


Richmond Wastewater Treatment Facility
20 Year Evaluation
30% Submittal

October 2023

Prepared for:
Town of Richmond, Vermont

Richmond
VERMONT

HOYLE TANNER

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1. PROJECT PLANNING

1.1 Background

The Richmond Wastewater Treatment Facility (WWTF) operates under NPDES Permit No. 3-1173 effective December 21, 2020. The Richmond Wastewater Treatment Facility is a 0.222 MGD activated sludge treatment facility that incorporates the use of an anoxic selector. In general, the treatment processes at the facility involve the use of screening, grit removal, anoxic selectors, aeration basins, secondary clarification, filtration, and disinfection. The solids train includes septage receiving, aerated sludge holding, and dewatering. A process flow diagram is located below in Figure 1-1.

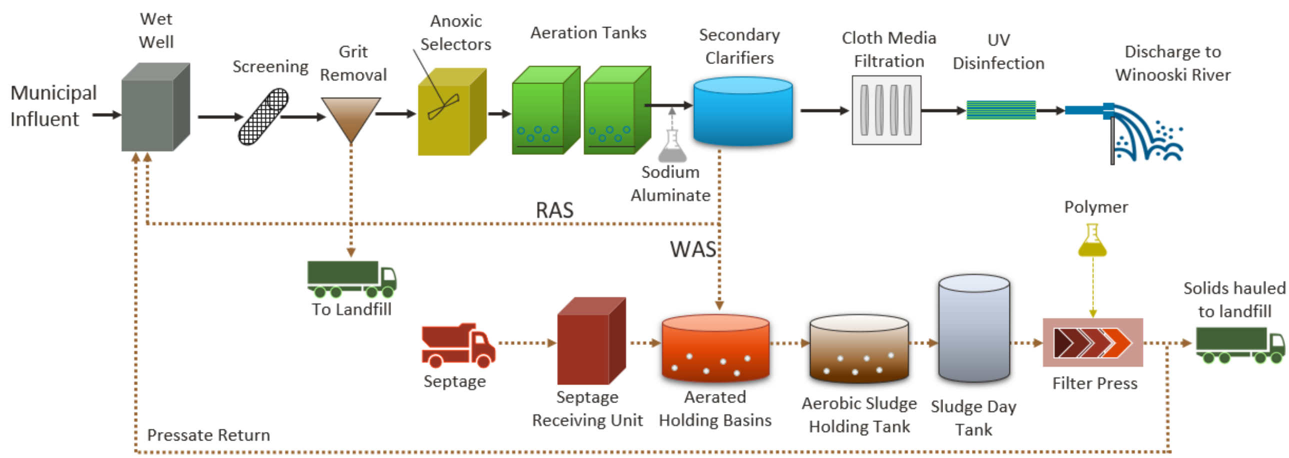


Figure 1-1. Process Flow Diagram

The following is a history of the facility:

- 1972: Original Facility Constructed
- 2005: Major Facility Upgrade (New Headworks Building w/ Grit Removal, Anoxic Selectors, Aeration Tank Diffusers, New Process Building w/ Filtration & UV Disinfection, New Aerated Septage Holding Tank)
- 2023: One influent pump replaced with used emergency pump

This 20-Year Evaluation will assess the existing facility and collection system to identify needs, develop alternatives to address the needs, and select a recommended alternative.

1.2 Scope of Services

Hoyle, Tanner's scope of services for this study is summarized in the following:

The 20-Year Evaluation will be prepared to incorporate the following information. The report will follow the State Water Investment Division (WID) format.

- Project Planning (Section 1)
- Existing Wastewater Facilities (Section 2)

- Existing Collection System (Section 3)
- Need for Project (Section 4)
- Alternatives Considered (Section 5)
- Selection of Alternative (Section 6)
- Proposed Project (Recommended Alternative) (Section 7)
    - Proposed Hydraulic Profile
    - Proposed Process Flow Diagram
    - Proposed Site Plan
    - Equipment selection details including design criteria
    - Opinion of Probable Construction Cost
    - Opinion of Probable Total Project Cost
    - Project phasing defined in a Sequence of Work
    - List of permits/approvals needed for State agencies
    - Proposed project schedule
    - Proposed next steps

1.3 Location

The Richmond WWTF is located on 281 Esplanade Street in Richmond, Vermont. The Richmond WWTF receives wastewater from residential, commercial, and industrial sources and discharges to the adjacent Winooski River through an outfall. An overall location map is provided in Figure A-1 in Appendix A.

1.4 Environmental Resources

The proposed project will not increase the hydraulic capacity of the Main WWTF as the improvements will be addressing age-related needs. All proposed work will occur within the Town property at the Main WWTF in previously disturbed areas.

1.4.1. Winooski River

The Richmond WWTF discharges through Outfall S/N 001 to a waste management zone in the Winooski River, a Class B (2) water, and a designated Cold Water Fish Habitat.

Class B waters are suitable for swimming and other primary contact recreation; irrigation and agricultural uses; aquatic biota and aquatic habitat; good aesthetic value; boating, fishing, and other recreational uses; and suitable for public water source with filtration and disinfection or other required treatment. A waste management zone is a specific reach of waters designated by a permit to accept the discharge of properly treated wastes that prior to treatment contained organisms pathogenic to human beings.

1.4.2. Floodplain

The record drawings from 2004 identify the 100-year flood elevation as 308.50 feet. In 2014, the FEMA Flood Insurance Study No. 50007CV002B for Chittenden County dated August 4, 2014, provided updated flood profiles for the Winooski River. Based on the 2014 Winooski River flood profiles and the facility's approximate distance of 1,300 feet from cross-section "BO" on Flood Profile 65P, the **100-year** and **500-year** flood elevations at the location of the Richmond WWTF are **310.50 feet** and **313.77 feet**, respectively. New flood elevation determination resulted in a 2-foot increase in the 100-year flood elevation from 308.50 feet to 310.50 feet. See Figure A-2 in Appendix A for the river profile at the

Richmond WWTF site and Figure A-3 for the FEMA FIRMette Map. The Richmond WWTF is not located within a regulatory floodway of the Winooski River as shown in Figure A-3 in Appendix A.

The ground elevation around the Process Building, which houses the chemical storage, UV disinfection, and filter units, is 313.50 feet. While the Process Building will be protected from a 100-year flood event, there is risk of flooding during a 500-year flood event.

During Tropical Storm Irene on August 28, 2011, the lower level of the Process Building flooded due to the Winooski River backing up the outfall pipe and overtopping the UV channel which has a top of wall elevation of 309.54 ft. After this flooding event, a check valve on the outfall and a pump was installed in the sump downstream of the effluent weir to discharge effluent flows to the outside grade during an emergency and prevent future flooding.

Inside the Process Building, the elevations of critical process components are as follows:

- Effluent wet well top of wall elevation = 313.30 ft
- Top of UV channel = 309.54 ft
- Top of filter tank wall = 313.22 ft

All of these critical component elevations are below the 500-year flood elevation and the UV channel walls are below the 100-year flood elevation. In the recent July 2023 flooding event, emergency use of a sump pump to lift flow to an external discharge point was used to prevent flooding of the Filter/UV Room.

The ground elevation of the Operations Building is at approximately 314.00 feet and is protected from the 100-year and 500-year flood events.

1.4.3. Wetlands

There are no classified wetlands located on the property as shown in Figure A-4 in Appendix A. A Wetlands Advisory Layer borders the north-west side of the property. The State of Vermont defines the Wetlands Advisory Layer to be wetland locations which have not been formally assessed for significance.

1.4.4. Rare and Endangered Species

No portions of the WWTF property are located in an area designated with the element occurrence of a rare or endangered animal or plant as shown in Figure A-5 of Appendix A.

1.4.5. Archeological Resources

Pending

1.4.6. Historical Preservation

Pending

1.5 Population Trends

The United States Census Bureau population data for the Town of Richmond from 1990, 2000, 2010, and 2020 were 3,729, 4,090, 4,081, and 4,167 respectively. Population data for the Town of Richmond is shown below in Table 1-1.

Table 1-1 United States Census Information, Richmond, Vermont

Census Year	Population	Previous 10-Year Growth (+/-)
1990	3,729	-
2000	4,090	9.68%
2010	4,081	-0.22%
2020	4,167	2.11%

1. From US Census Data

Historical census data shows positive growth in the Town of Richmond from 1990 to 2020. The Town of Richmond's 2018 Town Plan states "population predictions show a relatively stable population over the next 10-15 years (ranged from a decline of about 180 people to an increase of about 35 people)". The Town of Richmond is well suited for population growth due to its proximity to the greater Burlington area and location along the I-89 corridor. Along with this steady population growth, the Town is looking to extend the municipal wastewater service area to zoned growth areas of the Town that are growth-limited by on-site wastewater systems.

1.6 Community Engagement

1.6.1. Public Participation

The Town of Richmond actively engages the community and promotes public participation through the following:

- Public Meetings
- Local Newspaper Advertisements
- Front Porch Forum Postings
- Direct Mailings

2. EXISTING FACILITIES

2.1 Location Map

A location map is shown in Figure A-1 in Appendix A.

2.2 History

The Town of Richmond owns and operates the Richmond Wastewater Treatment Facility and associated 4.14 miles (21,880 linear ft) of sewer mains and the Bridge Street pump station that make up the collection system serving the service area. The history of the facility is as follows:

- 1972: Original Facility Constructed
- 2005: Major Facility Upgrade (New Headworks Building w/ Grit Removal, Anoxic Selectors, Aeration Tank Diffusers, New Process Building w/ Filtration & UV Disinfection, New Aerated Septage Holding Tank)
- 2023: One influent pump replaced with interim pump

Although some specific equipment upgrades and replacements have occurred in the past, the majority of the facility has not been upgraded since the major upgrade in 2005. The existing WWTF site plan and hydraulic profile are provided in Figures A-6 and A-7 respectively in Appendix A.

2.3 Existing Discharge Permit

The Richmond WWTF is permitted under National Pollutant Discharge Elimination System (NPDES) Permit No. 3-1173, effective January 1, 2021, to discharge treated effluent from outfall S/N 001 to the Winooski River.

Table 2-1 summarizes the WWTF's existing discharge permit flow and effluent quality requirements. The current version of the NPDES permit and fact sheet are publicly available at:

https://dec.vermont.gov/watershed/wastewater/discharge-permits

Table 2-1 Richmond WWTF Current NPDES Discharge Permit

Effluent Parameter	Annual Avg	Annual Total	Monthly Average	Weekly Average	Daily Maximum	Instantaneous Maximum
Flow	0.222 MGD	--	--	--	--	--
BOD_5	--		30 mg/L 55.5 lbs/day	45 mg/L 83.3 lbs/day	50 mg/L --	--
Total Phosphorus (TP)	--	-- 134 lbs/yr	0.8 mg/L --	--	--	--
Total Nitrogen (TN)	--	--	--	--	Monitor Only	--
Total Kjeldahl Nitrogen (TKN)	--	--	--	--	Monitor Only	--
Nitrate/Nitrite Nitrogen (NOx)	--	--	--	--	Monitor Only	--
Settleable Solids	--	--	--	--	--	1 ml/L
Total Suspended Solids (TSS)	--	--	30 mg/L 55.5 lbs/day	45 mg/L 83.3 lbs/day	50 mg/L --	--
E. Coli	--	--	--	--	--	77 col/100 mL
pH	--	--	Between 6.5 and 8.5 Standard Units			--

2.4 Original Design Criteria

Table 2-2 summarizes the original WWTF design criteria, as well as current loadings.

Table 2-2 Existing Influent Design Criteria and Current Loadings

Parameter	Design Criteria[1]	Current Loadings[2]
Average Daily Flow	0.222 MGD	0.073 MGD
Peak Hourly Flow	1.152 MGD[3]	0.660 MGD[4]
Biochemical Oxygen Demand (BOD)	324 mg/L 600 lbs/day	670.2 mg/L[5] 461.7 lbs/day
Total Suspended Solids (TSS)	272.5 mg/L 500 lbs/day	932.0 mg/L[5] 680.5 lbs/day
Total Phosphorus (TP)	10 mg/L 18.5 lbs/day	19.5 mg/L[5] 14.0 lbs/day

Notes:

1. Basis of Final Design, 2003.
2. Based on historic operating data from January 2018 to February 2023.
3. Noted in 2003 Basis of Final Design as "based on influent pumping capacity."
4. Additional data is needed to verify current peak hourly flow.
5. Influent sampling is taken at the influent channel of the wet well, upstream of RAS and pressate side streams. Samples can, at times, include return activated sludge (RAS) & pressate, when the wet well is used for flow equalization and an isolated influent sample is not possible.

The original design criteria from the 2003 Basis of Final Design was established as a daily organic load (lb/day). These loads were converted to a concentration (mg/L) based on the design average daily flow of 0.222 MGD.

Historical influent data was analyzed and revealed that the current influent TSS load of 680.5 lb/day TSS exceeds the original design criteria of 500 lb/day TSS. While historical influent BOD and TP loads do not exceed the original design criteria, the historical influent concentrations are greater than the equivalent original design concentrations. Currently, Richmond is operating at an average daily flow of 0.073 MGD, which is approximately 33% of the design flow. Given that the current TSS load exceeds original design criteria, and that current BOD and TP concentrations are greater than the equivalent design criteria concentrations, a closer look needs to be taken into sources of these high influent concentrations and evaluate ways to reduce influent loading to the WWTF. If current influent concentrations were to remain constant at the design flow of 0.222 MGD, the original design criteria would be severely exceeded.

2.5 Historical Operating Data

Historical operating data was reviewed from January 2018 through February 2023.

2.5.1. Flow

The Richmond WWTF records effluent flow from the v-notch weir located on the effluent channel. The average monthly effluent flow from January 2018 to February 2023 ranged from 0.05 to 0.11 MGD with an average of 0.073 MGD which is 31.5% of the design influent average daily flow of 0.222 MGD.

The peak day effluent flow from January 2018 to February 2023 ranged from 0.07 to 0.59 MGD with an average of 0.14 MGD. The 2003 basis of design did not include design criteria for peak day flow.

The peak hour effluent flow from January 2018 to February 2023 ranged from 0.27 to 0.66 MGD with an average of 0.54 MGD. The effluent flow measurement has a maximum range of 0.66 MGD that likely has been exceeded. The 2003 basis of design peak hour flow design criteria was 1.152 MGD based on influent pumping capacity. The historic maximum peak hour flow of 0.66 MGD is approximately 57.3% of the existing peak hour flow design flow, although it is suspected that this has been exceeded. Further investigation or additional flow monitoring is necessary to determine the facility's peak instantaneous flow and establish a peaking factor. Determining an accurate peak hour flow is important for influent pump selection to ensure pumps can convey peak hour flows throughout the facility.

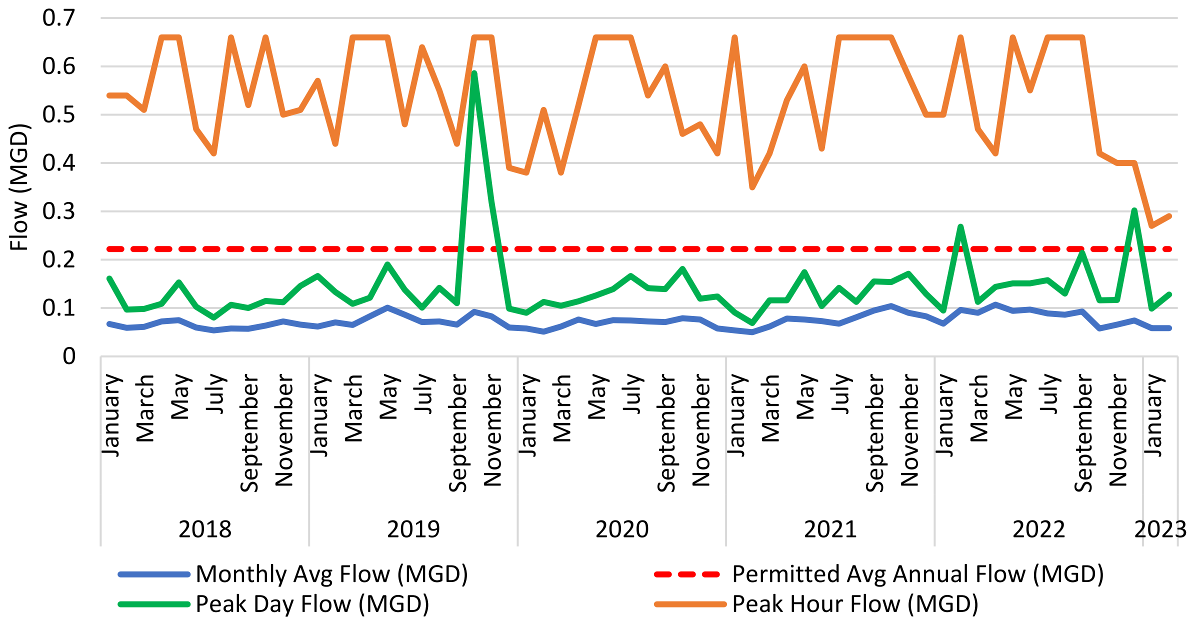


Figure 2-1 Historical Effluent Flow Data

While flow data includes the flow of pressate, there is no flow meter located on the dewatering pressate line that returns to the influent wet well. Therefore, it is difficult to understand the volume of pressate flow entering the biological process and the nutrient loads associated with pressate separate from municipal influent.

2.5.2. Biochemical Oxygen Demand (BOD)

Influent sample collection occurs at the influent wet well at the Richmond WWTF. Sample collection typically consists of combining grab samples over an 8-hour period to create a composite sample. The influent wet well receives flow from the collection system, return activated sludge (RAS), tertiary filter backwash, and pressate from the rotary press, however Richmond WWTF operators collect samples at the influent to the wet well, upstream of RAS and on days when dewatering is not running and pressate is not entering the wet well. While influent sampling does not necessarily isolate collection system quality from the RAS and pressate; historically, samples contain mostly municipal influent.

Influent BOD concentrations from January 2018 to February 2023 ranged from 170 to 1,700 mg/L with an average concentration of 670 mg/L, which is well above the 2003 influent BOD loading design criteria equivalent concentration of 324 mg/L. The variability in BOD concentration may be impacted by the volume of septage received and subsequently dewatered and when the grab sample is collected, however it is suspected that there may be significant industrial users on the collection system that are contributing to the high BOD concentration seen in the influent samples. The average historical BOD load associated with the influent concentration is 462 lbs/day, which is below the 2003 influent BOD loading design criteria of 600 lbs/day.

Influent BOD concentrations have been decreasing over this time period. The average influent BOD concentration from January 2018 through December 2020 was 754 compared to 554 mg/L from January 2021 to February 2023.

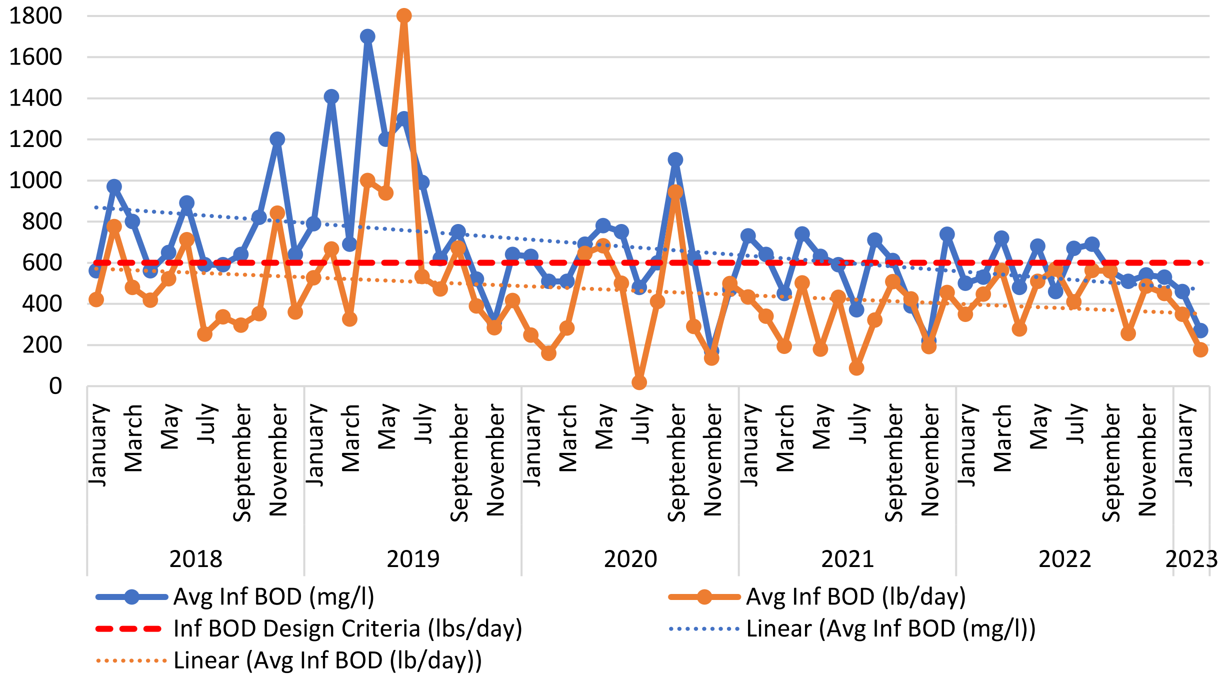


Figure 2-2 Influent BOD

Effluent BOD concentrations ranged from 1 to 17 mg/L with an average of 2.86 mg/L. The average effluent BOD load was 1.96 lb/day which is well below the permitted monthly average load of 55.5 lb/day. Based on this data, the existing biological process provides conditions that support effective BOD removal at current loadings. Over this period of time, BOD removal ranged from 98.53-99.90+% with an average of 99.60% removal.

