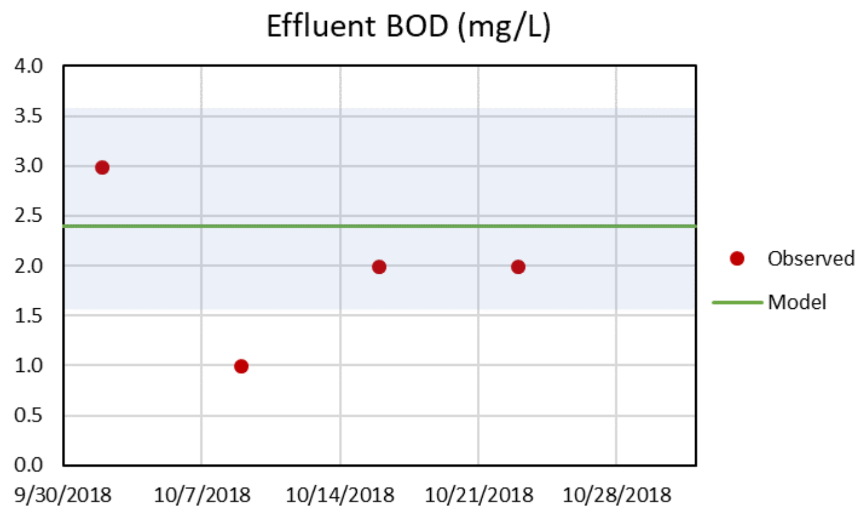


Figure 3. WAS concentration with diurnal influent flow and constant WAS, RAS flow.
The WAS concentration (blue) varies significantly during the day, reflecting the diurnal influent pattern. The observed TSS (orange) is collected during the time of greatest variation. The average of the dynamic WAS (grey) matches closely to the predicted average WAS by the calibrated Pro2D model (green dotted).







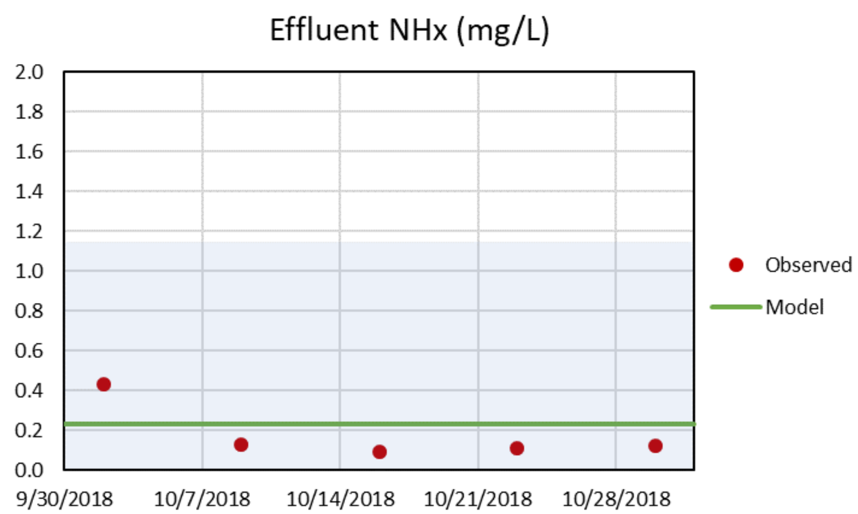


Figure 4. Measured vs Modelled Calibration

The measured performance (red dots) is plotted against the modeled values (green line) for Mixed Liquor concentration, WAS flow, effluent BOD, and effluent ammonia. The light blue band represents acceptable error.

Validation Results

The calibrated Pro2D model was then applied to a different period at Wellington to confirm the predictive capability of the model (i.e. validation). A stable month from spring was chosen to ensure the model is capable of predicted performance in different seasonal conditions. The average influent flow was higher, the load was lower, and the temperature was lower in April compared to October. The only changes made to the model were the average influent flows and loads, the average observed RAS flow rate and observed temperature. The SRT alone was adjusted to match the Mixed Liquor and WAS flow. The results of the validation are given in Table 4.