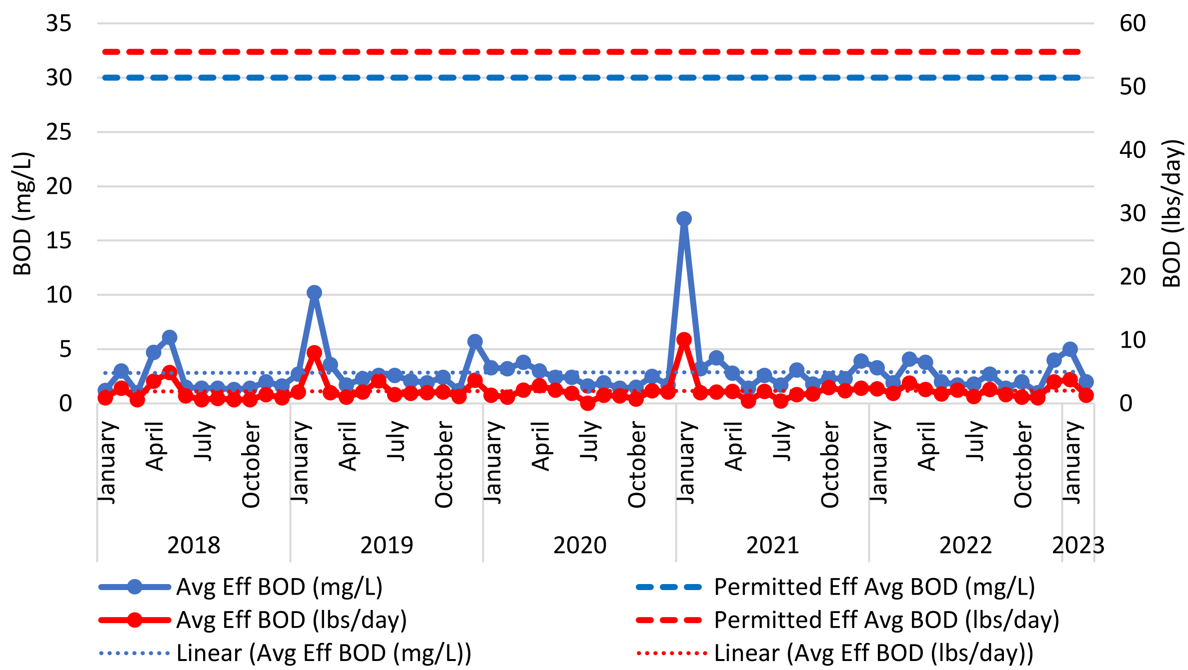


Figure 2-3 Historical Effluent BOD Data

2.5.3. Total Suspended Solids (TSS)

Influent TSS concentrations from January 2018 to February 2023 ranged from 271 to 4,120 mg/L with an average of 932 mg/L, which is well above the 2003 influent TSS loading design criteria concentration of 272.5 mg/L. The variability in TSS concentration may be impacted by the volume of septage received and subsequently dewatered when the grab sample is collected, however it is suspected that there may be significant industrial users on the collection system that are contributing to the high TSS concentration seen in the influent samples. The average TSS load was 680.5 lb/day TSS which is 1.36 times the 2003 design influent TSS load. Note, the influent TSS loading design criteria does not include side streams included in the influent sampling (RAS, pressate).

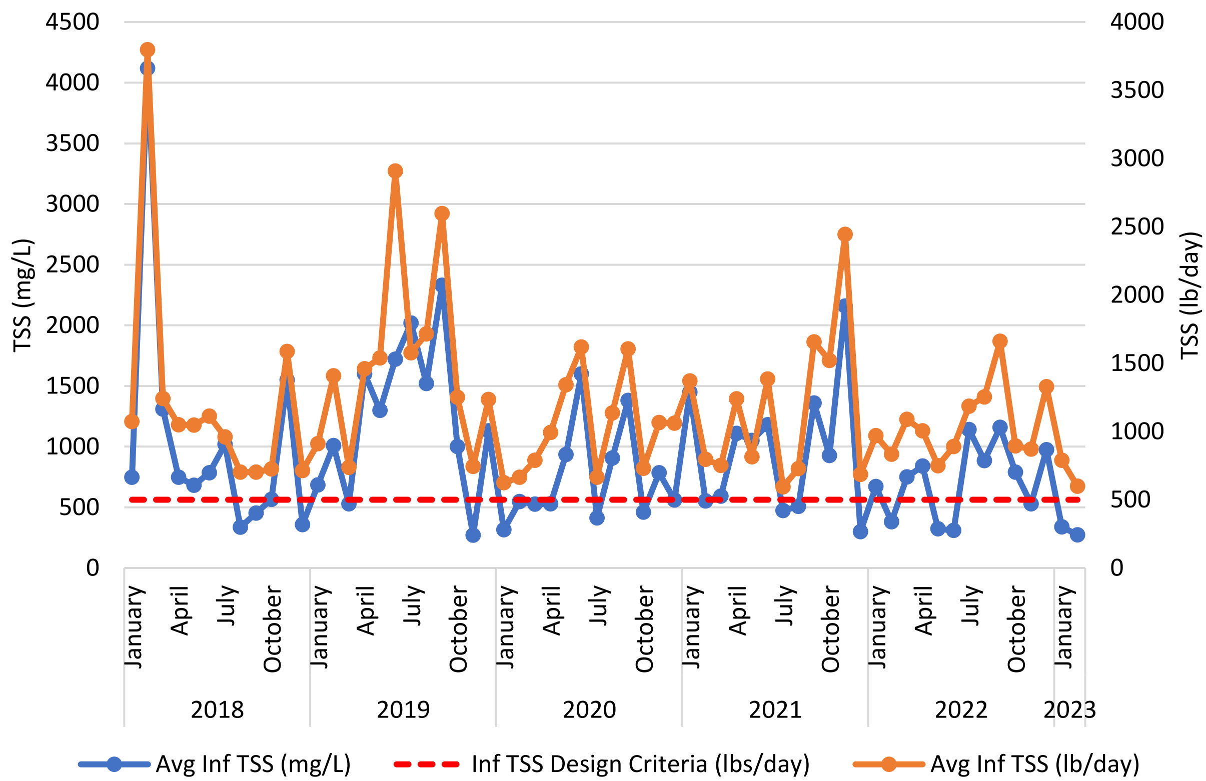


Figure 2-4 Historical Influent TSS Data

The Richmond WWTF has been performing well and removes between 98.15-99.95+% of TSS. From January 2018 to February 2023, the effluent TSS concentration ranged from 1.00 to 6.00 mg/L, with an average of 2.65 mg/L, which is well below the permitted monthly effluent TSS concentration of 30 mg/L. The average effluent TSS load from January 2018 to February 2023 was 1.86 lb/day which is also well below the permitted monthly effluent TSS load of 55.5 lb/day. Data indicates the WWTF has the capacity to treat the current TSS load.

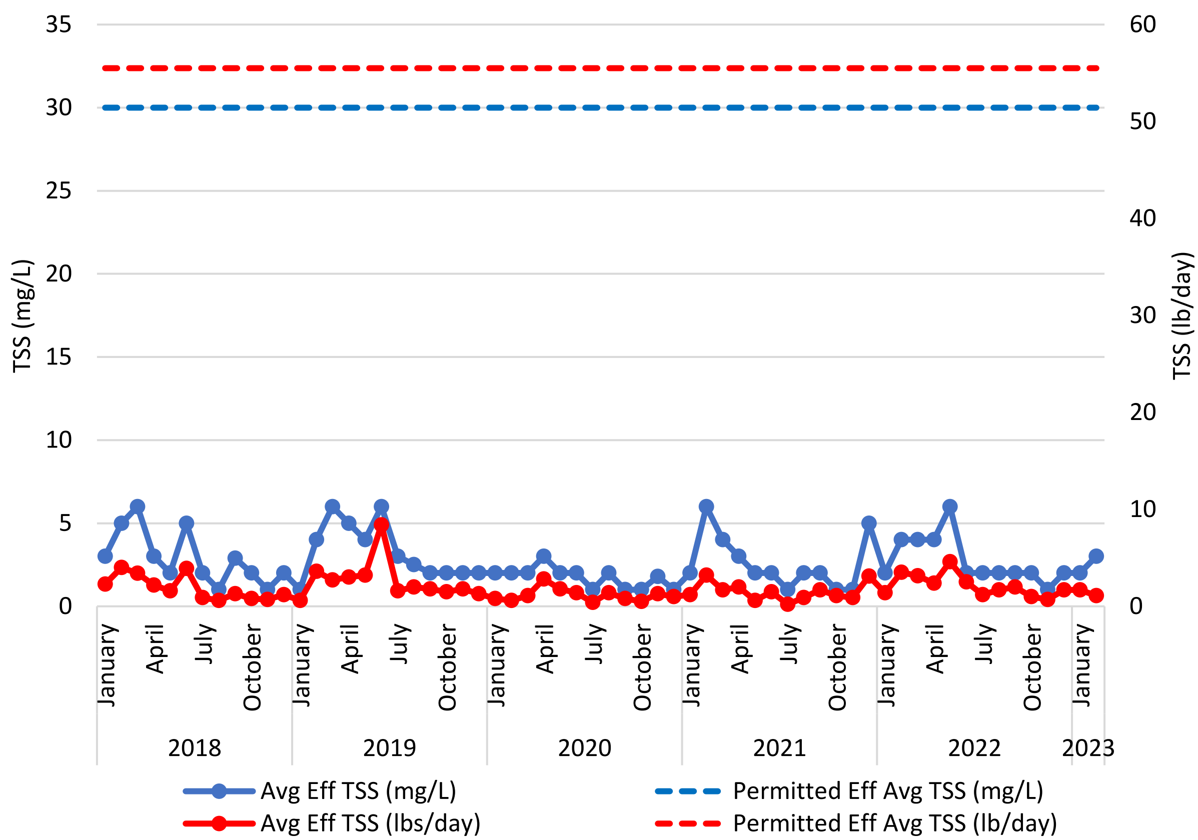


Figure 2-5 Historical Effluent TSS Data

2.5.4. Total Nitrogen (TN)

Historically, influent nitrogen samples have not been collected at the Richmond WWTF. However, a 24-hour sampling event took place from 9/20/2022 to 9/21/2022 which analyzed ammonia concentrations from composite samples taken at several different locations throughout the facility. Ammonia concentrations from the 24-hour sampling event are displayed in Table 2-3.

Table 2-3 Sampling Event Ammonia Data

Sampling Location	Ammonia Concentration (mg/L)
Influent Wet Well	30
Septage	74
Pressate	44.5

Richmond WWTF's permit requires quarterly reporting of effluent total nitrogen (TN), total Kjeldahl nitrogen (TKN), and nitrite and nitrate (NOx). In addition to these monitoring requirements, effluent ammonia is tested periodically. Data available for nitrogenous species monitoring is from January 2021 through February 2023.

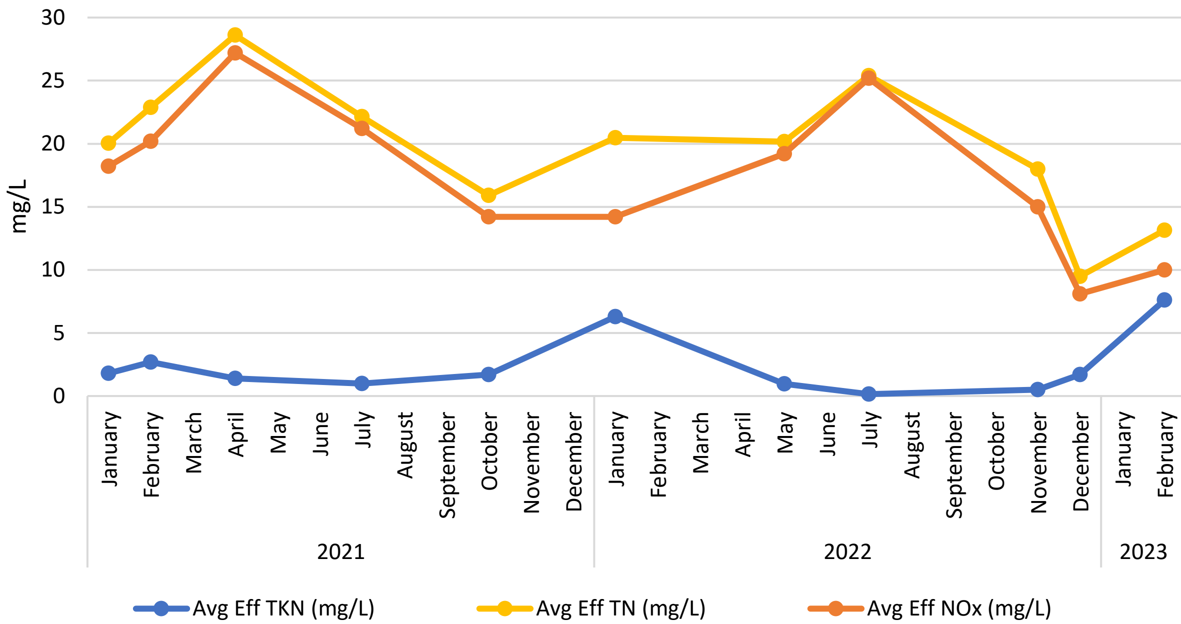


Figure 2-6 Historical Effluent Nitrogen

Effluent TN concentrations ranged from 9.49 to 28.60 mg/L with an average of 19.66 mg/L indicating that the system has some potential for denitrification. Effluent NOx concentrations ranged from 8.10 to 27.20 mg/L with an average of 17.52 mg/L. Effluent TKN concentrations ranged from 0.15 to 7.60 mg/L with an average of 2.35 mg/L indicating the system maintains effective nitrification.

2.5.5. Phosphorus

Influent total phosphorus (TP) concentrations from January 2018 to February 2023 ranged from 4.8 to 73 mg/L with an average of 19.45 mg/L. Pressate total phosphorus concentrations can be high strength, which can contribute to the total phosphorus concentration in the wet well if samples were collected on days when the press was running, however it is suspected that there may be significant industrial users on the collection system that are contributing to high TP concentrations seen in the influent samples. The historical influent TP concentration of 19.45 mg/L exceeds the existing influent TP design criteria is 10 mg/L.

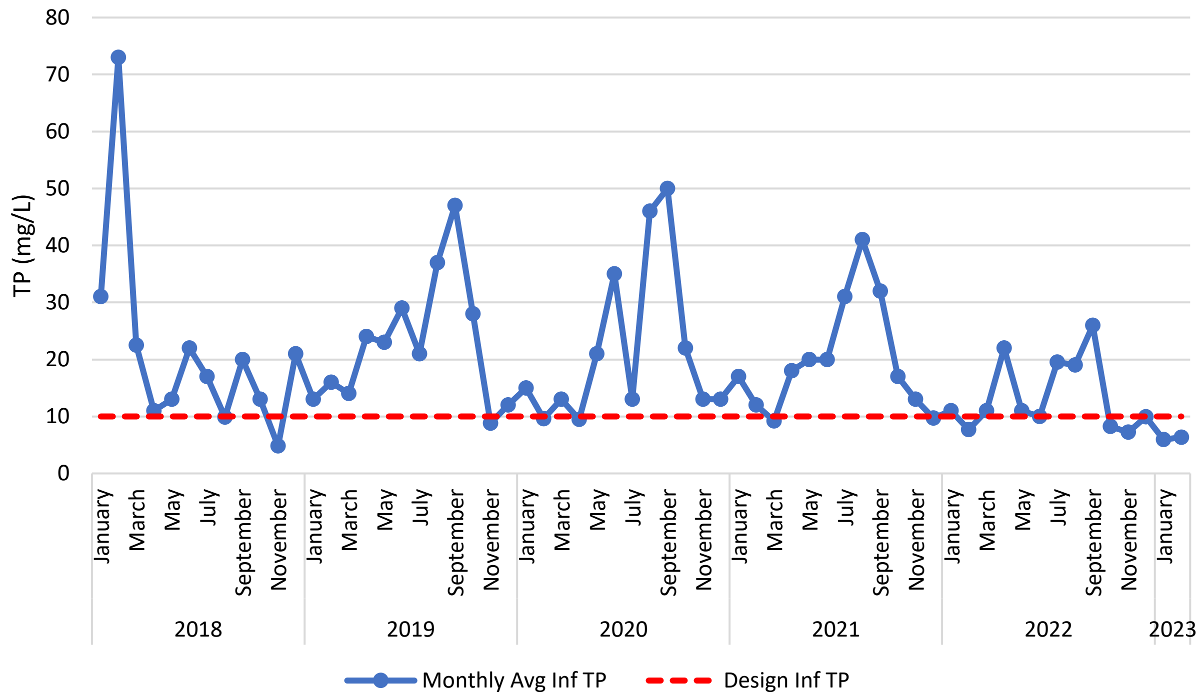


Figure 2-7 Historical Influent Phosphorus Data

Effluent TP concentrations from January 2018 to February 2023 ranged from 0.03 to 0.39 mg/L with an average of 0.10 mg/L which is well below the permitted average monthly effluent TP concentration of 0.80 mg/L. Monthly average effluent TP load ranged from 0.01 to 0.40 lbs/day with an average of 0.07 lbs/day. In addition to the permitted monthly average effluent TP limit, the facility has an annual permitted effluent TP load of 134 lbs/year, equivalent to approximately 0.37 lbs/day. The monthly average effluent TP load of 0.07 lbs/day is equivalent to 25.6 lbs/year. Over this time period, phosphorus removal ranged from 97.10-99.88+% with an average removal of 99.36%. Data indicates that the Richmond WWTF has the capacity to treat current the total phosphorus load. Overall, the facility achieves excellent TP removal.

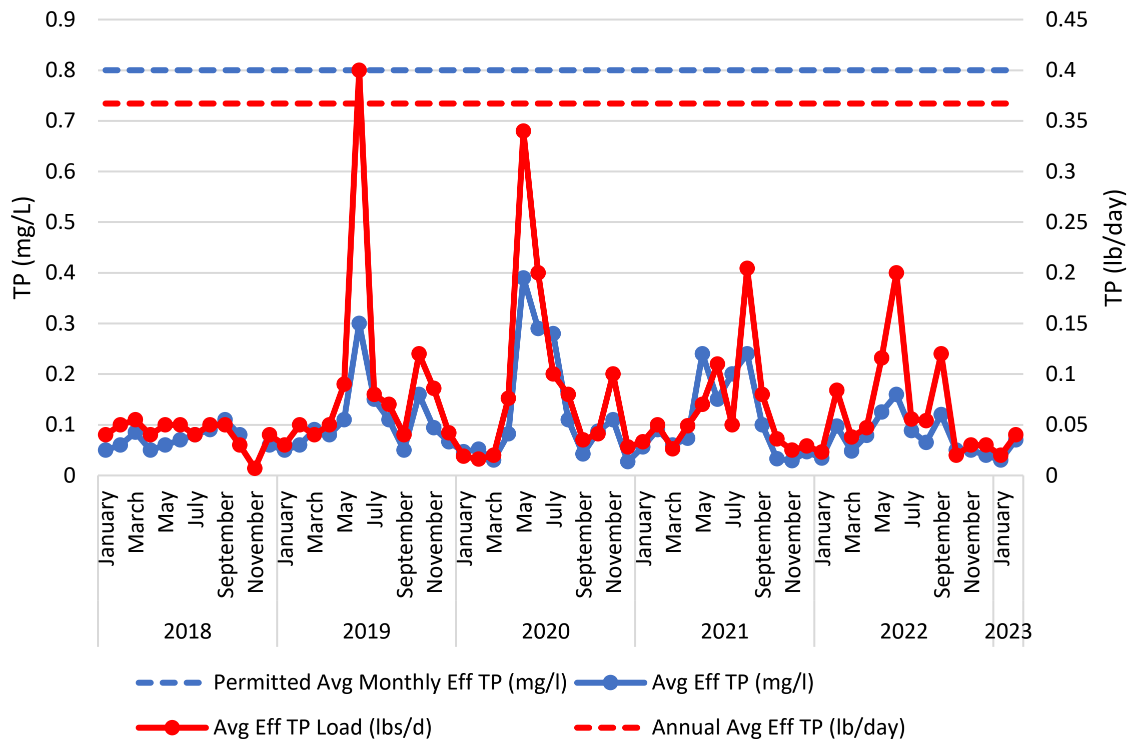


Figure 2-8 Historical Effluent Phosphorus Data

2.5.6. E. Coli

The Richmond WWTF had no exceedances of the permitted instantaneous maximum e. coli limit of 77 counts per 100 mL as displayed in Figure 2-9. From January 2018 to February 2023, maximum effluent e. coli ranged from <0.10 to 16 counts per 100 mL.

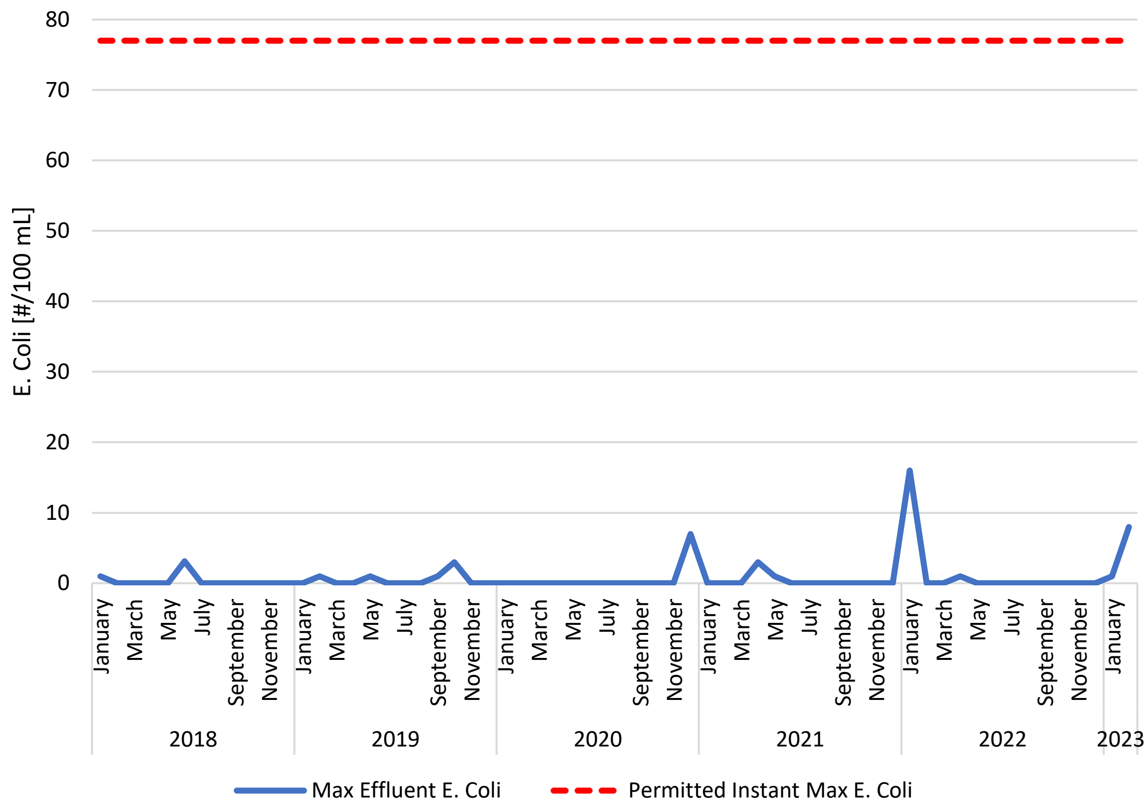


Figure 2-9 Effluent E. Coli

2.5.7. Septage Received

The monthly septage received from January 2018 to February 2023 ranged from 27,300 to 1,200,500 gallons per month with an average of 610,995 gallons per month. The Richmond WWTF was designed to accept 2,000,000 gallons of septage per year, or approximately 167,000 gallons per month. In 2021 and 2022, the Richmond WWTF received a total of 9,087,100 and 7,626,000 gallons per year, respectively. The facility is currently accepting significantly more septage than the original design, which is possible as significant available loading at the facility was freed up when the cheese production factory closed in 1999. Effluent quality continues to meet permit limits.

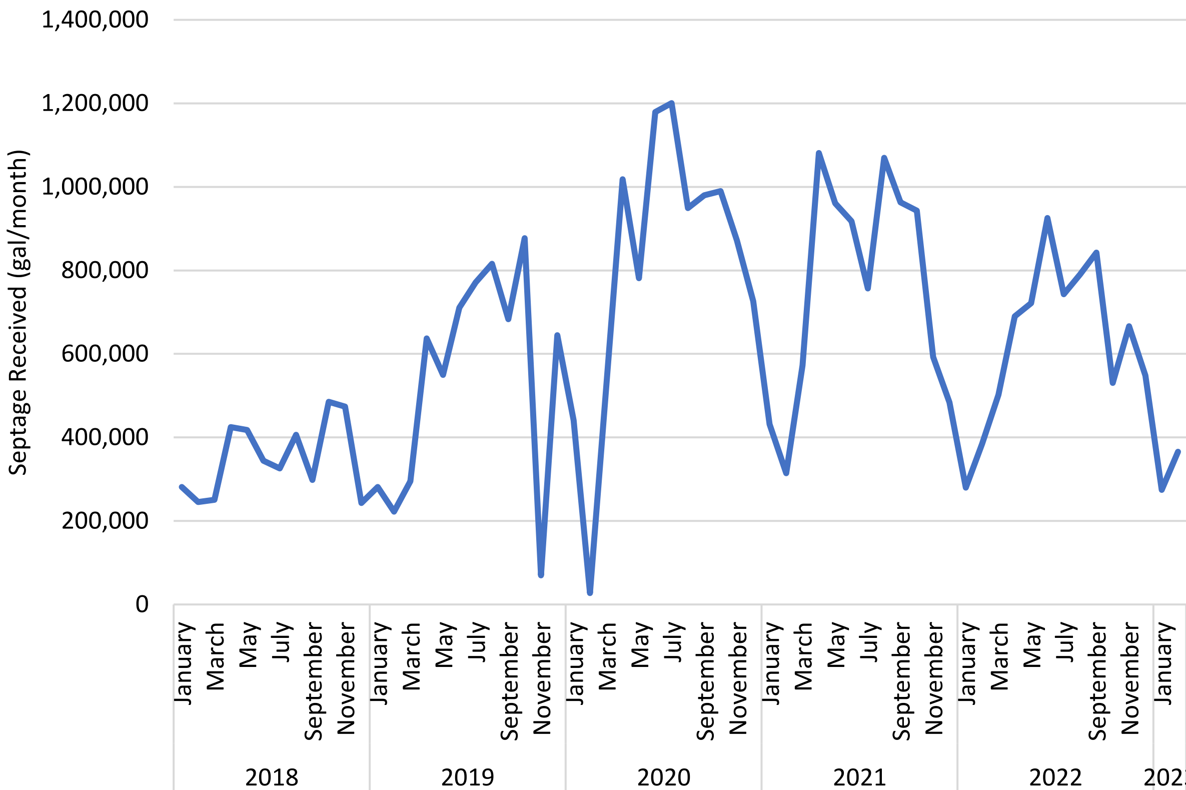


Figure 2-10 Historical Septage Receiving

Septage receiving is fed through a septage receiving unit prior to mixing with waste activated sludge (WAS). The combination of septage and WAS is then dewatered and pressate is directed to the influent wet well where it combines with municipal influent. Due to the lack of a flow meter and historical sampling of pressate, it can be difficult to draw conclusions on the impact of septage receiving and pressate return on the process. However, on 9/20/2022 through 9/21/2022, a sampling event took place at the Richmond WWTF as part of the plant's Phosphorus Optimization Plan study. Data was collected on both the raw septage and the pressate. Sampling results are located below in Table 2-4.

Table 2-4 Septage & Pressate Sampling Results

Location	BOD [mg/L]	TSS [mg/L]	VSS [% dry wt]	Orthophosphate [mg/L PO_4-P]
Septage	1,400	1,760	85.8	7.83
Pressate	110	458	77.5	4.50

2.5.8. Industrial Users

Stone Corral Brewery which discharges to the Richmond WWTF is considered a significant industrial user and is permitted under Pretreatment Discharge Permit No. 3-1560. The Richmond WWTF can receive a monthly average flow of up to 1,000 gpd from Stone Corral Brewery. The Brewery has a monthly average discharge effluent limit of 100 lb/day of BOD. They are required to test for BOD in their discharge 1x/week, TSS 2x/month, and TP 1x/month.

The Town of Richmond should consider evaluating local commercial and industrial users that may be discharging a significant load as one way to understand the influent nutrient load that is historically experienced at the WWTF.

2.6 Condition of Existing Wastewater Treatment Facility

Hoyle Tanner conducted a site visit on May 24, 2023, to the Richmond WWTF to assess the physical condition of WWTF process components and the site. The following section presents the findings of that assessment.

2.6.1. Influent Pumping

Figure 2-11 Influent Wet Well

Raw wastewater from the collection system enters the Richmond WWTF at the influent wet well, located next to the Operations Building, where it is mixed with RAS, pressate, and filter backwash. The roof drains for the Operations Building also drain to the wet well. The wet well is divided into two cells and the operators indicated that only one side of the wet well is being used due to problems with rags, bricks and stones that make their way through the collection system. A level transmitter in the wet well controls the pumping speed of the influent pumps which are located inside the Operations Building basement. The concrete of the wet well structure was not inspected during the field visit and the condition of the structure is unknown.

The influent pumping system consists of two influent pumps located in the lower level of the Operations Building. Influent pumping transfers wastewater from the wet well to the Headworks Building where it flows by gravity through the remaining treatment process. An in-line grinder that was part of the original design has been disconnected. There is a magnetic flow meter on the pump discharge header to measure flows. The operators indicated that they do not believe it accurately measures flow. Influent Pump #1 is driven by a 40 HP motor and was installed in 2023 as a temporary replacement for the existing Influent Pump #1 which had failed. Influent Pump #2 is a vertical mounted, flooded suction centrifugal pump driven by a 25 HP motor. Currently, Influent Pump #2 is not in use as it has a leaking seal and sprays the room when operating. The operators indicated that they have replaced both pump discharge check valves recently.

Figure 2-12 Influent Pumping System

Both pumps are equipped with a variable frequency drive (VFD) which is controlled by the level transmitter in the influent wet well. The operators indicate that they let the wet well back up to the first landing during high flow events and use it for flow equalization. They indicated that they can use the wet well up to this level and back up the influent sewer to within 1 foot of the rim of the upstream manholes in the park, which are elevated for flood proofing.

Access to the pumps is problematic with no overhead hatch to hoist pumps through. The operators noted that they have to bring equipment down three flights of stairs if maintenance is required.

The facility experiences significant large solids carried to the wet well through to the influent pumps including large rocks/bricks, large pieces of wood, rag balls greater in diameter than the pumps can pass. These materials often lead to pump clogging, damage to the pump impellers and emergency maintenance of the pumps. The operators indicated that the Town cleaned out the influent collection sewer up to Bridge Street in 2022 to try to mitigate the amount of debris that enters the wet well and pump suction.

Table 2-4 Influent Wet Well & Pump Existing Design Information

Description	Existing Design
Influent Wet Well	
Dimensions	26’8” L x 11’6” W x 2’ D
Operating Levels	Normal: 2’ Max: approx. 18’
Operating Volume	1,032 gal (at normal operating level)
Influent Pump #1	
Manufacturer/Model	Cornell Pump Co. GNHTA-CSV 40-4
Motor	40 HP
Nameplate RPM	1775 rpm
Influent Pump #2	
Type	Centrifugal, Vertical Mounted, Flood Suction
Motor	25 HP, VFD Driven
Capacity	800 gpm @ 65’ TDH (each)
Flow Meter	
Type/Manufacturer	Magnetic/Siemens/ MAG6000
Max Flow Read	800 gpm

Design Standards

- Convey peak design flow with largest pump out-of-service.

Assessment

The assessment of the major components for the influent wet well, influent grinder, and influent pump station are summarized in Table 2-5, and the major needs are described as follows:

Findings:

- Influent Pump #1 failed and was replaced with a used emergency pump in 2023. The pump is old and only a temporary solution.
- Influent Pump #2 is original to facility. The pump is in poor condition with a leaking seal. This pump is at risk of imminent failure leaving the facility with no pumping redundancy.
- Influent pumps routinely experience problems such as ragging and bricks/rocks causing damage.

- Influent grinder has been removed.
- Hatch not available for pump removal
- Roof drains are connected to the wet well.
- The concrete of wet well structure was not evaluated. It is recommended that a concrete assessment be conducted.
- Additional flow data is required to select influent pump replacements.

Table 2-5 Influent Wet Well and Pump Station Assessment

Item	Rank of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Wet Well			x			1972	20+	Concrete should be inspected.
Influent Pump #1	x					2023 (used)	0-2	Used emergency pump, not permanent solution
Influent Pump #2	x					1972	0-2	Leaking seal, imminent failure
Pump VFDs			x			2005	2-5	
Level Control System			x			2005	2-5	

2.6.2. Headworks Assessment

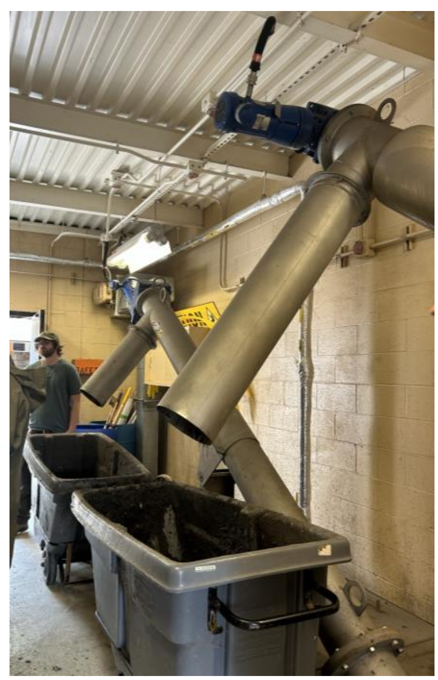


Figure 2-13 Grit Removal

Screening and grit removal consists of a package unit located in the Headworks Building. Screening and grit removal is accomplished using a combination unit that uses a rotary-type mechanical fine screen and an aerated grit removal chamber with a grit dewatering screw to screen inorganics and remove grit from the process stream. Wastewater is pumped to the unit via the influent pumps. Wastewater flows into the unit's screening basket where solids are retained. Screened material is removed from the screening basket and is spray washed to return organics to the process stream. The screened material is then transported up the unit's central screw conveyor, compacted, dewatered, and discharged to storage containers. The wastewater that flows through the screening basket passes directly into a grit removal chamber. Grit settles to the floor of the grit chamber and a grit transport screw moves the settled grit to a lateral sump. A grit dewatering screw transports the settled grit out of the lateral sump and dewaters the grit before it is discharged into storage containers. Air for the aerated grit chamber is fed from a new blower located in the one-room structure located over the aerobic sludge digester tank. Flow from the grit chamber is discharged directly into the first anoxic selector. The headworks can be bypassed by pumping directly to the aeration basins.

The rotary-type fine screen shaft has significant wear and while the screen does catch some rags, the operators indicate that it doesn't work well. The grit auger has significant wear and has been patched over the years. The operators indicated that grit that is captured is not being properly dewatered resulting in a product with a mud-like consistency. The operators have also noted a leak in the stainless steel package unit tank, which they have been patching regularly.

The Headworks Building's gas detection system is inoperable, and the ventilation system is activated by the light switch, indicating occupancy. It could not be determined if the existing ventilation meets the required air changes for this Class 1 Division I Hazardous space.

The existing design information for the screening and grit removal equipment is presented in Table 2-6 on the following page.

Table 2-6 Screening and Grit Removal Design Information

Description	Existing Design
Screening & Grit Removal Package	
Quantity	1
Manufacturer	Lakeside Equipment Corp.
Type/Model	Complete Plant/SO 03-191
Rotary Screen	
Type	Mechanical, Fine Screen
Bar Spacings	3/8”
Motor Power Draw	2 HP
Capacity	2.97 MGD
Screen Openings Spacing	1/2”
Screen Basket Diameter	31”
Grit Chamber	
Number of Units	1
Dimensions	26’ x 3.5’
SWD	9.17’
Capacity (Maximum Flow Rate)	2.97 MGD
Grit Transfer Screw	
Diameter	8”
Motor Drive	1 HP
Diameter	8”
Motor Drive	2 HP
Grit Blower	
Quantity	1
Type	Rotary Positive Displacement Blower
Power Draw	2 HP
Design Capacity	26 scfm

Design Standards

- Bypass Screens: Installations using mechanically cleaned screens or comminution devices should include multiple units or a manually cleaned bypass screen. (TR-16 Standards)
- Manually Cleaned Screens: Unobstructed openings between bars should be 1–2 inches (2.5–5 cm) wide. Manually cleaned screens should be placed on a slope of 30–45 degrees with the horizontal. (TR-16 Standards)
- Mechanically Cleaned Screens: Unobstructed openings between bars are generally 0.25–1.5 inches (0.6–3.8 cm) wide. (TR-16 Standards)
- Velocities: Screen chambers should provide good velocity distribution across and through the screen. Approach velocities in screen channels should be at least 1.3 feet per second at minimum flows (2.0 ft/sec is preferred if possible), or 2.5 ft/sec during diurnal peak flow periods. Approach velocities in screen channels serving combined systems should be at least 3 ft/sec during storm flows. Velocities through openings of mechanically cleaned screens should be 2–4 ft/sec. Velocities through manually cleaned screens should be limited to 1–2 ft/sec. (TR-16 Standards)
- Grit Removal: Grit can be removed in grit chambers or by centrifugal separation of primary sludge. Acceptable grit chambers include aerated, vortex (including induced vortex and multi-tray vortex units), detritus, and horizontal flow (velocity control tanks) units. A single, manually or mechanically cleaned grit chamber with bypass is acceptable for small plants serving sanitary sewer systems. (TR-16 Standards)
- Fire Protection: For coarse and fine screen facilities, grit removal tanks, and pre-aeration tanks that are continuously ventilated at 12 air changes per hour, the entire enclosed space is classified as Class I, Division 1. This space requires a portable fire extinguisher, combustible gas detection system, and hydrant protection.

Assessment

The assessment of the major components for the Headworks is summarized in Table 2-7 on the following page, and the major needs are described as follows:

Findings:

- The stainless tank of the package screening and grit unit is leaking.
- The effectiveness of screening and grit removal is poor.
- Grit auger is worn and has been repeatably patched.
- Grit is not properly dewatering, creating a mud-like consistency.
- Gas detection in the building is inoperable.
- Existing ventilation may not provide adequate air changes.
- No drain in the sump where package unit sits. Operators must use a trash pump to drain if level gets too high.

Table 2-7 Headworks Assessment

Item	Rank of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Rotary Screen		x				2005	2-5	
Grit Chamber		x				2005	10+	
Grit Blower		x				2005	2-5	
Grit Transfer Screw		x				2005	2-5	
Grit Dewatering Screw		x				2005	2-5	
Headworks Building			x			2005	10+	
Ventilation		x				2005	2-5	
Gas Detection	x					2005	0	inoperable

2.6.3. Biological Process

The biological treatment system at the Richmond WWTF consists of anoxic selectors followed by aeration tanks. The first part of the process consists of three anoxic selectors located inside the Headworks Building. The first two anoxic selectors each have a capacity of 3,000 gallons while the third anoxic selector has a 6,000 gallon capacity. Mixing throughout the selectors is achieved with submersible mixers. Upon the time of inspection, the submersible mixer in the first anoxic selector is inoperable and only one of the mixers has a Class 1, Division I explosion-proof motor. Since the site visit, the submersible mixer in the first anoxic selector has been repaired.

Figure 2-14 Biological Aeration Tank

Flow from the third anoxic selector is designed to split between two parallel aeration tanks, each having a volume of 150,000 gallons. Typical operation consists of one aeration tank in operation at a time. The aeration tanks have a fine bubble diffused air system and are fed air from two (2) 25 HP positive displacement blowers, each with a capacity of 450 scfm, located in the Process Building upper level. A single dissolved oxygen (D.O.) probe is provided but is not connected to SCADA for blower operation. The operators indicated they would like one D.O. probe for each tank. The outdoor intake for the aeration blowers has been disconnected due to noise complaints, and air for the blowers is drawn from the septage holding tank in the Operations Building Garage. This air has significant hydrogen sulfide concentration for the air space above a non-aerated sludge holding tank. Blower control is cycled on/off from SCADA; however, D.O. concentration in the aerated basin is not being used to inform the aeration schedule.

The aeration tanks are original to the Richmond WWTF and were constructed in 1972. The concrete of the tank structures was not evaluated during the site visit. The diffusers were not inspected during the site visit, however a uniform bubble pattern in the operating tank suggests that the diffusers are functioning as intended. The operators noted that Aeration Tank #1 was drained and cleaned out recently, however, Aeration Tank #2 has not been cleaned since the upgrade. The operators indicated that it is difficult to balance air in the air header between the tanks using manually operated butterfly valves and would like to see dedicated air lines from each blower to each aeration tank.

Aeration tank effluent is discharged from each tank over a fixed weir into an effluent channel where coagulant for chemical phosphorus removal is applied.

The existing design information for the biological system is presented in Table 2-8.

Table 2-8 Biological Treatment System Existing Design

Description	Existing Design
Anoxic Selector 1 & 2	
Quantity	2
Capacity	3,000 gal (each)
Dimensions	8.25’ x 6.0’
SWD	8.10’
MLSS Target	*Pending*
Anoxic Selector 3	
Quantity	1
Capacity	6,000 gal
Dimensions	16.50’ x 6.0’
SWD	8.10’
MLSS Target	4,000 mg/L
Submersible Mixers	
Quantity	3 (1/anoxic tank)
Type	Submersible Mixers
Manufacturer	Flygt
Power Draw	1.21 HP (each)
Aeration Basins	
Quantity	2
Dimensions	38’ x 38’
SWD	14’
Volume	150,000 gal (each)
Aeration System	
Type/Manufacturer	Fine Bubble Flexible Membrane Diffuser/Sanitaire Aeration System
Quantity	162/tank
Discharge Pressure	7 psig

Blowers	
Quantity	2
Manufacturer	Aerzen USA
Motor	25 HP (each)
Capacity	220 to 450 scfm
Control Strategy	DO Control (not operable)
VFD	Yes

Design Standards

- Liquid depths should not be less than 10 feet or more than 25 feet. (TR-16)
- Aeration systems should be sized for the maximum daily oxygen requirements (considering facility side streams, and seasonal variations in temperature and humidity) while maintaining an aeration basin DO concentration of 2 milligrams per liter. (TR-16)
- Oxygen supply should be designed based on 0.85–1.2 pounds of oxygen per pound of BOD removed plus 4.2 pounds of oxygen per pound of ammonia nitrogen oxidized at maximum daily loading conditions. (TR-16)
- Blower capacity must be based on the air volume required during summer temperature and humidity conditions. The size of motors for centrifugal compressors must be based on summer air flow rates and the coldest expected winter temperature (or other means provided to control mass air flow rate and prevent motor overload). (TR-16)
- Blower controls should be incorporated into the system, providing sufficient ability to meet oxygen demand in the various tanks in service through multiple blowers, variable blower output, dissolved oxygen monitoring, air flow measurement, and automated control valves. (TR-16)
- The size of air piping should be based on maximum expected summer temperatures and in-line velocities of 2,000–2,500 feet per minute. (TR-16)
- Fine bubble, full-floor coverage: 0.12 scfm per square foot of tank area. (TR-16)

Assessment

The assessment of the major components for biological process is summarized in Table 2-9 on the following page, and the major needs are described as follows:

Findings:

- Only mixers in anoxic selectors #2 and #3 are operable, and only one mixer has an explosion-proof motor.
- Aeration Tank #2 has not been cleaned out since the 2005 upgrade.
- Blowers cycle on/off from SCADA but are not controlled by the D.O. probe in the active aeration basin. New D.O. probes needed.
- Blower intake is from a sealed aerated solids holding tank and shows significant signs of corrosion in the garage, outdoor air intake was disconnected due to noise complaints.
- Air balance to the aeration tanks is challenging due to manually operated butterfly valves.
- The concrete tank structures of the anoxic tanks nor aeration tanks were not evaluated. It is recommended that a concrete assessment be conducted.
- Data on mixed liquor suspended solids (MLSS) is not collected at the facility.

Table 2-9 Biological Treatment System Assessment

Item	Rank of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Anoxic Tanks			x			2005	20+	Concrete should be inspected
Submersible Mixers		x				2005	2-5	Mixer #1 is inoperable. Do not have ex-proof motors on all mixers
Aeration Tank concrete			x			1972	20+	Concrete should be inspected
Aeration Tank walkway/railings				x		2005	20+	
Diffusers			x			2005	5-10	
Blowers			x			2005	5-10	Blower intake from septage holding tank is severely corroded.
VFDs			x			2005	2-5	
DO Probes		x				2005	0-2	New probes need

2.6.4. Coagulant Chemical Feed and Storage

There is one (1) 1,500 gallon sodium aluminate storage tank and two (2) positive displacement diaphragm type feed pumps located in the basement of the Process Building, though only one (1) feed pump is in use and the other is in storage. Secondary containment is provided in the form of a 3-ft high concrete retaining wall around the coagulant storage tank. An emergency eyewash station is located next to the coagulant storage tank; however, there is no emergency shower. The original design allowed for coagulant to be pumped to the aeration tank effluent channel and/or into both clarifier effluent pipes prior to the filter units. Typical operation is consistently dripping sodium aluminate through a 2” line into the aeration tank effluent channel and is based on an operator feed rate that is determined by effluent pH levels.

Figure 2-15 Sodium Aluminate Storage

Table 2-10 Chemical Storage & Feed Existing Design Information

Description	Existing Design
Coagulant Storage	
Coagulant	Sodium Aluminate (Alum)
Storage Tank Capacity	1,500 gallons
Containment Volume Required	1,875 gallons
Containment Volume Provided	1,544 gallons
Coagulant Feed – Process Building	
Feed Pump Type	Positive Displacement, Diaphragm Type
Number of Pumps	2 (1 duty, 1 in storage)
Flow Rates	0 – 48 gpd
Application Points	Aeration Tank Effluent Launder

Design Standards

- Redundancy: A minimum of two feed pumps, one duty and one standby, should be provided. (TR-16)
- Location: Chemical feed equipment should be located in a separate, dedicated room to reduce potential hazards and exposure. (TR-16)
- Storage: Space should be provided for at least 30-days of chemical storage under average design conditions. Tanks should have a liquid level indicator, overflow and receiving basin, and secondary containment. Secondary containment should be no less than 125% of the storage tank volume. (TR-16)
- Eye-Wash Fountains and Emergency Showers: Should be provided no more than 25 feet from points of hazardous chemical exposure and supply tempered water at 30-50 gpm and 20-50 psi for 15-30 minutes. (TR-16)

Assessment

The assessment of the major components of the chemical feed and storage are summarized in Table 2-11 on the following page, and the major needs are described as follows:

Findings:

- No emergency shower provided in chemical area.
- Only 1,544 gallons of containment provided, which does not meet design standards. The secondary containment volume required is 1,875 gallons.