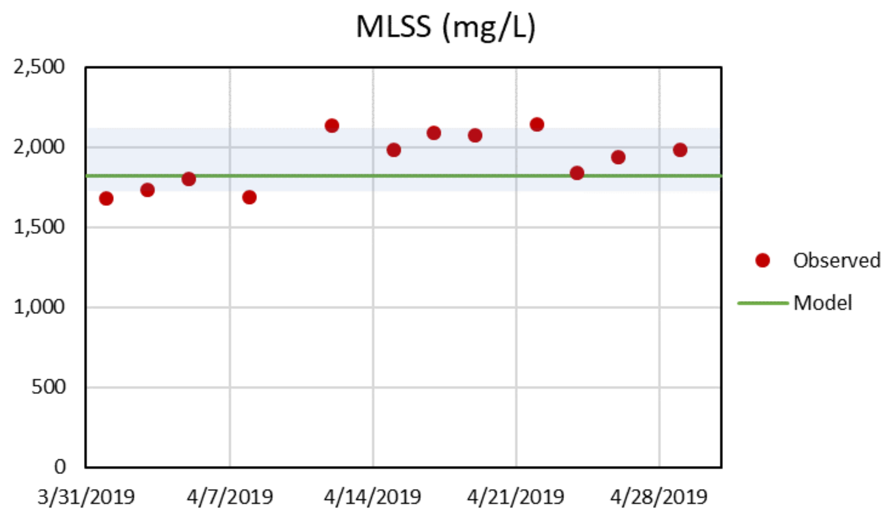
The acceptable error for MLSS concentration in a validation is +/- 10%, thus these results are within range. The deviation is also less than the variation in observations for this period. The deviation in the WAS concentration and hauled solids are attributed to the same reasons as described in the calibration.

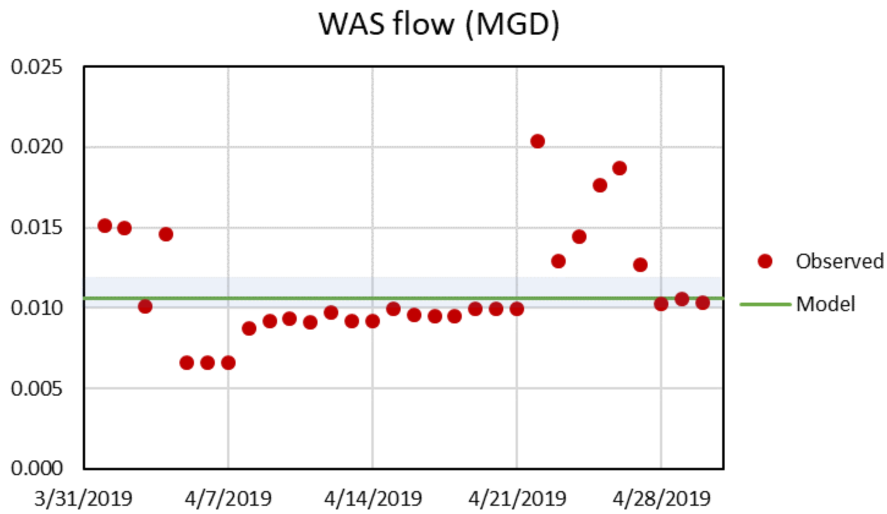
The acceptable error for effluent ammonia is +/- 1 mg/L, while this validation is only off by 0.27 mg/L (the % error on small values is misleading). The effluent BOD prediction is lower than the observed by almost 4 mg/L. There appears to have been a toxicity event that inhibited BOD removal in the month previous to the validation period which may have not quite resolved by April.

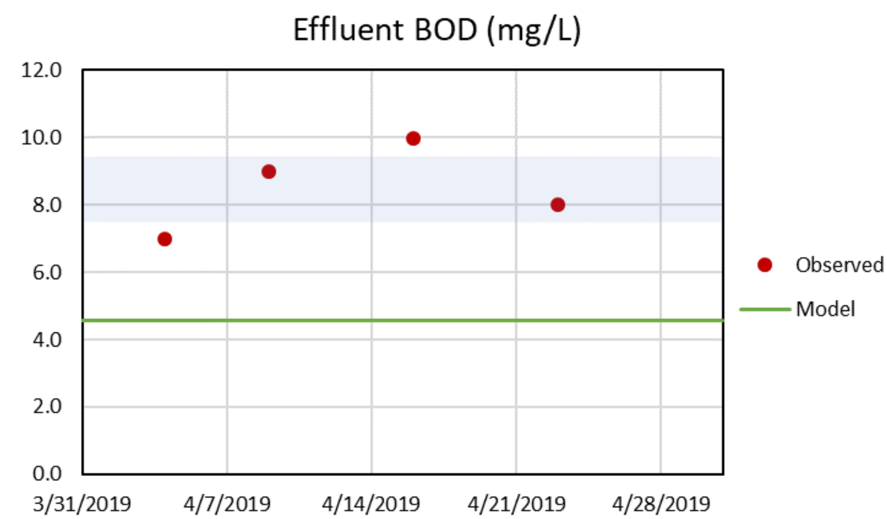
The simulated values for MLSS, WAS flow, effluent BOD and effluent ammonia are plotted against observed values for the month of April 2019 and given in Figure 5.

Table 4. Validation Measured vs Model Results

Component	Measured	Model	% Deviation
Bioreactor			
MLSS (mg/L)	1,923	1,821	-5.3%
MLVSS (mg/L)		1,507	
VSS/TSS		83%	
WAS			
Flow (MGD)	0.0113	0.0107	-5.4%
TSS (mg/L)	11,492	9,309	-19.0%
VSS (mg/L)		7,672	
VSS/TSS		82%	
Digested Sludge			
Flow (MG/day)		0.008	
TSS (mg/L)		8,346	
VSS (mg/L)		7,025	
TSS (lb/d)		238	
VSS (lb/d)		200	
VSS/TSS		84%	
Decant Flow		3,164	
Belt Filter Press Solids			
Flow (MGD)		0.0004	
TS (lb/d) Hauled wet solids	14,002	3,607	-74%
VSS/TSS		84%	
Final effluent			
BOD (mg / L)	8.50	4.56	-46%
NH_3 (mg N / L)	0.08	0.35	361%
NO_x (mgN/L)		3.52	
OP (mg P/L)		1.22	
Alkalinity (mg caco3/L)		143	
Belt Filter Press Filtrate			
Flow (MGD)		0.0207	
TSS (lb/d)		31	
NH_3 (mg N / L)		14.11	
OP (mg P/L)		8.75	


MLSS (mg/L)
2,500
2,000
1,500
1,000
500
0
3/31/2019
4/7/2019
4/14/2019
4/21/2019
4/28/2019
Observed
Model


WAS flow (MGD)
0.025
0.020
0.015
0.010
0.005
0.000
3/31/2019
4/7/2019
4/14/2019
4/21/2019
4/28/2019
Observed
Model


Effluent BOD (mg/L)
12.0
10.0
8.0
6.0
4.0
2.0
0.0
3/31/2019
4/7/2019
4/14/2019
4/21/2019
4/28/2019
Observed
Model

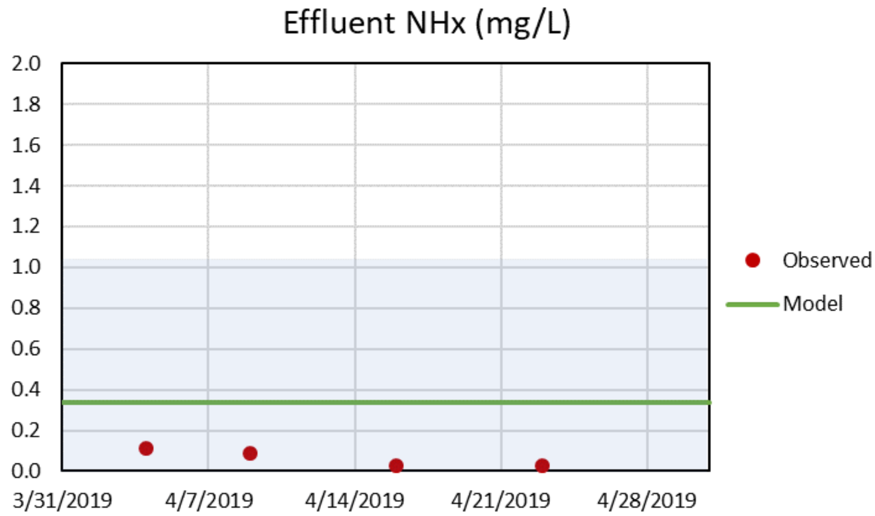


Figure 5. Measured vs Modelled Validation

The measured performance (red dots) is plotted against the modeled values (green line) for Mixed Liquor concentration, WAS flow, effluent BOD, and effluent ammonia. The light blue band represents acceptable error.

Summary

Operation and performance data from the plant were collected and analyzed. This data, in combination with insights and guidance from facility staff, was used to develop, calibrate, and validate a satisfactory steady state process model in the Pro2D2 software of the Wellington Wastewater Treatment Plant. Key operational parameters were well-predicted by the model, providing confidence in its ability to evaluate alternatives for the facility.