Table 2-11 Chemical Feed and Storage Assessment

Item	Rank of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Coagulant Pumps			x			2005	2-5	
Coagulant Storage Tank				x		2005	20+	
Secondary Containment				x		2005	20+	Inadequate volume

2.6.5. Secondary Clarification

Figure 2-16 Secondary Clarifier

Flow from the aeration tank effluent channel can be split between the two rectangular secondary clarifiers by a splitter box with stop gates. Typically, only one (1) clarifier is in operation at a time. Each rectangular secondary clarifier is 50-ft long and 11-ft wide. An access bridge extends the length of the clarifiers. The drives for the chain and flight scrapers are located on the walkway above each clarifier. Effluent troughs have v-notched weirs to control discharge from the clarifiers. Clarified effluent is discharged through a 12" pipe to filtration.

Flight scrapers scrape sludge along the bottom of the secondary clarifiers toward the influent end of each clarifier into a sump. A waste activated sludge (WAS) pump, located in the basement of the Operations Building, removes sludge from the clarifiers and transports it to the septage holding tanks. Return activated sludge (RAS) is drawn from the same sump and conveyed back to the influent wet well via a gravity line. RAS flow is controlled by an electrically actuated pinch valve and flow is measured with a magnetic flow meter located on the RAS line.

The operators noted that they recently had drained both clarifiers, cleaned, and inspected the flight scrapers and both are in good working condition. Operators also noted that there was no grease skimmer on the clarifiers and that grease routinely passes through to the filters causing fouling of the cloth media.

Design information for the existing secondary clarifiers is presented in Table 2-12 on the following page.

Table 2-12 Secondary Clarifier Existing Design Information

Description	Existing Design	Notes
Secondary Clarifiers		
Number of Units	2	
Dimensions	50’ x 11’	
SWD	10’	
Effective Weir Length	108’	
Surface Overflow Rate (w/ 2 units in service)	202 gpd/sf @ ADF (0.222 MGD) 1,047 gpd/sf @ PHF (1.152 MGD)	Meets design standards listed below
Weir Overflow Rate (w/ 2 units in service)	2,056 gpd/ft @ ADF (0.222 MGD) 10,667 gpd/ft @ PHF (1.152 MGD)	Meets design standards listed below
Sludge Collector		
Manufacturer/Model	FMC Corp. / Link-Belt Environmental Equipment/ EE5057-G	
WAS Pumps		
Quantity	1	
Type	Double Disc, Positive Displacement Pump	
Manufacturer	Penn Valley Pump Co. Inc.	
Motor	7.5 HP	
Rating	150 gpm @ 18’ TDH	
Equipment	VFD	
RAS Valve		
Quantity	1	
Type	Pinch Valve	
Manufacturer	Red Valve	
RAS Flow Meter		
Type/Manufacturer	Magnetic/Siemens	
Model	SITRANS FM MAG 6000	
Size/Length	4-inch	
Typical RAS Rate	90% of effluent	

1. Average RAS Rate = 91.4% of Effluent Flow (January 2018 – February 2023)

Design Standards

- Surface Overflow Rate @ PHF
  - Extended Aeration 1,000 gpd/sf, 1,200 gpd/sf Contact Stabilization based on influent only (10 State Standards)
  - 1,140 gpd/sf (TR-16 @ SVI = 150 mL/g, MLSS = 3,000 mg/L)
    - Facility does not have MLSS data, SOR for MLSS = 3,000 mg/L used
- Weir Overflow Rate
  - Maximum weir loading rate of 20,000 gpd/ft at PHF for plants with an average capacity equal to or less than 1 MGD. (10 State Standards)
- Peak Solids Loading Rate @ PDF + Peak RAS Flow
  - Extended Aeration 35 lbs/d/sf, Contact Stabilization 40 lbs/d/sf (10 State Standards)
- Minimum of 12 ft side water depth. (10 State Standards)

Assessment

The assessment of the major components for the secondary clarifiers is summarized in Table 2-13, and the major needs are described as follows:

Findings:

- Clarifiers are in good working condition.
- Concrete tank structures were not evaluated. It is recommended that a concrete assessment be conducted.
- The surface overflow rate @ PHF meets the TR-16 design standard, assuming MLSS = 3,000 mg/L. If MLSS is greater than 3,000 mg/L, then the SOR standard may be exceeded.
- Lack of MLSS data does not allow for accurate solids loading rate analysis on the secondary clarifiers.

Table 2-13 Secondary Clarifier Assessment

Item	Rank of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Clarifier #1 Drive				x		2005	10-15	
Clarifier #2 Drive				x		2005	10-15	
Internal Mechanisms				x		2005	10-15	
Launders, weirs				x		2005	10-15	
Tankage				x		1972	20+	Concrete repaired in 2005 upgrade.
Walkway/Railings				x		2005	20+	

2.6.6. Return and Waste Activated Sludge Pump System

There is one (1) waste activated sludge (WAS) pump located in the Pump Room in the lower level of the Operations Building. The WAS pump is a 7.5 HP Penn Valley Pump double disc, positive displacement pump rated for 150 gpm at 18 ft TDH. The WAS pump moves sludge to the aerated holding basins. The original design also allowed for the WAS to be pumped to the Aerobic Sludge Holding Tank. Operators indicated that many of the valves on the discharge piping are frozen, although they are systematically rebuilding them one at a time. WAS is wasted two (2) times per day for 1-hour intervals and is controlled by a timer. Operators indicate that the pump model is obsolete, and it is hard to get parts.

Figure 2-17 WAS Pump

RAS flows by gravity back to the influent wet well. RAS flow is controlled by an electrically actuated pinch valve and flow is measured with a Siemens magnetic flow meter. Operators indicated that the RAS flow rate is set at 110 gpm. Historical data from January 2018 through February 2023 indicates that RAS is approximately 90% of forward flow.

Table 2-14 RAS and WAS Existing Design

Description	Existing Design
WAS Pump	
Quantity	1
Type	Double Disc, Positive Displacement Pump
Manufacturer	Penn Valley Pump Co. Inc.
Motor	7.5 HP
Rating	150 gpm @ 18' TDH
Equipment	VFD
RAS Valve	
Quantity	1
Type	Electrically Actuated Pinch Valve
Manufacturer	Red Valve
RAS Flow Meter	
Type/Manufacturer	Magnetic/Siemens SITRANS FM MAG 6000
Size/Length	4-inch

Design Standards

- At facilities with an average design flow of 10 MGD or less, waste sludge pumping facilities should normally be designed with a maximum capacity of 25 percent of the average design flow and should provide a minimum flow rate of approximately 80 gallons per minute (to allow velocity of 2 feet per second in a 4-inch diameter pipe) (10 State Standards).

- Suitable devices for observing, sampling, and controlling return activated sludge flow from each settling tank hopper shall be provided (10 State Standards).

Assessment

The assessment of the major components for sludge pumping is summarized in 2-15, and the major needs are described as follows:

Findings:
- The WAS pump is obsolete and parts are hard to obtain.
- The RAS pinch valve and flow meter operate well, however should be inspected for wear.
- There is only one (1) WAS pump with no redundancy provided.
- Multiple WAS plug valves are frozen.

Table 2-15 Sludge Pumping Condition Assessment

Item	Ranking of Existing Condition					Year Installed	Projected Remaining Useful Life	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
WAS Pump			x			2005	2-5	Pump model is obsolete and no redundancy is provided for
WAS plug valves		x				2005	2-5	Many frozen valves
RAS Pinch Valve			x			2005	2-5	
RAS Flow Meter			x			2005	2-5	

2.6.7. Filtration

The tertiary process is located in the Filter/UV Room on the upper level of the Process Building and consists of two (2) steel package tanks, each containing two (2) cloth media disks with 5-micron fabric. Flow is split between the two units by way of a common manifold, though typical operation consists of one (1) unit in operation at a time. Flow can be throttled to each filter by means of a manual butterfly valve on the influent pipe to each filter unit.

Flow entering the filtration tank passes through the cloth membrane by gravity. Filtered water then enters the internal portion of the disk where it flows through the center-tube to an effluent box. Solids collected from the bottom of the tank are pumped via a 3 HP sludge removal pump to a drain line that drains back to the influent sewer. Filtered effluent flows by gravity via a 12” pipe to the UV disinfection system.

Figure 2-18 Filtration Tank

When a backwash cycle is initiated, the filter drive activates which rotates the filter disk at 1 RPM. The first backwash valve then opens, and the waste pump starts. Backwash is recycled to the influent wet well. One (1) of the backwash pumps was replaced in 2023. The other is on order and anticipated to be replaced in fall 2023.

Operators indicated that the filter units have been going into continuous backwash, causing the operators to have to manually waste sludge from the bottom of the filter tank by opening a drain valve. One suspected reason is polymer carryover from dewatering clogging the cloth media surface. Another is grease and solids carryover from the secondary clarifiers fouling the cloth fabric. All cloth media was replaced within the past few years. The operators noted that the filters are not controlled by the SCADA system and they have to manually turn on the second filter unit during high flows.

During the site visit, corrosion of the steel tanks was observed.

The influent weir for the filters is at 311.39 ft. and the top of the filter tank wall is at 313.22 ft, which are both below the 500-year flood elevation of 313.8 ft.

Design information for the existing filtration equipment is presented in Table 2-16.

Table 2-16 Filtration Existing Design

Description	Existing Design
Filter Tanks	
Number of Tanks	2
Manufacturer/ Model Number	Aqua-Aerobic Systems, Inc. Cloth Media Filter ADFP-54X2E-PC
Number of Disks, Total	4 (2, 2-Disk Units)
Dimensions	8' x 9'-2"
SWD	7.84'
Max Water Level	9.61'
Filter Area Provided	53.8 sf/disk 107.6 sf/unit 215.2 sf total
Hydraulic Loading Rate	1.43 gpm/sf @ ADF (0.222 MGD) – one filter unit (2 disks) 7.43 gpm/sf @ PHF (1.152 MGD) – one filter unit (2 disks) 0.72 gpm/sf @ ADF (0.222 MGD) – two filter units (4 disks) 3.72 gpm/sf @ PHF (1.152 MGD) – two filter units (4 disks)
Solids Loading Rate[1]	0.52 lbs TSS/sf/day @ ADF (0.222 MGD) – one filter unit (2 disks) 2.68 lbs TSS/sf/day @ PHF (1.152 MGD) – one filter unit (2 disks) 0.26 lbs TSS/sf/day @ ADF (0.222 MGD) – two filter units (4 disks) 1.34 lbs TSS/sf/day @ PHF (1.152 MGD) – two filter units (4 disks)
Sludge Removal Pump	
Quantity	2
Power Draw	3 HP
Motor Drives	
Quantity	2
Power Draw	½ HP

1. Assuming tertiary influent concentration of 30 mg/L TSS.

Design Standards

- Filter systems should be designed to accommodate peak hourly flows with one unit in backwash mode and to accommodate filters operating at design maximum headloss through filter media. (TR-16)
- Filters should include provisions for automatic bypass in the event of filter media binding as well as provisions for positive flow distribution. (TR-16)
- Effluent filtration systems should include automatic control features to initiate backwash based on intervals of time or on high filter headloss. (TR-16)
- Filter systems should be provided with instrumentation to monitor headloss and turbidity of both filter influent and effluent, and to monitor for filter influent and backwash flows. (TR-16)
- Disc filters should be housed in heated and ventilated enclosures. (TR-16)
- Loading rates at peak hourly flow should not exceed 6.5 gpm/sf of filter surface area. (TR-16)
- A minimum of two filter units should be provided. (TR-16)
- A minimum of 100% of peak hourly design capacity with largest unit out of service. (VTDEC Design Guidance)

Assessment

The assessment of the major components associated with the filtration system are summarized in Table 2-17 on the following page, and the major needs are described as follows:

<u>Findings:</u>

- Filter units have been going into continuous backwash as cloth media is fouled. Operators need to manually waste solids using drain valves at bottom of tank.
- Polymer carry over from dewatering has negatively impacted cloth media performance.
- Significant corrosion on the steel tank, trough, and rusted filter drains was observed.
- Ventilation in the filter room is sealed/shut off and is not functional.
- Filters not connected to SCADA for automatic operation at high flows, must be manually start.
- Suction of settled solids is not functioning properly
- Filter tank wall elevation is below the 500-year flood elevation.

Table 2-17 Filtration System Assessment

Item	Rank of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Filter Tanks			x			2005	5-10	Rust needs to be addressed
Filter cloth		x				2022	0-2	Cloth is routinely fouled.
Sludge Pumps		x				2005	0-2	Sludge pump is not working properly
Backwash Pumps			x			2005/2023	2-5/10+	One backwash pump replaced in 2023
Motor Drives			x			2005	2-5	
Backwash Valves			x			2005	2-5	

2.6.8. Ultraviolet Disinfection

Ultraviolet (UV) disinfection is located in the Filter/UV Room on the upper level of the Process Building. The UV disinfection system consists of a package unit with two (2) banks of UV lamps in series with ten (10) modules in each bank and four (4) lamps in each module. The banks are housed in a stainless-steel channel with transition boxes for flanged pipe attachment at both inlet and outlet ends. A fixed serpentine weir located downstream of the second bank controls the surface water level and directs flow into an outlet transition box. Each bank provides full disinfection treatment at peak flows, providing full redundancy.

The operators indicated that the UV intensity meter was not working, however they have ordered a replacement and will install it soon. The operators also indicated that they have never had a disinfection violation.

It should be noted that the weir in the UV unit is at 308.95 ft and the top of UV channel is at 309.54 ft. These elevations are below the 100-year flood elevation of 310.5 ft and the 500-year flood elevation of 313.8 ft.

The design information for the existing UV disinfection system is presented in Table 2-18 on the following page.

Table 2-18 UV Disinfection Existing Design Information

Item Description	Existing
UV Disinfection Design Conditions	
Average Daily Flow	0.222 MGD
Peak Hourly Flow	1.0 MGD
TSS	10 mg/L
UV Dose @ UVT 60% and Peak Flow	36,724 uWs/cm^2
Effluent Standards	77/100 ML E. coli
UV Disinfection System	
Manufacturer / Model	Trojan System / UV3000 PTP
Number of UV Banks	2 (1 duty, 1-stand-by)
Number of UV Modules (Total)	20 (10/bank)
Number of Lamps per Module	4
Total Number of UV Lamps	80
Liquid Depth	12.6”
Width	30”
Length	23”-2”

Design Standards

- Dosage Monitoring: Each UV module should be equipped with a UV intensity meter responding only to light between 2,525 and 2,550 angstroms. The sensing device for this meter should be fixed at the area of minimum expected intensity. The sensor should be installed within a quartz sleeve. (TR-16)
- Contact Period: Sufficient contact time is required in a UV reactor to provide the established design dose at the delivered UV intensity under peak flow conditions.
- Control Equipment: Each UV module should activate a local and remote alarm signal when the UV intensity drops to 80 percent of original output. A spare PLC processor with a current program should be available.
- UV Dose: The system will provide a minimum UV dose of at least 30,000 microwatt-seconds per square centimeter at peak flow. (VT UV Disinfection Standard)
- Open Channel Units: For open channel units, at least two banks of lamps shall be provided, which operate in series. The multiple open channel units shall cumulatively provide at least the minimum required dosage at the facility’s peak flow rate. (VT UV Disinfection Standard)

Assessment

The assessment of the major components for the disinfection system is summarized in Table 2-19 on the following page and the major needs are described as follows:

Findings:

- The UV intensity meter is not working, however a replacement has been ordered and will be installed in the near future.

- UV top of channel wall elevation is below the 100-year and 500-year flood elevations.

Table 2-19 UV Disinfection System Assessment

Item	Rank of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
UV Disinfection System			x			2005	5-10	May need new controls.
Stainless Steel UV Channel				x		2005	20+	Top of channel is below the 100-yr flood elevation

2.6.9. Effluent Flow Measurement

Flow leaving the UV disinfection system flows from the UV outlet transition box through a 12-inch pipe into the effluent wet well. Effluent flow measurement is achieved by the use of a 90° v-notch weir and an ultrasonic level detector in the wet well. The 90° v-notch weir can accurately measure up to 1.616 MGD with 1.0 feet of head above the bottom of the v-notch according to Isco Open Channel Flow Measurement Handbook, 3rd Edition. Historical operating data suggested that the open channel Sigma 980 effluent flow meter seems to max out at 0.66 MGD, which would indicate a calibration issue or a malfunctioning flow meter. To solve this issue, the effluent flow meter was corrected and recalibrated. Additional effluent flow data collection beginning in October 2023 will help inform design criteria.

Figure 2-20 Effluent Wet Well & V-Notch

The TR-16 Standard is to accurately measure peak flows at the 25-year flood elevation and protect against the 100-year flood. As discussed in Section 1.4.2, the 100-year and 500-year flood elevations at the Richmond WWTF site are EL. 310.50 ft and EL. 313.77 ft, respectively. The invert of the v-notch weir is EL. 306.14 ft, which is below the updated 100-year flood elevation. A check valve is located on the effluent line to prevent water from hydraulically backing up the outfall during a 100-year or 500-year flood event. During the recent July 2023 flooding event, emergency use of a sump pump to lift flows to an external discharge point was used to prevent flooding of the Filter/UV Room. While the sump pump was able to keep up with flows during the most recent flooding event, it has not been properly sized to handle peak flows and provide redundancy.

A plant water system is not provided, and operators noted the WWTF uses significant potable water for screening and dewatering.

Effluent sampling is accomplished by an auto sampler located adjacent to the effluent channel that takes 24-hour composite samples from the effluent sump.

Table 2-20 Effluent Flow Measurement Existing Design Information

Item Description	Existing
Control Device	90° V-Notch Weir
Bottom of V-Notch Weir Elevation	306.13-ft
Measurement	Ultrasonic Flow Meter
Manufacturer/Model	Sigma 980
Flow Meter Capacity	Min: 0.03 MGD at 0.2' above v-notch Max: 1.616 MGD at 1' above v-notch Recorded Max: 0.66 MGD[1]

Notes:
1. Max effluent flow from historical operating data January 2018 to February 2023

Assessment

The assessment of the major components for the effluent flow measurement system is summarized in Table 2-21 on the following page, and the major needs are described as follows:

Findings:
- The effluent flow meter did not appear to be accurate based on recorded historical flow data. Since then, the meter has been corrected and recalibrated.
- The effluent weir elevation is at 306.14 feet which is below the 100-year flood elevation of 310.50 feet and will impact flow measurement accuracy during a flood event.
- The effluent wet well top of wall elevation is at 313.30 ft, which is below the 500-year flood elevation of 313.8 feet.
- Effluent check valve and sump pump downstream of weir is an emergency fix and not sized for peak flows or redundancy.
- No plant water system is provided.

Table 2-21 Effluent Flow Measurement Assessment

Item	Ranking of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
90° V-Notch Weir				x		2005	20+	
Ultrasonic Flow Meter	x					2005	0-2	Flow meter was recently corrected and recalibrated.
Emergency effluent sump pump		x				2007	2-5	Not sized for peak flows. No redundancy.
Effluent Sampler			x			2005	5-10	

2.6.10. Outfall

Disinfected effluent leaves the effluent wet well by means of a 12” pipe which flows to Manhole No. 13 prior to being discharged to the Winooski River through an 18” reinforced concrete outfall pipe. A check valve was added to the outfall after Tropical Storm Irene. This and the temporary provision of using a sump pump in the effluent well to discharge effluent flows to a higher elevation of the Winooski River prevented flooding of the lower level of the WWTF during the July 11, 2023 flood event.

The outfall was not observed during the site visit and therefore the condition could not be assessed.

Assessment

The assessment of the major components for the outfall is summarized in Table 2-22, and the major needs are described as follows:

Findings:

- Condition assessment of the existing outfall was not performed.

Table 2-22 Outfall Assessment

Item	Ranking of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
18” Outfall to Winooski River						1972	unknown	A condition assessment of the outfall was not performed.

2.6.11. Septage Receiving Facilities

The Richmond WWTF has the ability to accept up to 2,000,000 gallons of septage per year. Septage receiving facilities consist of a septage receiving unit located on the upper level of the Processing Building with two (2) aerated holding basins below. The septage receiving unit consists of a cylindrical bar screen complete with screen basket, rotating rake, cleaning comp, screw conveyor, dewatering screw, screenings press with drive unit mounted in a stainless-steel tank, tank spray wash system, motorized inlet valve, and liquid level sensing system. Septage haulers manually record the volumes discharged. The Town has indicated that they would prefer to have a flow meter on the discharge to allow for more accurate record keeping.

Figure 2-21 Septage Receiving Unit

Septage, along with WAS, discharges directly to either of the two aerated holding basins, each with a capacity of 23,000 gallons, located below the floor of the upper level of the Processing Building. Each tank is equipped with a diffused fine bubble aeration system and fed air from a 10 HP, 200 scfm positive displacement blower with a VFD located in the basement of the Process Building. Alternatively, the diffused aeration system can also be fed from the aeration tank blowers.

A single sludge transfer pump is provided to transfer sludge from either of the two tanks to the aerobic sludge holding tank or between the two tanks. The operators indicated that the pump runs well, however the model is obsolete, and it is hard to get replacement parts. The aerated holding basins were designed to be decanted to increase the solids concentration using manually operated decant valves and ports on each basin that drain to the influent sewer; however, the operators indicated that decant has never been used and has since been disconnected.

The inside of the holding tanks was not accessed and therefore not evaluated. During the time of the site visit, operators indicated that there is about 4 feet of solids accumulated in the tanks and they have been experiencing issues with the diffusers in the aerated holding basins. They indicated that as the tanks are considered a confined space, they will need to contract with an outside firm to do the cleaning. Since the visit, solids have been removed from both holding tanks.

The ventilation system within the septage receiving area in the upper Process Building is inoperable. Additionally, vents have been cut into the electrical room housing the motor control center (MCC) in the septage receiving garage, allowing heat from the electrical room to vent into the septage receiving area. The septage receiving area is classified as Class I, Division 1, hazardous space per NFPA 820. Equipment located in this space and shared air space are required to have explosion proof motors.

The design information for the existing septage receiving facilities is presented in Table 2-23 on the following page.

Table 2-23 Septage Receiving Facilities Existing Design Information

Description	Existing Design
Septage Receiving Unit	
Manufacturer	Lakeside Equipment Corp.
Hydraulic Capacity	400 gpm (up to 3% solids)
Aerated Holding Basins	
Quantity	2
Dimensions	21.58' x 14.58'
SWD	10'
Capacity	23,000 gal (each)
Aerated Holding Basin Tank Blower	
Number of Blowers	1
Location	Process Building Basement
Blower Type	Positive Displacement
Power Draw	10 HP
Capacity	200 scfm @ 5 psig
VFD	Yes
Aerated Holding Basin Aeration System	
Diffusers	Fine Bubble, Membrane Type
Manufacturer	Sanitaire Aeration System
Sludge Transfer Pump #1	
Quantity	1
Location	Process Building Basement
Type	Double Disc, Positive Displacement
Power	7.5 HP
Capacity	150 gpm @ 18 ft TDH
VFD	Yes

Design Standards

- Without pretreatment or wastewater process modifications, septage addition should not exceed 2-5 percent of actual wastewater flow on any day and must be slowly metered into the wastewater stream during periods of the day with higher flow. (TR-16)
- The receiving station area must collect and contain any septage spilled during unloading. Equipment and space for washdown must be provided, including water with ample pressure, hose, and spray nozzle. (TR-16)
- Receiving facilities should provide for the containment, collection, and treatment of odors. (TR-16)
- A sludge storage system should be equipped with mixing devices to prevent separation of solids and to provide a more uniform feed-to-dewatering device. Aeration may be required if the sludge is unstabilized. (TR-16)
- A minimum mixing and oxygen requirement of 15-20 cfm per 1,000 cf of tank volume is recommended for WAS with the largest blower out of service. If diffusers are used, the nonclog type is recommended, and they should permit continuity of service. If mechanical aerators are used, a minimum of 1.0 HP per 1,000 cf should be provided. (TR-16)

- Pumps for handling the septage should be non-clogging and capable of passing 3-inch diameter solids. (10 State Standards)
- Sludge withdrawal piping should have a minimum diameter of 8 inches for gravity withdrawal and 6 inches for pump suction and discharge lines. For dilute sludges, the available head should provide a velocity of at least 3 feet per second at the design flow. (TR-16)
- Class I Division 1 sludge storage wet wells, pits, and holding tank spaces require combustible gas detection system, portable fire extinguisher, and hydrant protection. (NFPA 80)

Assessment

The assessment of the major components for the septage receiving facilities is summarized in Table 2-24, and the major needs are described as follows:

Findings:

- The septage receiving unit is reaching the end of its anticipated useful life and was found to be in poor condition, the bearings are gone, and the auger has significant wear.
- Septage receiving unit must be manually cleaned out by operators.
- The Town is interested in a key card system with a flow meter for recording septage hauler discharges.
- There is significant solids accumulation in each aerated holding basin, causing clogging issues with the diffusers.
- Aerated holding basins are a confined space.
- The aerated holding basins decant system has been disconnected.
- Plug valves on septage holding tanks are not operable.
- There is no redundancy provided for the sludge transfer pump. The pump model is obsolete making parts hard to obtain.
- The ventilation system is not operable in the garage where the septage receiving unit is located. Vents have been cut into the electrical room providing airflow between the two spaces.
- The septage receiving room is not compliant with NFPA 820 hazardous classifications.

Table 2-24 Septage Receiving Facility Assessment

Item	Rank of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Septage Receiving Unit			x			2005	2-5	
Aerated Holding Basins				x		2005	20+	Significant solids accumulation
Septage Receiving Diffusers	x					2005	0-2	Diffusers are clogged by accumulated solids
Tank Decant	x							Decant has been disconnected.
Septage Receiving Blower			x			2005	5-10	
Sludge Transfer Pump #1			x			2005	2-5	No redundancy provided. Model is obsolete.

2.6.12. Aerobic Sludge Holding Tank

Sludge from the two aerated holding basins is pumped by the sludge transfer pump to the aerobic sludge holding tank. The aerobic sludge holding tank can hold 34,000 gallons and is equipped with coarse bubble diffusers. Air to the diffuser system is supplied by a 10 HP, 170 scfm blower located in the one-room structure located over the aerobic sludge holding tank, which also houses the grit blower. The intake for the blower is internal to the building.

The aerobic sludge holding tank was not drained to evaluate the condition of the concrete tank, nor the existing diffusers. The operators noted that the tank needs to be cleaned as there is an accumulation of rocks, grit, and rags in the tank which are creating plugging issues with the downstream sludge transfer and feed pumps. The non-uniform bubble pattern observed on the surface of the tank contents during the evaluation indicates that some of the diffusers may be clogged and inoperable.

Figure 2-22 Aerobic Sludge Holding Tank

Table 2-25 Aerobic Sludge Holding Tank Existing Information

Description	Existing Design
Aerobic Sludge Holding Tank	
Dimensions	38’ x 8.5’
SWD	14’
Capacity	34,000 gal (each)
Aerobic Sludge Holding Aeration System	
Diffusers	Coarse Bubble
Type	unknown
Number	unknown
Air Required for Mixing	140 scfm
Blowers	
Number of Blowers	1
Manufacturer	Aerzen USA Corp.
Blower Type	Positive Displacement
Capacity	170 scfm at 7 psi
Motor	10 HP
VFD	Yes

Assessment

The assessment of the major components for the aerobic sludge holding tank is summarized in Table 2-26, and the major needs are described as follows:

Findings:

- The concrete tank structure was not able to be evaluated. It is recommended that a concrete assessment be conducted.
- Operators noted that the tank needs to be cleaned as there is an accumulation of rocks, grit, and rags.
- Observance of non-uniform buddle pattern at surface indicates that some diffusers in the aerobic sludge holding tank are not operational and need to be changed.

Table 2-26 Aerated Sludge Holding Assessment

Item	Ranking of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Aerobic Sludge Holding Tank			x			1972	20+	Concrete assessment needed. Tank requires cleaning due to accumulation of rags/rocks/grit.
Diffusers Coarse Bubble	x					1972	0-2	Some diffusers appear clogged. Need to be replaced.
Blower			x			2005	5-10	

2.6.13. Dewatering Facilities

A separate Dewatering Building is located at the far side of the site and houses sludge pumping equipment, a sludge day tank, rotary press, polymer feed system, dewatered sludge cake conveyors, and a sludge trailer. A site visit by the manufacturer of the dewatering equipment, Fournier, was made on September 30, 2023, and the summary report of the technician's findings is presented in Appendix B.

Figure 2-23 Sludge Day Tank

When the dewatering equipment is in operation, sludge is removed from the aerobic sludge holding tank using the sludge transfer pump located in the Dewatering Building which originally discharged through a sludge grinder to the sludge day tank, also located in the Dewatering Building. The sludge grinder has since been removed by the operators.

The sludge day tank is a 3,000-gallon fiberglass tank which supplies sludge to the dewatering process. The tank has an access hatch at the top and is equipped with a mixer, to keep the contents mixed, and a level detector that is integrated into the controls of the sludge transfer pump and the sludge feed pump. The operators have noted that the sludge day tank needs to be cleaned out as the sludge feed pump routinely gets plugged from rags passing through. The operators also noted that there is no way to remove the mixer from the day tank as there is no headroom above the tank for the long shaft.


Figure 2-24 Rotary Press

The sludge feed pump pumps sludge directly to the dewatering equipment, specifically the flocculator, where sludge is mixed with polymer from the polymer feed system. This pump is directly controlled by the rotary press PLC. The inspection report noted that sludge gravity flows through the pump from the day tank, either indicating that new pump interior parts are needed, or sludge valves need replacing.

A two-channel rotary press, located in the Sludge Dewatering Building, provides sludge dewatering using rotating perforated screens. Sludge is fed from the sludge day tank into the flocculator where polymer is injected into the sludge via a polymer feed pump. Conditioned sludge then enters a manifold and is divided into a series of circular channels. In the channel, sludge is sandwiched between two (2) low speed rotating perforated screens. The operators noted that maintenance on the rotary press has lapsed, resulting in operational issues. It was noted that regular cleaning is necessary every six months at a minimum. Currently, only one (1) channel in the two (2) channel rotary press is operational. The manufacturer's report indicates that the one inoperable channel has significant damage to it's screen, however all four screens are very warn.

The polymer feed system is located in the Sludge Dewatering Building and injects polymer to the sludge dewatering flocculator. The polymer feed system consists of a package system that provides for the mixing of polymer with water in a mixing chamber and then uses a chemical metering pump to transfer dilute polymer to a batch tank, where the polymer can age. A progressive cavity pump equipped with a VFD then feeds the dilute polymer to the Rotary Press flocculator. The polymer feed system is automatically controlled by the rotary press control panel.

Figure 2-25 Rotary Press Conveyor System

A conveyor system is used to transfer dewatered sludge cake from the rotary press to a roll-off container where it is taken off-site for final disposal. The conveyor consists of three parts: a discharge conveyor, the inclined conveyor, and the distributing conveyor. All three are hollow flight screw type conveyors. The discharge conveyor runs horizontally for 9 feet from the discharge chutes of both channels of the rotary press to the inclined conveyor. The inclined conveyor is 20 feet in length with an approximate slope of 37 degrees. This conveyor brings dewatered cake up to the middle of the distributing conveyor. The distributing conveyor is approximately 32 feet in length and is elevated over the parked roll-off container. The distributing conveyor has openings at either end, and two pneumatically operated slide gates to distribute dried cake evenly within the container. The hollow flight conveyor within the distributing conveyor has a reversing motor to allow cake to be discharged to either end. The operators noted that both the auger and screw were replaced in the Spring of 2022, and that the screw broke again in December of 2022. During the repair, staff noted that the liner needed to be replaced.

Figure 2-26 Inclined Conveyor

During the site visit, it was noted that the Dewatering Building does not have proper ventilation. The operators have noted there are significant odors issues and a build-up of ammonia gases, especially in the winter season when garage doors are kept closed. At the time of the site visit, operators noted that the only control for the overhead door is inside the building and across the room from the main entrance. Since then, a switch has been installed on the exterior of the building to allow operators the ability to open the overhead door and vent gases without having to enter the building. A fair amount of corrosion of metal components in the Dewatering Building was also observed during the site visit.

Figure 2-27 Dewatering Building

Additionally, the floor of the dewatering building does not have a rear floor drain, causing difficulties managing drainage from the sludge trailer. Operators expressed interest in a water meter on the water line to the dewatering building to monitor the cost of potable water used during dewatering, as well as a dedicated electrical meter for the building to assess power consumption.

Table 2-27 Sludge Dewatering Existing Design Information

Description	Existing Design
Sludge Transfer Pump (From Digester to Day Tank)	
Quantity	1
Type	Double Disc, Positive Displacement
Manufacturer	Penn Valley Pump Co. Inc.
Capacity	50 gpm
Max TDH	25.5 ft
Min TDH	11.5 ft
Max Suction Lift	19 ft
Min Suction Lift	5 ft
Motor	5 HP
VFD	Yes
Sludge Day Tank	
Quantity	1
Capacity	3,000 gal
Type	Fiberglass Tank
Mixer	Shaft impeller
Sludge Feed Pump (From Day Tank to Rotary Press)	
Quantity	1
Type	Double Disc, Positive Displacement
Manufacturer	Penn Valley Pump Co. Inc.
Flow Rate	0-60 gpm
Max TDH	20 ft
Min TDH	0 ft
Motor	3 HP
VFD	Yes
Control	Rotary Press PLC
Sludge Dewatering	
Solids Feed[1]	Volume Pressed: 26,445 gpd
Solids Concentration[2]	WAS: 9,300 mg TSS/L Septage: 1,760 mg TSS/L
Average Solids Feed %[3]	0.26%
Sludge Disposal	98 wet tons/month[1] 28.6 dry tons/month[3]
Manufacturer/Model	Fournier Industries Inc. 2-900/2000CV
Type	Rotary Press with Flocculator
Number of Units	1
Channels	2
Channel Diameter	36”
Motor	7.5 HP
Through-put[4]	100 lbs/hr/channel
Dewatered Sludge Cake Average Solids %[5]	29.2%

Description	Existing Design
Flow Meter	
Type/Manufacturer	Magnetic/Endress and Hauser
Size	3”
Conveyor System	
Type	Hollow flight screw conveyor
Discharge Conveyor Length	9’
Inclined Conveyor Length	20’ @ 37 degrees
Distributing Conveyor Length	32’
Slide Gates	Pneumatically operated
Polymer Feed System	
Type	Liquid polymer activation, dilution, and feed system including mixing chamber and feed pump
Storage	55-gallon drums
Chemical Feed Pump Type	Positive Displacement, Diaphragm Type
Number of Pumps	1
Metering Pump Flow Range	0.4 – 8.0 gph
Dilution Water Flow Range	120 to 1,200 gph
Application Points	Sludge Dewatering Flocculator

Notes

1. Average is from historical operating data from January 2018-February 2023.
2. Concentration is from 9/21/2022 sampling event.
3. Calculated from historical operating data and sampling event.
4. From Rotary Press Basis of Design
5. Cake solids is average of five dewatered sludge testing results from 2022.

Design Standards

- The operating period should not usually exceed 30 hours per week, which allows one-shift operation with time for chemical makeup, cleanup, and delays. (TR-16)
- Hydraulic loading rate of a single drive unit ranges from 5 to 250 gpm, with a maximum loading rate of 50 gpm per channel. (TR-16)
- Facilities should be provided for ventilation of the dewatering area. (TR-16)
- Floors should be pitched and drained for cleaning purposes and should be slip-proof. (TR-16)
- Volume of containment area be no less than 125% of the tank volume for hazardous or corrosive chemicals. (TR-16)

Assessment

The assessment of the major components of the dewatering system are summarized in Table 2-28, and the major needs are described as follows:

<u>Findings:</u>

- Significant wear and damage to the screens of the dewatering press screens was observed by the manufacturer’s technician during the September 30th site visit. Significant repairs are warranted to keep dewatering operations functional.

- The grinder has been removed from the sludge transfer line.
- No ventilation is provided in the sludge dewatering room resulting in accumulation of corrosive gases and odors within the building.
- No gas detection system in building.
- Day tank needs to be cleaned.
- Mixer in the sludge day tank has a long shaft and there is no way to remove.
- Lack of floor drain at rear of sludge trailer makes it difficult to manage drainage from trailer.
- Polymer system defaults to over-pumping when power is lost.
- Operators noted that the hollow flight augers of the conveyors are deteriorated and need replacement, as well as the liner.
- A check valve on the line between the aerobic sludge holding tank and day tank was added and is cleaned daily.
- The feed pump to the dewatering press plugs frequently.

Table 2-28 Dewatering Facilities Assessment

Item	Rank of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Sludge Transfer Pump			x			2005	2-5	
Sludge Day Tank			x			2005	2-5	Needs cleaning. No way to get mixer shaft out of tank.
Sludge Feed Pump		x				2005	0-2	Plugs frequently.
Rotary Press	x					2005	0-2	Maintenance and repairs required to remain operational
Flocculator	x					2005	0-2	Leaking at seal shaft
Flow Meter			x			2005	2-5	
Conveyor System	x					2005	0-2	Screw auger is shot. Liner has been replaced several times
Dewatering Building		x				2005	2-5	Severe corrosion inside building due to build-up of corrosive gases

2.6.14. Operations Building

The Operations Building was built in 1972 and has undergone modifications and renovations in the 2005 upgrade. The upper level houses the office and laboratory, bathroom, and break room, while the lower level houses the RAS and WAS pump systems, influent pumping, and mechanical room. The roof drains of the Operations Building are connected directly to the influent wet well. Currently, there is no access hatch to reach the influent pumps located underneath the laboratory, and the operators expressed a need for better access.

The operators indicated that the control on the boiler have all been replaced and they are continuously having to repair it. They also noted that HVAC systems throughout the facility need to be completely replaced.

As part of the 2005 upgrade, an extension to the existing Operations Building was constructed to include the garage that houses septage receiving, workshop space, and an electrical room containing the motor control center (MCC).

Observations noted during the May 2023 site visit include the following:

- Boiler needs to be replaced.
- HVAC systems need to be replaced
- Laboratory refrigerator is inoperable.
- Operations Building roof drains to the influent wetwell.
- Doors and windows are original to plant.
- No method to hoist influent pumps from basement.
- Washer/dryer discharges to the floor drain and vents to the building interior.

2.6.15. Site

As previously mentioned in Section 1.4.2, the new flood elevation determination by FEMA in 2014 resulted in a 2 foot increase in the 100-year flood elevation. The new **100-year** and **500-year** flood elevations at the location of the Richmond WWTF are **310.50 feet** and **313.77 feet**, respectively.

The ground elevation around the Process Building, which houses the chemical storage, UV disinfection, and filter units, is **313.50 feet**. While the Process Building will be protected from a 100-year flood event, there is risk of flooding during a 500-year flood event.

The ground elevation of the Operations Building is at approximately **314.00 feet** and is protected from the 100-year and 500-year flood events.

Other site observations made during the May 2023 site visit include the following:

Stand-by Generator:
There is a 150-kW diesel engine driven generator, rated 150 kW at a governed speed of 1,800 rpm providing 480 volt, 3-phase stand-by electrical service located outside of the Blower Building. The generator has an approximately 390-gallon skid-mounted No. 2 diesel fuel tank capable of providing 24 hours of operation. When a loss of utility power occurs, the amount of load placed on the stand-by generator is limited to loads deemed critical for maintaining operation, including: one (1) influent pump, screening & grit removal, anoxic selector submersible mixers, one (1) aeration tank blower, secondary clarifiers, RAS pinch valve, RAS flow meter, two (2) filter units, UV disinfection system, influent & effluent flow metering, automatic samplers, SCADA system & PLC, heating, and lighting.

While the operators indicated that the generator operates well, it is noted that there is a need for an electrical assessment for the Richmond WWTF as one does not currently exist.

Plant Water System:
The Richmond WWTF does not have a plant water system and Town water is used throughout the site. The operators have expressed a desire to implement a plant water system to reduce the Town water consumption and cost.

Former Sludge Drying Bed: The former sludge drying bed has been converted to a storage area. There is no electricity to this area and the operators indicated wanting to supply power and heat to the garage to allow for other opportunities for use.

Site Fence: The facility is surrounded by a security fence with access gates at the driveway entrance and behind the Storage Garage. Feedback from the trucking company that delivers the sludge trailer indicates the need for a wider gate opening at the main entrance.

Assessment

The assessment of the major components is summarized in Table 2-29 for the WWTF site.

Table 2-29 Site Assessment

Item	Ranking of Existing Condition					Year Installed	Projected Remaining Life (years)	Notes
	Poor		Fair	Good				
	1	2	3	4	5			
Process Building					x	2005	20+	Finished floor at 313.50 is below the 500-yr flood elevation
Generator				x		2005	10-15	An electrical assessment of the generator is recommended.
Storage Garage	x					?	0-2	Installation year unknown – between 1972 and 2005. No electricity.
Security Fence & Entrance Gate			x			2005	5-10	Wider main entrance gate needed for sludge trailer
Yard Hydrants & yard piping			x			1972/2005	2-5	Addition yard piping added in 2005

2.6.16. WWTF Electrical System and Instrumentation

General

A detailed electrical and instrumentation review by an electrical engineer was not included in the scope of work. General observations are provided in the following section.

Applicable Codes and Standards

The electrical systems design for the refurbishment of the wastewater treatment facility must meet applicable State of Vermont and Fire, Electrical and Energy codes. The electrical systems design for the planned upgrades at the WWTF will consider the following codes and standards:

- Vermont Fire and Building Safety Code (2015)
- The National Electrical Code (NFPA 70) (2020)
- The National Fire Alarm and Signaling Code (NFPA 72) (2013)
- Vermont Access Rules (2012)
- Americans with Disabilities Act Accessibility Guidelines (ADAAG), July 26, 1991
- Vermont Commercial Building Energy Standards (CBES) (2020)
- NFPA 1 (2015), Fire Code
- NFPA 101 (2015), Life Safety Code
- IBC (2015), International Building Code
- NFPA 37 (2010), Standard for the Installation and Use of Stationary Combustion Engines and Gas Turbines
- NFPA 110 (2013), Standard for Emergency and Standby Power Systems
- NFPA 820 (2012), Standard for Fire Protection in Wastewater Treatment and Collection Facilities
- Technical Report #16 (TR-16) Guides for the Design of Wastewater Treatment Works prepared by the New England Interstate Water Pollution Control Commission.

2.6.17. Existing Conditions

Motor Controllers

The motor control center (MCC) is located in the Operations Building Garage. Outside ventilation has been dismantled and vents have been cut into the room connecting the air space with the Operations Garage that is classified as a Class I, Division I hazardous space as it contains the septage receiving unit.

PLC/SCADA System/Alarm Communications

The central plant PLC/SCADA system was installed in the 2000 upgrade.

LCS recently repaired the dialer from the alarm system to have the alarm send texts to staff cell phones until the alarm is cleared in SCADA. The Town intends to have LCS set up to alarm system to call the pager when there is an alarm condition and the cell service is not working.

2.7 Condition of Collection System

Pending

2.8 Condition of Pump Station

Pending

2.9 Financial Status of Any Existing Facilities

2.9.1. Wastewater Revenue

The Town of Richmond's Water and Wastewater Budget pays for wastewater treatment, pump station and force main conveyance operation, maintenance, and capital costs within the Town. A detailed Water and Wastewater Budget for Fiscal Year (FY) 2024 is presented in Appendix C. Table 2-30 summarizes budgeted wastewater revenue generated to support existing municipal wastewater facilities.

Table 2-30 Richmond Wastewater Revenue

Description	FY 22 Budget	FY 22 Actual	FY 23 Budget	FY 24 Budget
Wastewater Revenue				
Sewer User Receipts	$357,337	$373,213	$361,326	$292,874
Hook-Up Fees - Sewer	$1,000	$3,213	$1,000	$1,000
Net Interest on Checking Account	$1,200	$1,998	$1,500	$14,000
Fund Balance Usage	--	--	$48,394	--
Septage Receipts	$430,000	$483,577	$460,000	$550,000
Wastewater Revenue Subtotals	**$789,537**	**$862,001**	**$872,220**	**$857,874**

2.9.1.1. User Rate Structure

Richmond's sewer user rates are summarized in Table 2-31. The average annual fee for the average homeowner is $565.73 per year.

Table 2-31 Richmond Wastewater User Rate Structure

Sewer User Type	User Rate
Commercial	$475.05/Annual Fee $16.17/1,000 gal treated
Residential	$169.72/Annual Fee $18.87/1,000 gal treated

2.9.2. Wastewater Expenses

Table 2-32 summarizes budgeted wastewater expenses for Richmond's municipal wastewater facilities.

Table 2-32 Richmond Wastewater Expenses

Description	FY 22 Budget	FY 22 Actual	FY 23 Budget	FY 24 Budget
Wastewater Expenses				
Wastewater Administration Expenses	$251,622	$264,477	$305,046	$344,635
Wastewater Operating Expenses	$329,900	$393,699	$359,900	$416,800
Wastewater Capital Expenses	$208,015	$207,955	$207,274	$96,439
Wastewater Expenses Subtotals	**$789,537**	**$866,131**	**$872,220**	**$857,874**

2.10 Water/Energy/Waste Audits

There are no water, energy, and waste audits for the Richmond WWTF.

3. NEED FOR PROJECT

3.1 Health, Sanitation and Security

The reliable function of the wastewater treatment system is required to protect public health and sanitation by meeting the requirements of the Richmond WWTF NPDES discharge permit. The Richmond WWTF has effectively met its permit requirements over the past 5 years and has not had any exceedances during this time.

3.2 Aging Infrastructure

Age related needs were identified in the assessments completed in Section 2.5.8 for the Richmond WWTF. Some items requiring upgrade are original (1972) to the plant while others were implemented during the 2005 upgrade. Equipment has reached the end of its useful life and upgrade is recommended. Table 3-1 summarizes the needs for the WWTF.

Table 3-1 Summary of Major Deficiencies

Item Description	Projected Date of Required Upgrade				
	<2 Years	2 to 5 Years	6 to 10 Years	11 to 15 Years	20+ Years
Influent Pump #1	✓				
Influent Pump #2	✓				
Grit Removal		✓			
Anoxic Tank Mixers & VFD’s		✓			
Aeration Tank Diffusers & Blowers			✓		
Clarifier Drives, Internal Mechanisms, Launders, & Weirs, & Walkway/Railings				✓	
Clarifier Tankage					✓
WAS Pump		✓			
RAS Flow Meter		✓			
Filtration Backwash Pump, Motor Drives, & Backwash Valves		✓			
Filtration Sludge Pumps & Filter Cloth	✓				
Filter Tanks			✓		
UV Disinfection System			✓		
Stainless Steel UV Channel					✓
Aerobic Sludge Holding Tank					✓
Aerobic Sludge Diffusers	✓				
Aerobic Sludge Blower		✓			
Sludge Day Tank, Transfer Pump, Feed Pump & Flow Meter		✓			
Rotary Press & Flocculator	✓				
Conveyor System	✓				
Dewatering Building		✓			

In addition to the summary of the major deficiencies at the Richmond WWTF, an emphasize is placed on the following immediate needs:

- Influent Pumps: the influent pumps are at risk of imminent failure and immediate replacement is recommended.

3.3 Reasonable Growth

3.3.1. Existing and Future Wastewater Flows

A proposed sewer service expansion area exists along the east side of West Main Street in Richmond, VT from Richmond Village to the I-89 interchange. Wastewater in the area is currently treated by individual on-site systems which is limiting growth potential. The expansion project is split up into three (3) phases, where Phase 1 and Phase 2 involve the expansion of the sewer collection system along the east side of West Main Street. Phases 1 and 2 are estimated to have a total future average daily wastewater flow of 8,420 gpd.

Phase 3 of the expansion project includes extending the sewer collection system to an existing mobile home park and commercial fuel company. Phase 3 of the future expansion area is considered separately from Phases 1 & 2 as implementation of Phase 3 is unknown at this time. Table 3-2 below depicts the future average daily wastewater flows associated with each phase of the sewer expansion area.

Table 3-2 Future Sewer Expansion Area Wastewater Flows

Expansion Phase	Future Average Daily Wastewater Flow[1] (gpd)
Phase 1	2,945
Phase 2	5,475
Phases 1 & 2 Total	**8,420**
Phase 3	36,420
Phases 1, 2, & 3 Total	**44,840**

Notes:
1. July 2021 West Main Sewer Extension Preliminary Engineering Report

In addition to the proposed expansion areas, the current unconnected committed allocations are 3,530 gallons per day between the residences and commercial users listed in Table 3-3.

Table 3-3 Unconnected Committed Sewer Allocated Flows

Area	Unconnected Committed Sewer Allocated Flows[1] (gpd)
Peaceable Kingdom (residential)	1,680
Whistle Stop Lane (residential)	680
112 E. Main Street (residential)	210
DS0022 (residential & commercial)	960
Total	**3,530**

Notes:
1. July 2021 West Main Sewer Extension Preliminary Engineering Report

Combining the unconnected committed sewer allocated flows with the proposed expansion area Phase 1-3 flows, the total flow of proposed and committed unconnected sewer allocated flows is 48,370 gpd.

Assuming that the current unconnected committed sewer allocated flows and the total proposed expansion area are implemented during the next 20-years, Table 3-4 presents the future wastewater treatment capacity at the Richmond WWTF.

Table 3-4 Future Wastewater Treatment Flow Capacity

Description	Wastewater Flow (gpd)
WWTF Permitted Design Capacity	222,000
80% of WWTF Permitted Design Capacity	177,600
Historical Average Daily Flow[1]	73,000
Unconnected Committed Sewer Flows	3,530
Proposed Future Expansion Area Flows (Phases 1-3)	44,840
Total Remaining WWTF Treatment Capacity[2]	**56,230**

Notes:
1. Historical Average Daily Flow from January 2018 – February 2023
2. Assuming proposed future expansion area flows are implemented

As discussed in Section 1.5, average daily flow entering the Richmond WWTF is not anticipated to exceed the design flow of 0.222 MGD over the 20-year planning horizon.

3.4 Design Criteria

3.4.1. Influent

The original influent design criteria, current influent conditions, and proposed influent design criteria for the liquid treatment processes at the Richmond WWTF are presented in Table 3-5 on the following page. Historical operating data is discussed in Section 2.5.

In order to determine the peak hour flow and current peaking factor, additional data collection is required due to the historical inaccuracy of the effluent flow meter. This peaking factor will be used to determine the future peak hour flow throughout the wastewater treatment facility. Since this data collection issue was identified, the effluent flow meter has been corrected and recalibrated. Additional data collection beginning in October 2023 will help inform decisions surrounding pending design criteria.

The proposed design criteria are pending future conversations with the Town of Richmond regarding the intent of future septage receiving volumes and practices. There is no flow meter on the dewatering pressate line, making it difficult to determine the nutrient load to the liquid treatment stream associated with pressate and originating from septage receiving.

Table 3-5 Richmond WWTF Proposed Influent Design Criteria

Parameter	Original Design [1,2]	Current Conditions[3,4]	Proposed Design Criteria
Average Daily Flow	0.222 MGD	0.073	0.222 MGD
Peak Daily Flow	-	*pending*	*pending*
Peak Hour Flow	1.152 MGD	*pending*	*pending*
Biochemical Oxygen Demand	324 mg/L 600 lbs/day	670 mg/L 411 lbs/day	*pending*
Total Suspended Solids	270 mg/L 500 lbs/day	932 mg/L 573 lbs/day	*pending*
Total Phosphorus	10 mg/L	19.5 mg/L	*pending*
Total Nitrogen	-	-	-
Temperature (min/avg/max)	10/_/20°C	4/15/26°C	*pending*

Notes:

1. Source: Basis of Design, 2003
2. Original design criteria BOD and TSS concentration are back calculated using design loads and design ADF. Original design criteria TP load is back calculated using design concentration and design ADF.
3. Based on Monthly Operating Report data from January 2018 to February 2023. Note that influent data is from wet well samples, which include municipal influent, RAS, and pressate.
4. Historical BOD and TSS loads are back calculated using historical average flows and concentrations.

3.4.2. Effluent

Effluent design criteria for the Richmond WWTF are based on the existing NPDES permit and are provided in Table 3-3 on the following page. The existing NPDES permit expires on December 31, 2025.

Table 3-3 Richmond WWTF Upgrade Effluent Design Criteria

Parameter	Original Design Criteria[1]	Proposed Design Criteria[2]
Flow (Annual Average)	0.222 MGD	0.222 MGD
BOD (Monthly Average)	30 mg/L	30 mg/L
TSS (Monthly Average)	30 mg/L	30 mg/L
Total Phosphorus (Monthly Average)	0.8 mg/L	0.8 mg/L
Total Phosphorus (Annual Load)	134 lbs/year	134 lbs/year
Total Nitrogen (Annual Average)	Monitor Only	Monitor Only
Total Kjeldahl Nitrogen (TKN) (Daily Maximum)	Monitor Only	Monitor Only
Nitrate/Nitrite Nitrogen (NOx) (Daily Maximum)	Monitor Only	Monitor Only
Settleable Solids (Instantaneous Maximum)	1.0 mL/L	1.0 mL/L
E. coli (Instantaneous Maximum)	77 CFU/100 ml	77 CFU/100 mL
pH	6.5-8.5 S.U.	6.5-8.5 S.U.

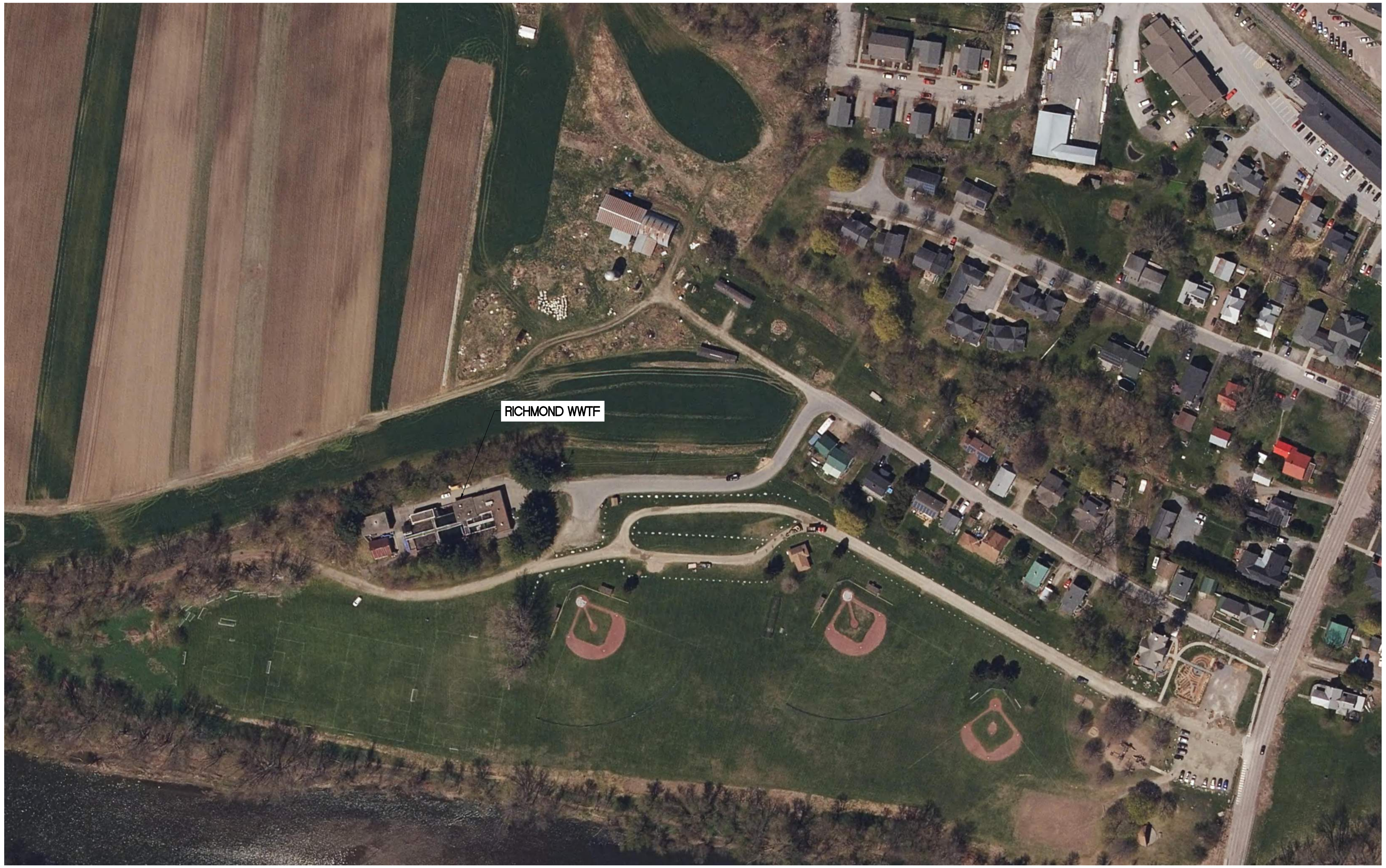
Notes:

1. Source: Richmond WWTF current NPDES Discharge Permit No. 3-1173, effective date January 1, 2021.
2. Proposed Effluent Design Criteria is from the WWTF's NPDES Discharge Permit No. 3-1173, effective date January 1, 2021.

APPENDICES

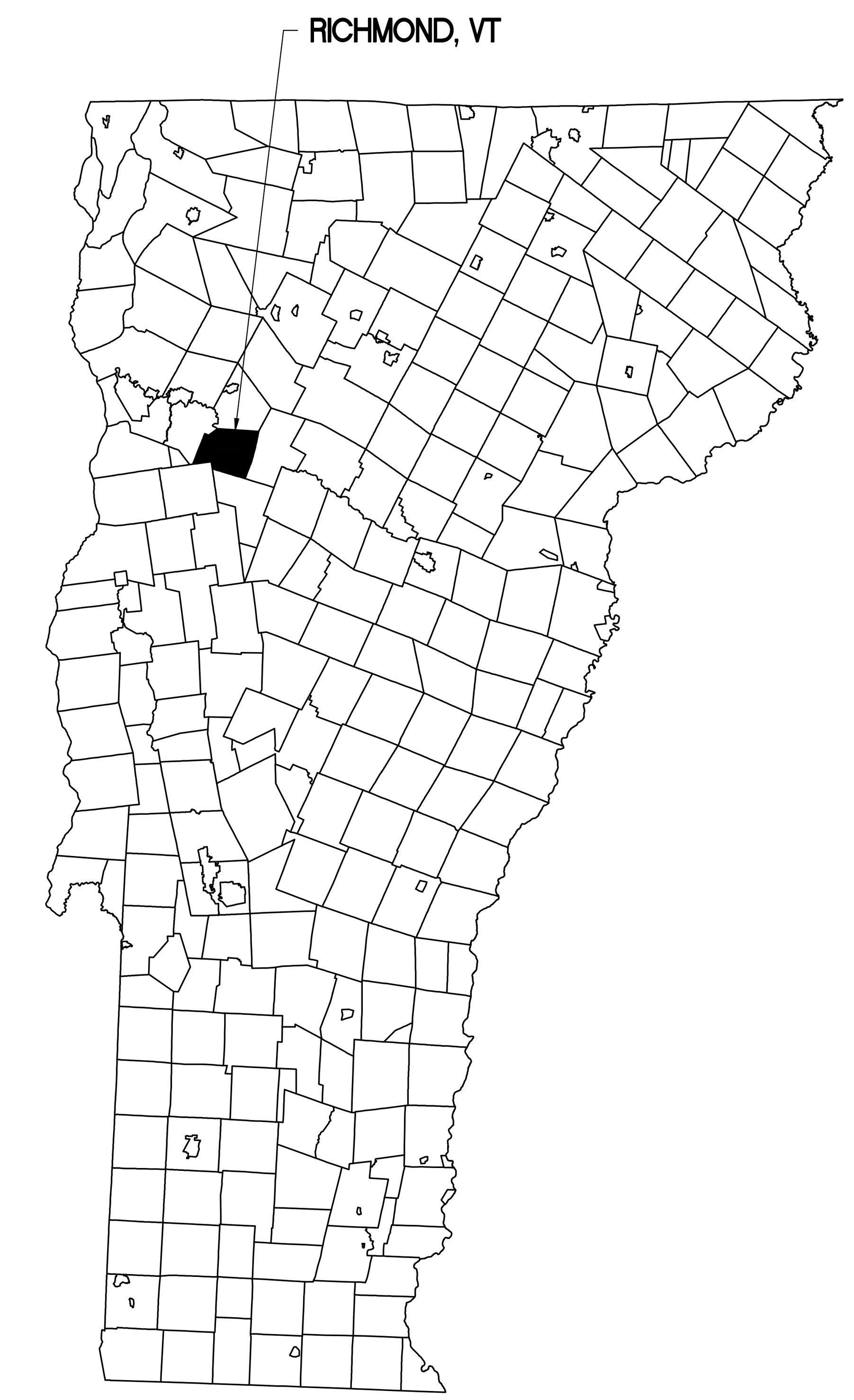
APPENDIX A

FIGURES



PLAN
SCALE: 1"=100'

GRAPHIC SCALE
0 100 150 200
(IN FEET)



ENGINEER

DRAFT

ISSUE	DESCRIPTION	DATE

HOYLE TANNER PROJECT NO.
23.102601.00

CHECKED BY: JA
DRAWN BY: JEN
DESIGNED BY: AD

HOYLE TANNER
150 Dow Street
Manchester, NH 03101
(603) 669-5555
www.hoyletanner.com

DATE: MAY, 2023
SCALE: AS NOTED

RICHMOND WWTF 20-YEAR EVALUATION
RICHMOND, VERMONT

LOCATION MAP

FIGURE NO.
A-1

SHEET 1 OF 3

22_102601_00 – FIGS.DWG

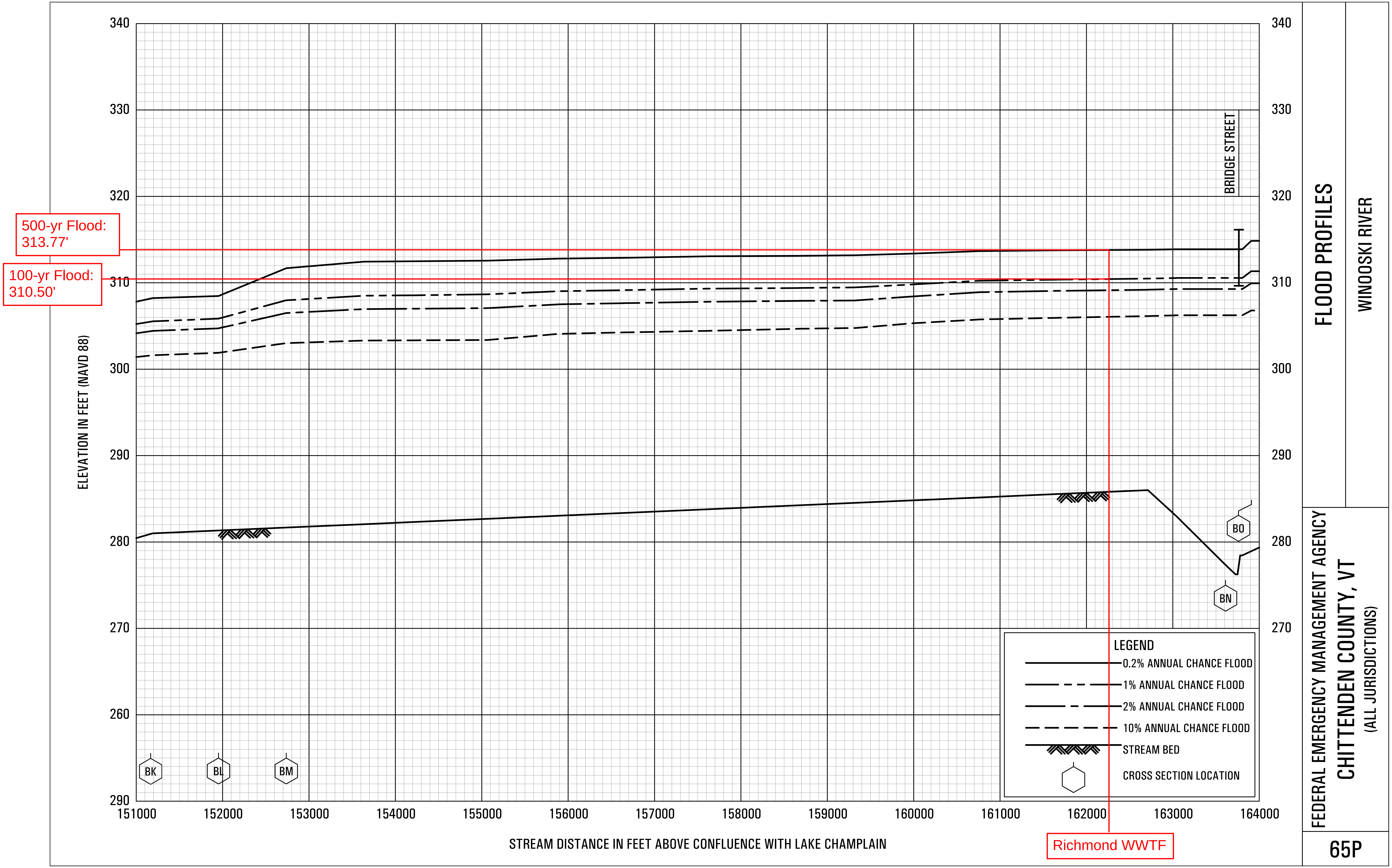

500-yr Flood: 313.77'
100-yr Flood: 310.50'
Richmond WWTF
BRIDGE STREET
ELEVATION IN FEET (NAVD 88)
STREAM DISTANCE IN FEET ABOVE CONFLUENCE WITH LAKE CHAMPLAIN
340
330
320
310
300
290
280
270
260
290
151000
152000
153000
154000
155000
156000
157000
158000
159000
160000
161000
162000
163000
164000
BK
BL
BM
BN
BO
LEGEND
0.2% ANNUAL CHANCE FLOOD
1% ANNUAL CHANCE FLOOD
2% ANNUAL CHANCE FLOOD
10% ANNUAL CHANCE FLOOD
STREAM BED
CROSS SECTION LOCATION
FLOOD PROFILES
WINOOSKI RIVER
FEDERAL EMERGENCY MANAGEMENT AGENCY
CHITTENDEN COUNTY, VT
(ALL JURISDICTIONS)
65P

FIGURE A-2

National Flood Hazard Layer FIRMette

FEMA

73°0'22"W 44°24'25"N

72°59'44"W 44°24'N

Feet 1:6,000

0 250 500 1,000 1,500 2,000

Basemap: USGS National Map: Orthoimagery: Data refreshed October, 2020

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS
- Without Base Flood Elevation (BFE) *Zone A, V, A99*
- With BFE or Depth *Zone AE, AO, AH, VE, AR*
- Regulatory Floodway

OTHER AREAS OF FLOOD HAZARD
- 0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
- Future Conditions 1% Annual Chance Flood Hazard *Zone X*
- Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
- Area with Flood Risk due to Levee *Zone D*

OTHER AREAS
- NO SCREEN Area of Minimal Flood Hazard *Zone X*
- Effective LOMRs
- Area of Undetermined Flood Hazard *Zone D*

GENERAL STRUCTURES
- Channel, Culvert, or Storm Sewer
- Levee, Dike, or Floodwall

OTHER FEATURES
- B 20.2 / 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation
- 8 Coastal Transect
- 513 Base Flood Elevation Line (BFE)
- Limit of Study
- Jurisdiction Boundary
- Coastal Transect Baseline
- Profile Baseline
- Hydrographic Feature

MAP PANELS
- Digital Data Available
- No Digital Data Available
- Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

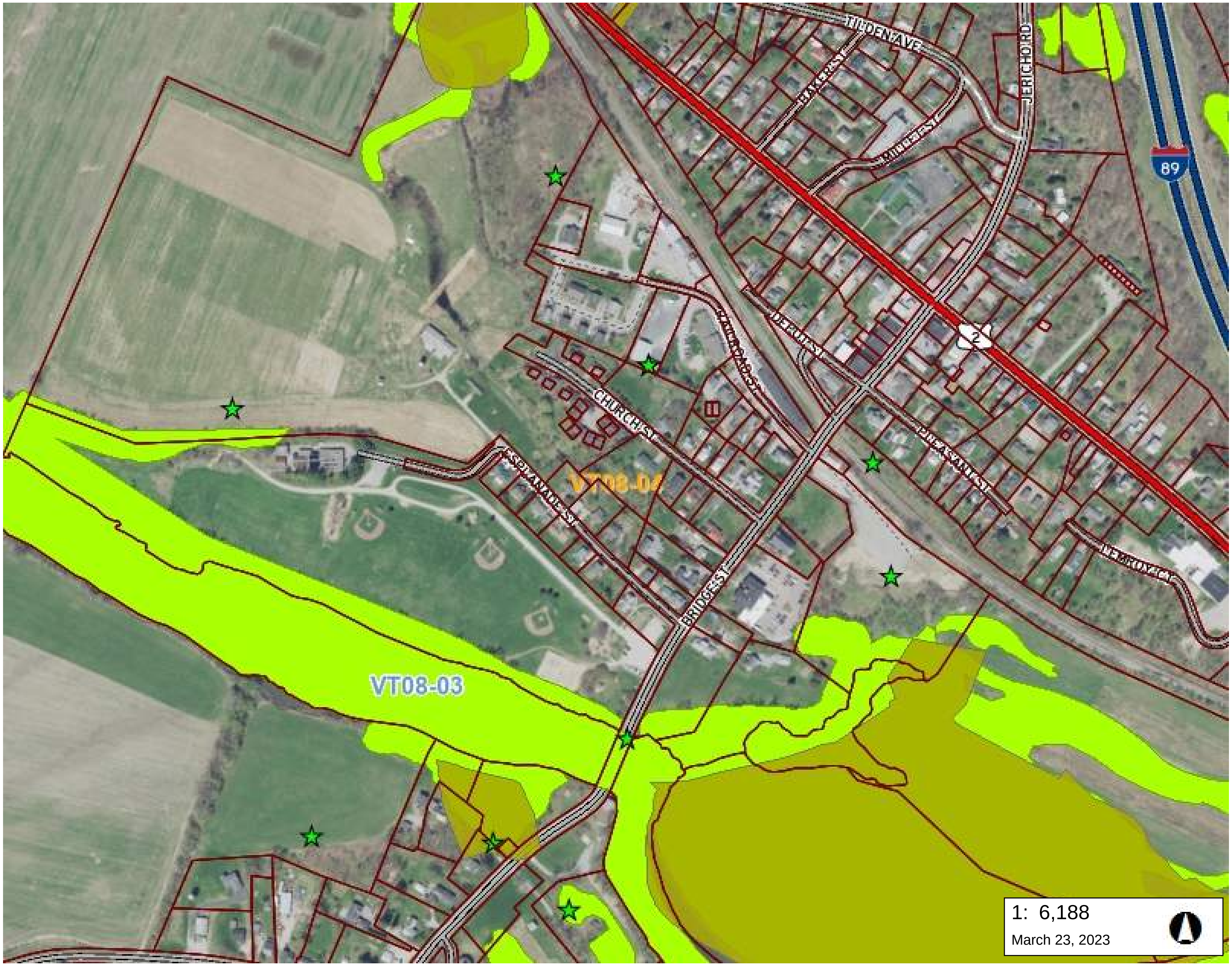
The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 3/23/2023 at 1:22 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

Natural Resources Atlas

Vermont Agency of Natural Resources

vermont.gov



1: 6,188

March 23, 2023

LEGEND

- Wetland Projects
- Watersheds for 303(d) List
- Priority Waters List (Lakes and
  - Part B (impaired TMDL not requirec
  - Part D (impaired with approved TM
  - Part E (altered exotic species)
  - Part F (altered flow regulation)
- Stressed Waters List (Streams
- Stressed Waters List (Lakes ar
- Designated ORW (Streams an
- Prospective ORW (Streams ar
- Prospective ORW (Lakes and
- Wetland - VSWI
  - Class 1 Wetland
  - Class 2 Wetland
  - Wetland Buffer
- Wetlands Advisory Layer
- River Main Stem Waterbodies
- WBID Watersheds
- Stormwater Impaired Watershe
- Parcels (standardized)
- Roads
  - Interstate
  - US Highway; 1
  - State Highway
  - Town Highway (Class 1)
  - Town Highway (Class 2,3)

314.0 0 157.00 314.0 Meters

WGS_1984_Web_Mercator_Auxiliary_Sphere 1" = 516 Ft. 1cm = 62 Meters

THIS MAP IS NOT TO BE USED FOR NAVIGATION

DISCLAIMER: This map is for general reference only. Data layers that appear on this map may or may not be accurate, current, or otherwise reliable. ANR and the State of Vermont make no representations of any kind, including but not limited to, the warranties of merchantability, or fitness for a particular use, nor are any such warranties to be implied with respect to the data on this map.

NOTES

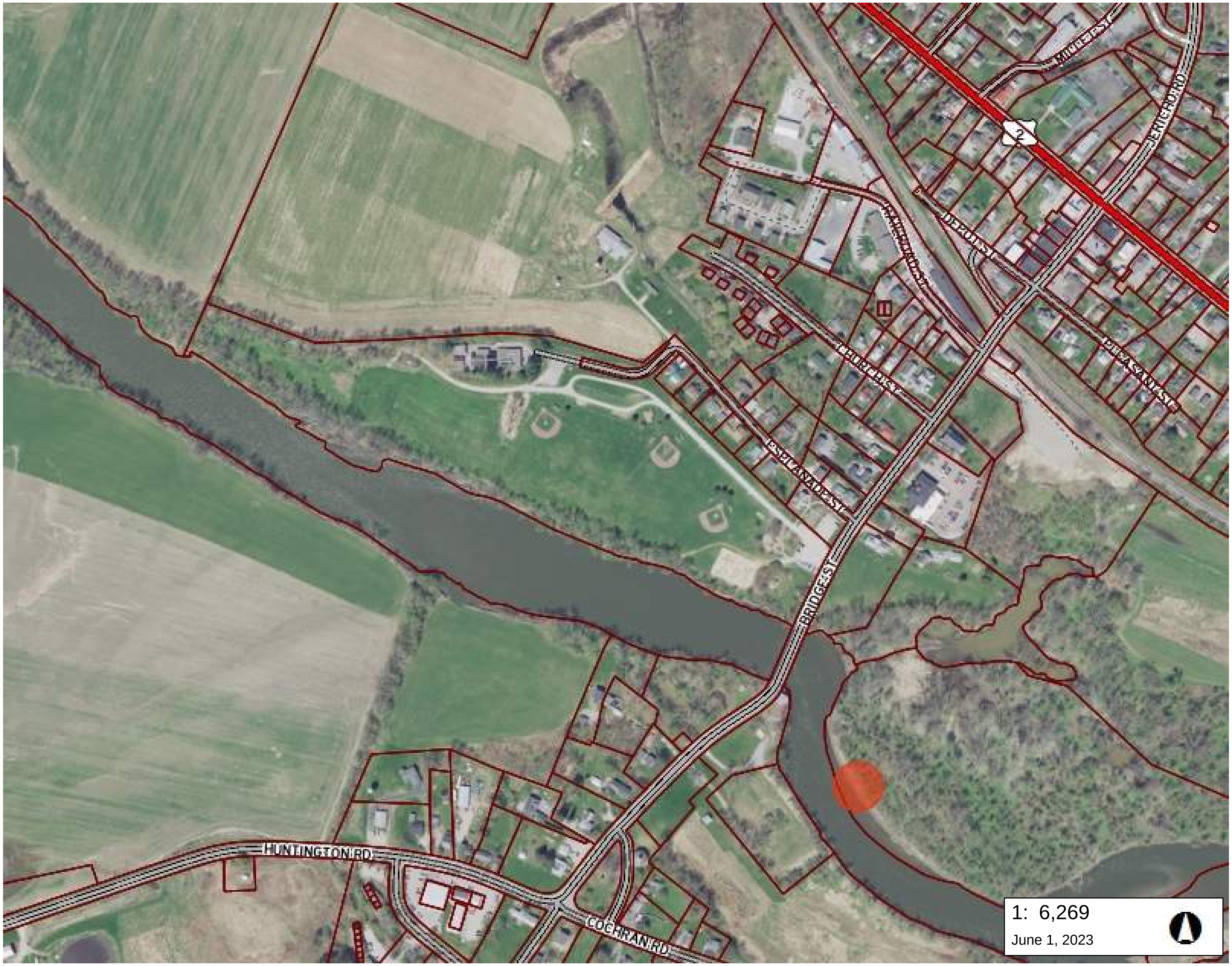
Watershed & Wetlands

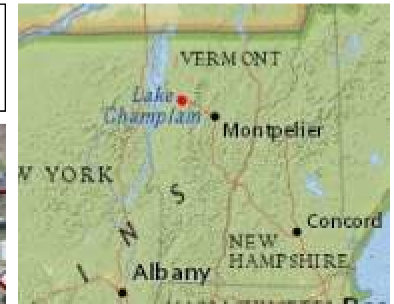
FIGURE A-4

Richmond WWTF - Endangered Species

Vermont Agency of Natural Resources

vermont.gov





LEGEND

Rare Threatened and Endange

- RTE Animal
- RTE Plant

Indiana Bat Hibernacula

Indiana Bat Summer Range

- Observed
- Potential

Parcels (standardized)

Roads

- Interstate
- US Highway; 1
- State Highway
- Town Highway (Class 1)
- Town Highway (Class 2,3)
- Town Highway (Class 4)
- State Forest Trail
- National Forest Trail
- Legal Trail
- Private Road/Driveway
- Proposed Roads

Town Boundary

NOTES

Map created using ANR's Natural Resources Atlas

318.0 0 159.00 318.0 Meters

WGS_1984_Web_Mercator_Auxiliary_Sphere 1" = 522 Ft. 1cm = 63 Meters

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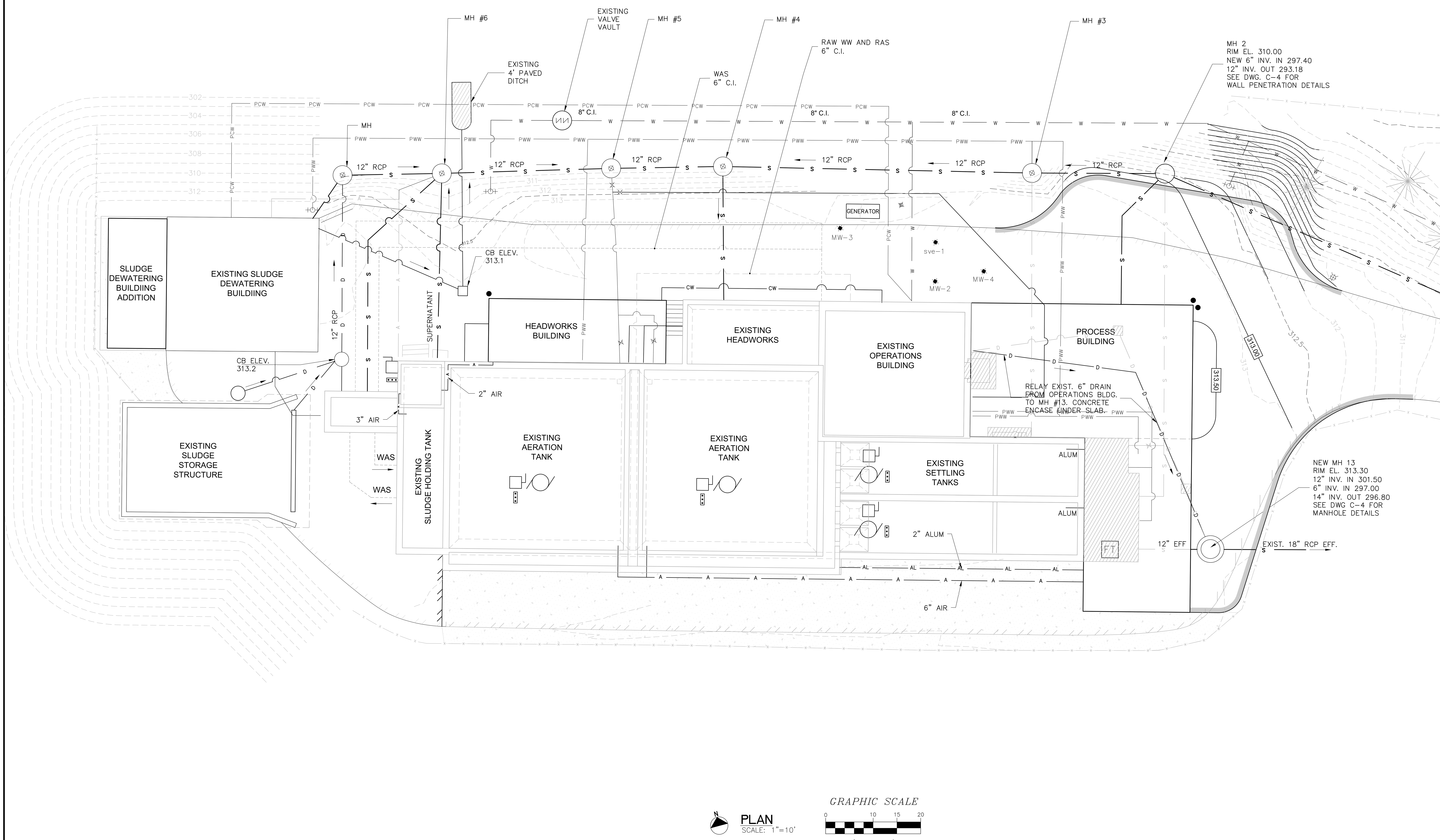
FIGURE A-5



ENGINEER

DRAFT

ISSUE	DESCRIPTION	DATE

HOYLE TANNER PROJECT NO. 23.102601.00

CHECKED BY: JA
DRAWN BY: JEN
DESIGNED BY: AD
DATE: MAY, 2023
SCALE: AS NOTED

HOYLE TANNER
150 Dow Street
Manchester, NH 03101
(603) 669-5555
www.hoyletanner.com

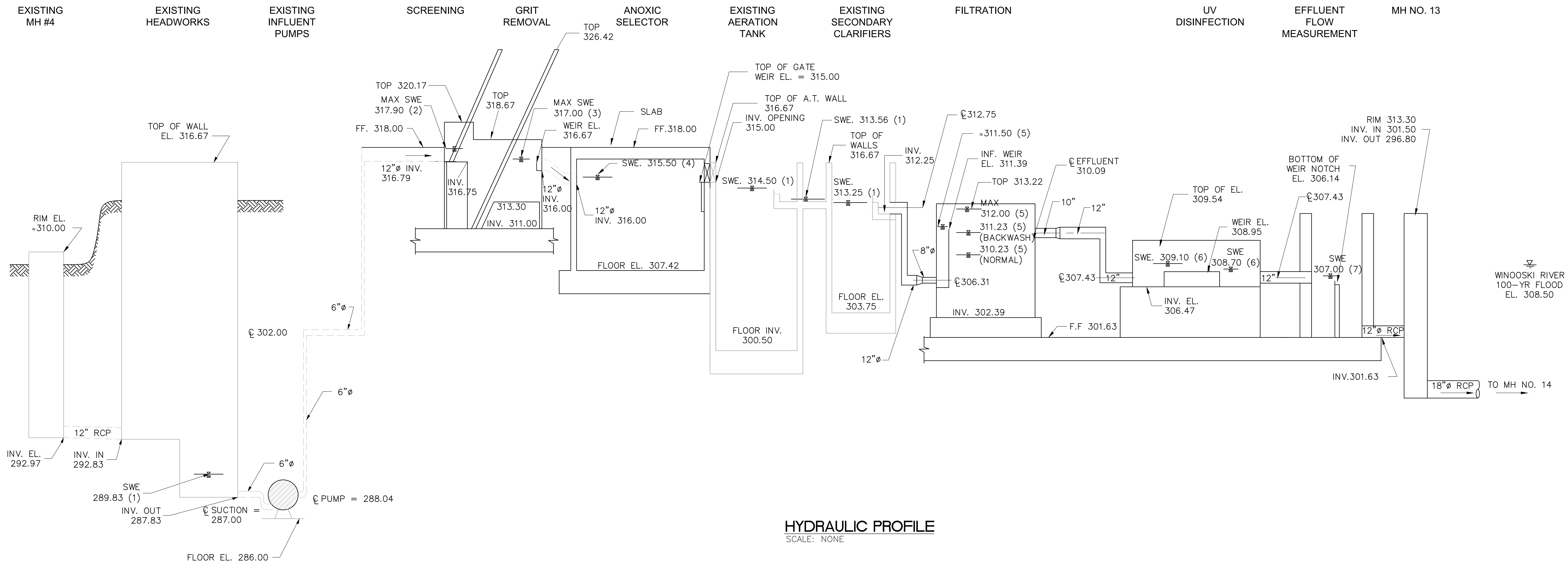
RICHMOND WWTF 20-YEAR EVALUATION
RICHMOND, VERMONT

EXISTING SITE PLAN

FIGURE NO.
A-6

SHEET 3 OF 3

22_102601_00 – FIGS.DWG



HYDRAULIC PROFILE
SCALE: NONE

NOTES:

1. SURFACE WATER ELEVATION (SWE) BASED ON MAXIMUM FLOW OF 1.14 MGD THROUGH ONLY ONE OF EACH DUPLICATE UNIT AS PER RECORD DRAWINGS BY WEBSTER-MARTIN, INC., JULY 1971.
2. HEADLOSS ACROSS SCREEN UNIT BASED ON CALCULATIONS BY LAKESIDE EQUIPMENT CORPORATION AT 2.97 MGD.
3. HEADLOSS AT EFFLUENT WEIR OF GRIT REMOVAL PACKAGE UNIT BASED ON CALCULATIONS BY LAKESIDE EQUIPMENT CORPORATION AT DESIGN PEAK HOURLY FLOW OF 1.152 MGD.
4. DESIGN PEAK HOURLY FLOW OF 1.152 MGD
5. HEADLOSS THROUGH FILTER UNIT BASED ON CALCULATIONS BY AQUA-AEROBICS SYSTEMS, INC. AT 3254 GPM THROUGH ONE FILTER UNIT.
6. HEADLOSS THROUGH UB UNIT BASED ON CALCULATIONS BY TROJAN TECHNOLOGIES, INC. AT PEAK HOURLY FLOW OF 1.0 MGD.
7. PEAK HOURLY FLOW OF 1.0 MGD.

22_102601_00 - FIGS.DWG

ENGINEER

DRAFT

ISSUE	DESCRIPTION	DATE

HOYLE TANNER PROJECT NO. 23.102601.00

CHECKED BY: JA
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HOYLE TANNER
150 Dow Street
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(603) 669-5555
www.hoyletanner.com

DATE: MAY, 2023
SCALE: AS NOTED

RICHMOND WWTF 20-YEAR EVALUATION
RICHMOND, VERMONT

EXISTING HYDRAULIC PROFILE

FIGURE NO. A-7

SHEET 2 OF 3

APPENDIX B

FOURNIER ROTARY PRESS
INSPECTION REPORT
9/30/2023

September 30th, 2023

Richmond Vermont

Visit Summary

Introduction

The purpose of this visit was to inspect the condition of the two channel Rotary Press located in Richmond Vermont and provide operational assistance to the personnel. The press has an estimated 45,000hrs on it. This is a two 36" channel press, model number 2-900/2000, and serial number PR-09-0/99.

Inspection

Screens: The facility only runs sludge through one channel and has recently complained about the filtrate quality. They do not run on the other channel due to damaged screens. Opening the restrictor arms on both channels and cleaning the channels out to inspect, I discovered that all four screens were very worn.

Frames: The frames on the channels were rusting and worn were the restrictor arms moved. You could see the yellow cover seal between the wheel and the frame starting to come through with trash.

September 30th, 2023

Shaft: The shaft of the press was in ok condition. The shaft only supports one channel on each side, so there was limited exposure to the atmosphere. There is sone rust around on the end, however that should not effect the removal.

Three-way valve: The existing three-way valve does not work. It has been stuck in the dewatering position and does not turn. This causes problems with a proper startup. Sludge also leaks out of the supply tank, through the sludge pump and, because the valve is stuck in the dewatering position, into the channels.

Floccualtor: The flocculator seal shaft has been leaking for some time. It's hard to tell the extent of the damage.

Panel: The panel is the old style with speed dials for the flocculator speed and rotary press speed. The outlet pressure is controlled directly at the channel. If you adjust it on the panel, it doesn't do anything.

Channel Wash Manifold: The Festo valves on the channel wash manifold do not seem to work. The piping and spray bars are not in good condition.

Sludge Pump: The facility has a double disc sludge pump and is in good condition. However, the sludge tank gravity feeds past the pump when it is in the off position.

Polymer System: The polymer system does not have an indication of concentration. A batch of diluted polymer is sent to a 50-gallon tank. From there it is pumped to the floccuator at a ratio of the sludge flow.

Conveyor System: The conveyor seems to be in good condition apart from a couple rusted out holes in the lower one. Also, on the inclined conveyor it is open at the top. The employees said this was from cake building up due to an unopened slide gate that has since been fixed.

Operation

The operators at the facility are operating the press as best as possible for the condition it is in. During a normal startup the three-way valve sends sludge down the drain, in recirculation, this gives the operator time to determine that they have a good flocculation. Because the valve does not work it sends it directly to the rotary press. If the sludge tank is full and the sludge supply valve is open to the pump, it passes the pump and goes into the rotary press without it even on or running. So, when they start instead of recirculating, they hit dewatering and it goes right to the rotary press. The first sludge the press receives is not flocculated. Once it starts to flocculate, the one operating channel begins to produce cake. However, a lot of solids are going through the worn screens and down the filtrate. Because they are running a blend of digested and septic sludge the press is still able to produce some cake. I believe if they were just running digested sludge, they would have a hard time getting anything to come out of the channel. The only way to keep sludge producing cake is to treat it gently. If they produce high pressure inside the channel sludge comes out the screens instead of out the front as cake. The key is low outlet pressure 5psi, low inlet pressure 1.2psi, and moderate rotary press speed 30%. This minimizes the sludge in the filtrate being sent to the head of the plant. The press still produces 25-30% cake dryness with a flow of 20-30gpm through the channel. Sludge total solids was 1.45% Polymer consumption was 26 active lbs/dry ton.

Observations and Conclusion

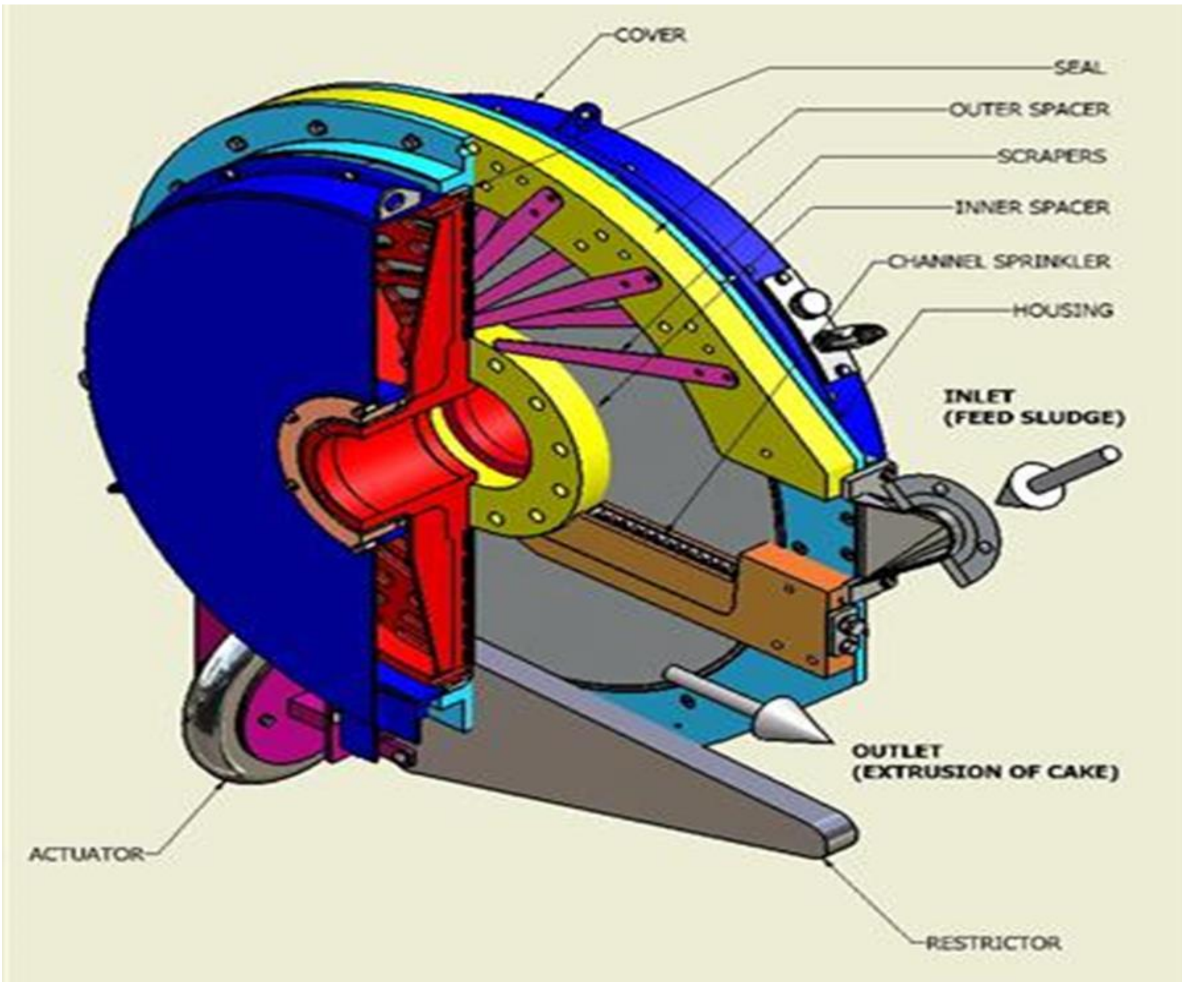
The Richmond, Vermont facility needs:

1. New Channels – Everything on the channels needs replacement so I would recommend whole new channels. Even if you could spare some parts, we might need to cut the wheels from the shaft.
2. New Flocculator Assembly – The top of the floccuator need to be dismantled and probably everything needs to be replaced below the gearbox.
3. Three-way Valve – I would suggest they replace them with two two-way valves.
4. Channel Wash Valves – If it doesn't already come with the new channels.
5. Air supply – A new air regulator needs to be installed. As of right now I do not believe they have air going to the three-way valve and they only have it to the bellows.

Possible upgrades needed:

1. Sludge pump - The sludge supply tank feeding the pump is gravity feeding through it. The pump may just need new interior parts. Or the sludge valve needs to be closed off when the press is not running.
2. Polymer system – We determined the polymer was making down a batch of 0.25% concentration, with a polymer to sludge ratio of 8% and a consumption of 26 active lbs/dry ton. An upgrade of the polymer system could produce lower consumption.

3. Panel HMI – The installation of a new panel would make it easier for the operators to use the press. The current old program is outdated and not optimal.
4. Conveyor system – There are a couple holes that need to be patched or sections replaced.



APPENDIX C

FY24 WATER & SEWER BUDGET

FINAL Water FY24

Account #	Description	Budgeted FY22	Actual FY22	Budgeted FY23	Budgeted FY24	+INCREASE (DECREASE)
WATER REVENUE						
20-6-00-3-00.00	Water User Receipts	326,560	333,594	317,547	320,384	0.89%
20-6-00-3-01.00	Sale of Water from Hydrant	1,500	2,046	1,500	1,500	0.00%
20-6-03-5-40.05	Net Interest on Checking Account	500	856	500	6,000	1100.00%
20-6-00-4-10.02	Hook On Fees – Water	500	250	500	500	0.00%
20-0-00-0-00.00	Fund Balance Usage	-	-	27,339	-	-100.00%
20-6-10-4-10.04	Fire Service Fees	50,432	50,432	51,148	49,899	-2.44%
	Water Revenue Sub Totals	379,492	387,178	398,534	378,283	-5.08%
WATER RESOURCES ADMINISTRATION EXPENSES (30% of total)						
20-7-80-0-10.00	Salaries	63,593	65,885	77,277	83,002	7.41%
20-7-80-0-10.30	Insurance Opt Out	1,500	1,500	1,500	-	-100.00%
20-7-80-0-10.99	Overtime	900	3,135	2,400	2,400	0.00%
20-7-80-0-11.00	Social Security/Medicare	5,081	5,168	6,181	6,576	6.39%
20-7-80-0-12.00	Municipal Retirement	4,031	5,624	5,318	5,765	8.41%
20-7-80-0-15.00	Health Insurance	7,372	8,299	15,023	19,310	28.54%
20-7-80-0-15.01	Health Savings Account	1,248	420	458	313	-31.66%
20-7-80-0-15.03	Long Term Disablity	420	408	407	570	40.05%
20-7-80-1-16.00	Uniforms	400	164	400	400	0.00%
20-7-80-1-20.00	Office Supplies/Postage	300	296	300	670	123.33%
20-7-80-1-22.00	Office Equipment	200	155	200	200	0.00%
20-7-80-1-22.01	Computer	-	69	-	450	100.00%
20-7-80-1-22.02	Computer Support	-	646	1,777	1,800	1.29%
20-7-80-1-24.00	Advertising	200	-	200	200	0.00%
20-7-80-1-26.01	Administrative Expense	9,000	9,000	9,000	12,450	38.33%
20-7-80-1-26.03	Audit Expenses	6,673	3,030	1,305	1,450	11.11%
20-7-80-1-27.00	Staff Training/Education/Licenses	800	956	800	1,300	62.50%
20-7-80-1-27.01	Safety Training	100	-	100	100	0.00%
20-7-80-1-29.00	Travel	300	-	300	300	0.00%
20-7-80-1-30.00	Telephone	2,500	1,527	2,500	2,500	0.00%
20-7-80-1-42.00	Association Dues	200	140	200	200	0.00%
20-7-80-1-43.00	Legal	-	664	-	500	100.00%
20-7-80-1-48.00	W & S General Insurance	8,498	7,023	5,782	6,300	8.96%
	Water Administration Expense Totals	113,316	114,109	131,428	146,756	11.66%
WATER OPERATIONS EXPENSES						
20-7-83-4-16.00	Personal Protective Equip	500	244	500	500	0.00%
20-7-83-4-31.00	Heat	600	626	600	600	0.00%
20-7-83-4-32.00	Electricity	8,500	10,799	8,500	10,100	18.82%
20-7-83-4-34.00	Trash Removal	800	2,117	800	1,500	87.50%
20-7-83-4-41.00	System Permits/Fees/Licenses	1,900	1,293	1,900	1,900	0.00%
20-7-83-4-45.00	Water Contracted	5,000	3,041	5,000	4,000	-20.00%
20-7-83-4-45.02	Equipment Rental	500	220	500	500	0.00%
20-7-83-4-46.00	Engineering	2,000	145	2,000	1,000	-50.00%
20-7-83-4-50.00	Gas, Oil & Diesel Fuel	500	227	500	500	0.00%
20-7-83-4-52.00	Fleet Maintenance	1,000	46	1,000	1,000	0.00%
20-7-83-4-62.02	Water Line	20,000	2,261	20,000	15,000	-25.00%
20-7-83-4-62.03	Pumps/Tanks	5,000	5,251	5,000	5,000	0.00%
20-7-83-4-62.04	Asphalt Repair	5,000	-	5,000	5,000	0.00%
20-7-83-4-62.05	Equipment Purchase	500	33	500	500	0.00%
20-7-83-4-62.06	Supplies	1,000	96	1,000	1,000	0.00%
20-7-83-4-62.07	Meters	3,000	653	3,000	3,000	0.00%
20-7-83-4-65.00	Water Treatment Chemicals	1,000	779	1,000	2,600	160.00%
	Water Operating Expense Totals	56,800	27,831	56,800	53,700	-5.46%

WATER CAPITAL EXPENSES							FY23 Predicted Year End Balances
20-7-90-5-93.01	Water Capital Reserve	36,000	36,000	36,000	-	-100.00%	115,991
20-7-90-5-90.03	Short-term (10 yr) capital fund	20,000	20,000	20,000	20,000	0.00%	126,651
20-7-90-2-90.09	Distribution System Capital fund	15,000	15,000	15,000	20,000	33.33%	21,100
20-7-90-2-90.16	Water Reservoir gap principal (2025)	25,857	25,857	25,857	25,857	0.00%	263,742
20-7-90-2-90.17	Water Reservoir gap interest	1,975	1,482	1,482	990	-33.20%	
20-7-90-5-90.01	RF3-302 Water Reservoir principal (2048)	37,705	37,705	37,705	37,705	0.00%	
20-7-90-5-93.02	RF3-335 East Main principal	25,140	25,140	25,140	25,140	0.00%	
20-7-90-2-90.07	Jericho Road Loan Principal (2032)	26,208	26,208	26,208	26,208	0.00%	
20-7-90-2-90.08	Jericho Road Loan Interest	11,491	11,491	10,549	9,562	-9.36%	
20-7-90-5-90.13	RF3-365 Bridge Upper & Crossing Principal (2047)	10,000	9,865	9,865	9,865	0.00%	
	RF3-444 Bridge Street Middle (2062)	-	-	2,500	2,500	0.00%	
20-7-90-1-00.00	Unbudgeted Capital Expense			-	-	0.00%	
	Water Capital Expense Totals	209,376	208,748	210,306	177,827	-15.44%	
	TOTAL WATER REVENUE	379,492	387,178	398,534	378,283	-5.08%	
	TOTAL WATER EXPENSES	379,492	350,688	398,534	378,283	-5.08%	
	BALANCE	-	36,490	-	-		

UNASSIGNED FUNDS FY22 YEAR END AUDIT	(64,135)
UNASSIGNED FUNDS FY23 USAGE/GROWTH	255,145
PREDICTED UNASSIGNED FUNDS YEAR END FY23	191,010
DRAFT FY24 WATER EXPENSES AS OF 05/01/2023	378,283
15% OF FY24 BUDGET EXPENSES	56,742
UNASSIGNED FUNDS IN EXCESS OF 15%	134,268
Available Unassigned funds & Total FY23 Reserve Funds	398,010

FINAL Wastewater FY24

Account #	Description	Budgeted FY22	Actual FY22	Budgeted FY23	Budgeted FY24	+INCREASE (DECREASE)
WASTEWATER REVENUE						
21-6-00-3-00.01	Sewer User Receipts	357,337	373,213	361,326	292,874	-19%
21-6-00-4-10.03	Hook On Fees – Sewer	1,000	3,213	1,000	1,000	0%
21-6-03-5-40.05	Net Interest on Checking Account	1,200	1,998	1,500	14,000	833%
21-0-00-0-00.00	Fund Balance Usage	-	-	48,394	-	-100%
21-6-01-4-11.10	Septage Receipts	430,000	483,577	460,000	550,000	20%
	Waste Water Revenue Subtotal	789,537	862,001	872,220	857,874	-2%
WASTEWATER RESOURCES ADMINISTRATION EXPENSES (70% of total)						
21-7-80-0-10.00	Salaries	148,381	153,731	180,312	199,270	11%
21-7-80-0-10.30	Insurance Opt Out	3,500	3,500	3,500	-	-100%
21-7-80-0-10.99	Overtime	2,100	7,314	5,600	5,600	0%
21-7-80-0-11.00	Social Security/Medicare	11,857	12,058	14,423	15,344	6%
21-7-80-0-12.00	Municipal Retirement	9,405	13,146	12,408	13,451	8%
21-7-80-0-15.00	Health Insurance	17,201	19,363	35,053	45,056	29%
21-7-80-0-15.01	Health Savings Account	2,913	980	1,070	731	-32%
21-7-80-0-15.03	Long Term Disablity	960	953	949	1,330	40%
21-7-80-1-16.00	Uniforms	900	355	900	900	0%
21-7-80-1-20.00	Office Supplies/Postage	500	691	500	500	0%
21-7-80-1-22.00	Office Equipment	400	281	400	400	0%
21-7-80-1-22.01	Computer	-	2,299	-	1,050	100%
21-7-80-1-22.02	Computer Support	-	308	4,145	4,200	1%
21-7-80-1-24.00	Advertising	400	-	400	400	0%
21-7-80-1-26.01	Administrative Expense	21,000	21,000	21,000	29,050	38%
21-7-80-1-26.03	Audit Expenses	6,237	7,071	3,045	3,383	11%
21-7-80-1-27.00	Employee Training/Education/Licenses	1,800	739	1,800	2,770	54%
21-7-80-1-27.01	Safety Training	300	-	300	300	0%
21-7-80-1-29.00	Travel	700	-	700	700	0%
21-7-80-1-30.00	Telephone	3,800	3,548	3,800	3,800	0%
21-7-80-1-42.00	Association Dues	400	326	400	400	0%
21-7-80-1-43.00	Legal	3,000	2,856	3,000	3,000	0%
21-7-80-1-48.00	W & S General Insurance	15,868	13,958	11,341	13,000	15%
	Wastewater Administration Expense Subtotal	251,622	264,477	305,046	344,635	13%

WASTEWATER OPERATIONS EXPENSES

21-7-82-2-32.01	Electricity	1,000	-	1,000	1,000	0%
21-7-82-2-62.03	Pump Station Maintenance	1,800	-	1,800	1,800	0%
21-7-82-3-16.00	Personal Protective Gear	500	982	500	500	0%
21-7-82-3-31.00	Heat	10,000	8,400	10,000	10,000	0%
21-7-82-3-32.00	Plant Electricity	40,000	38,747	40,000	45,000	13%
21-7-82-3-32.02	WWTF water bill	32,000	31,515	32,000	32,000	0%
21-7-82-3-34.00	Rubbish Removal	1,500	4,213	1,500	4,300	187%
21-7-82-3-41.00	System Permits/Certs/Licenses	800	1,128	800	800	0%
21-7-82-3-45.00	Wastewater Contracted	7,500	5,339	7,500	7,500	0%
21-7-82-3-45.01	Biosolids Contracted	4,500	5,994	4,500	4,500	0%
21-7-82-3-45.02	Equipment Rental	500	70	500	500	0%
21-7-82-3-45.03	Biosolids Disposal/CSWD	120,000	153,967	130,000	160,000	23%
21-7-82-3-46.00	Engineering	500	2,281	500	500	0%
21-7-82-3-50.00	Gas, Oil & Diesel Fuel	1,800	735	1,800	1,800	0%
21-7-82-3-52.00	Fleet Maintenance	2,500	188	2,500	2,500	0%
21-7-82-3-62.00	Wastewater Facil Repair	8,000	19,672	8,000	12,000	50%
21-7-82-3-62.01	Biosolids Facility Repair	8,000	14,003	8,000	9,000	13%
21-7-82-3-62.02	Collection System Repair	4,000	1,222	4,000	4,000	0%
21-7-82-3-65.00	Wastewater Chemicals	10,000	7,947	10,000	17,000	70%
21-7-82-3-65.01	Biosolids Chemicals	70,000	94,220	90,000	98,100	9%
21-7-82-3-66.00	Supplies	5,000	3,076	5,000	4,000	-20%
	Wastewater Operating Expense Subtotal	329,900	393,699	359,900	416,800	16%

WASTEWATER CAPITAL EXPENSES

							FY23 Predicted Year End Balances
21-7-90-5-93.00	Wastewater Capital Reserve	70,000	70,000	70,000	10,000	-86%	156,595
21-7-90-5-93.04	Short-term (10 yr) capital fund	50,000	50,000	50,000	10,000	-80%	158,305
21-7-90-5-93.11	Collection System Capital Fund	10,000	10,000	10,000	-	-100%	468,056
21-7-90-2-90.01	RFL-101 planning-ww (2027)	12,081	12,021	12,081	12,021	0%	782,956
21-7-90-2-90.06	Project 7a Sanitary Sewer (2032)	14,093	14,093	14,093	14,093	0%	
21-7-90-2-90.02	Phosphorus SRF(2026)	22,220	22,220	22,220	22,220	0%	
21-7-90-2-90.14	Jericho Rd Loan Principal (2032)	20,592	20,592	20,592	20,592	0%	
21-7-90-2-90.16	Jericho Rd Loan Interest	9,029	9,029	8,288	7,513	-9%	
21-7-82-1-00.00	Unbudgeted Capital Expense			-	-	0%	
	Wastewater Capital Subtotal	208,015	207,955	207,274	96,439	-53%	

TOTAL WASTEWATER REVENUE	789,537	862,001	872,220	857,874	-2%
TOTAL WASTEWATER EXPENSES	789,537	866,131	872,220	857,874	-2%
BALANCE	-	(4,130)	-	-	

UNASSIGNED FUNDS FY22 YEAR END AUDIT	382,021
UNASSIGNED FUNDS FY23 USAGE/GROWTH	159,314
PREDICTED UNASSIGNED FUND BALANCE YEAR END FY23	541,335
DRAFT FY24 SEWER EXPENSES AS OF 05/01/2023	857,874
15% OF FY 24 BUDGET EXPENSES	128,681
UNASSIGNED FUNDS IN EXCESS OF 15%	412,654

Available Unassigned funds & Total FY23 Reserve Funds	1,195,610

Water Budget - Fire Protection Calculation

48%	Tank loan	37,705	0.48	18098
48%	Gap loan	26,847	0.48	12887
5%	Total Water Budget	378,283	0.05	18914
				49899

Proprietary Net Position Unrestricted

FY23 Projections are ONLY for Reserve funds
(see previous sheets for unassigned fund projections)

	FY18	FY19	FY20	FY21	FY22	FY23 Usage Budgeted	FY23 Contribution	FY23 Usage as of 04/30/23	FY23 Predicted Year-End
Water Audit Unrestricted	298,778	243,190	(58,859)	349,163	156,230				
Water Reserves									
Short Term Capital	40,000	55,575	75,270	95,270	114,714	0	20,000	(18,722)	115,991
Water Capital	85,817	21,070	53,742	66,359	90,651	0	36,000	0	126,651
Distribution	63,265	52,702	57,430	(11,708)	15,000	0	15,000	(8,900)	21,100
Total Reserves	189,082	129,347	186,442	149,921	220,365	0	71,000	(27,622)	263,743
Water Audit Unrestricted minus Reserves	109,696	113,843	(245,301)	199,242	(64,135)				
Sewer Audit Unrestricted	471,819	587,860	781,772	974,341	1,048,217				
Sewer Reserves									
Wastewater Capital	197,761	331,572	355,769	388,056	92,755	0	70,000	(6,160)	156,595
Short Term Capital	30,682	53,522	85,496	117,479	115,385	0	50,000	(7,080)	158,305
Collection System	36,735	78,405	87,630	82,755	458,056	0	10,000	0	468,056
Total Reserves	265,178	463,499	528,895	588,290	666,196	0	130,000	(13,240)	782,956
Sewer Audit Unrestricted minus Reserves	206,641	124,361	252,877	386,051	382,021				

Water and Wastewater Items Identified for Repair or Replacement in FY23 and FY24

Water		**Outside Consultant**	**Estimated Cost**		**Fiscal Year**
	Wire and calibrate PH meter at water house.	Tom Allen	TBD		FY23 or FY24
	Water tank mixer replacement		$	20,000	FY24
	Water tank cleaning		$	9,000	FY24
	Repair common alarm	Tom Allen	TBD		FY23 or FY24
	Excavation to locate Borden St. water valve		$	10,000	FY23
Wastewater					
	Repair pager dialer	Tom Allen	TBD		FY23 or FY24
	Replace check valve for pump station	Phil Laramie	TBD		FY23
	Repair meters for hours of operation on each pump	Dan Pratt	TBD		FY23
	Rebuild backwash pump 1	Phil Laramie	$	5,000	FY23
	Rebuild backwash pump 2	Phil Laramie	$	5,000	FY24
	Purchase New backwash pump		TBD		FY23
	Grit motor assessment and repair	Dan Pratt	TBD		FY23
	Replace auger liner	Phil Laramie	TBD		FY23
	Replace air valve on dewatering press		TBD		FY23
	Repair hazardous gas alarm	Phil Laramie	TBD		FY23
	Purchase UV meter		$	2,128	FY23
	Install UV meter	Tom Allen	TBD		
	Clean and repair aeration tanks and all holding tanks	Obtaining Quotes	$30,000 - $50,000		FY23 or FY24
	Purchase meter for septage receiving	Obtaining Quotes	TBD		FY23
	Wastewater Mixer		$	8,500	FY23
	New Influent Pumps	Rough Estimate	$	60,000	FY24

Prioritize projects.
Use Unassigned funds dows to 15% of the FY24 budgeted expenses.
Then use Reserve Funds down to zero.
Then back to unassigned funds only if the project is criticle.