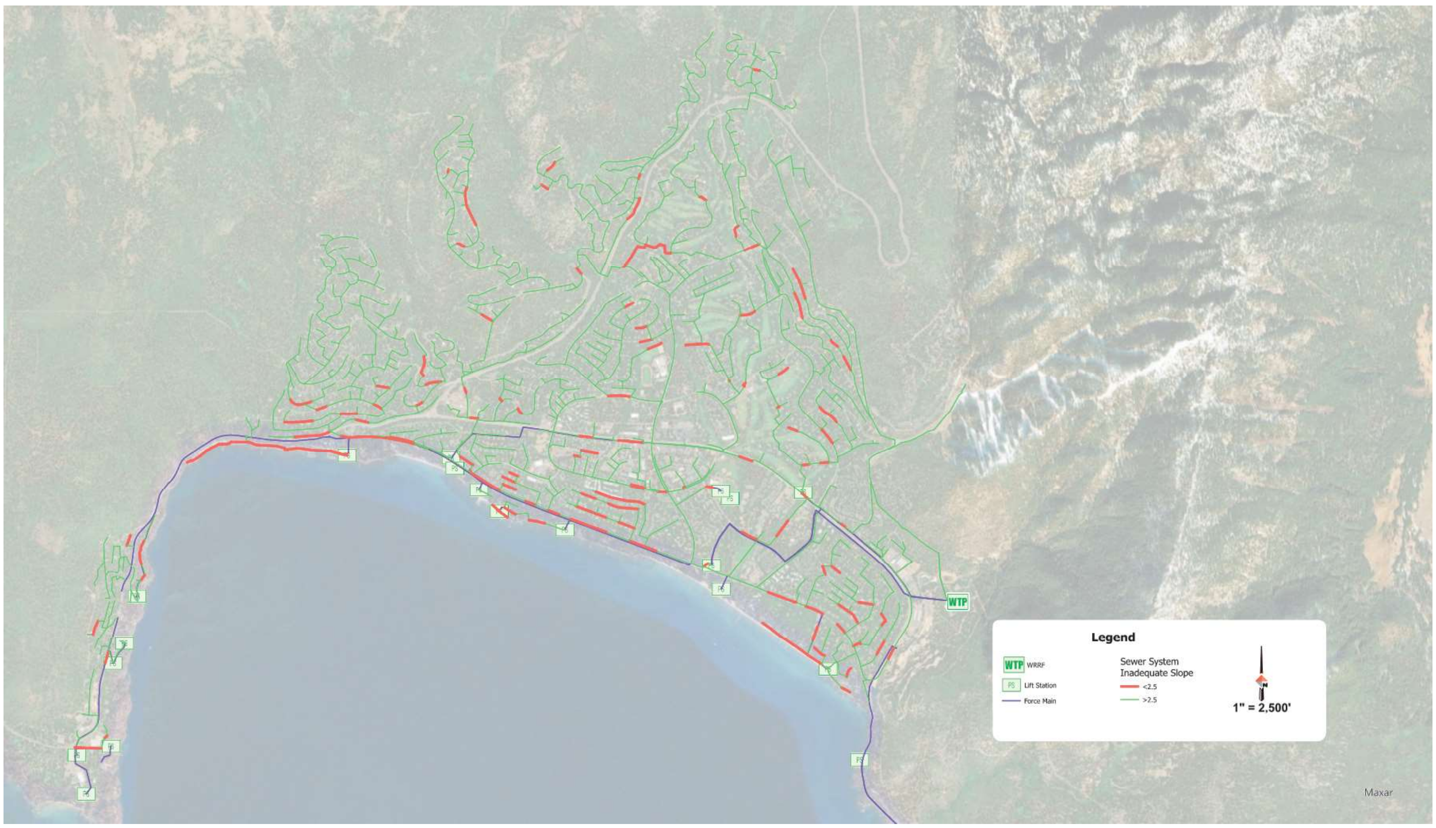

Figure 24: Gravity Sewer Mains with Inadequate Slope

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3.4.3 Lift Station Rehabilitation and Capacity

Many of the lift stations within the District still utilize the original equipment installed at the time of construction. This includes pumps and motors that are nearing or passed the end of their useful lifecycle. Table 52 gives the year of construction of each lift station, as well as the last known year each lift station was rehabilitated in some way.

Table 52: Lift Station Construction and Rehabilitation History

Lift Station ID	Year of Construction	Year of Last Rehab
SPS-1	1963	Unknown
SPS-2	1977	1997
SPS-4	Unknown	Unknown
SPS-5	1999	Unknown
SPS-6	1971	2018
SPS-7	1970	2014
SPS-8	1989	2011
SPS-9	1970	Unknown
SPS-10	1965	2011
SPS-11	1976	2019
SPS-12	1978	2005
SPS-13	1977	2005
SPS-14A	1996	Unknown
SPS-14B	1997	Unknown
SPS-15	1977	Unknown
SPS-16	2012	n/a
SPS-17	1980	Unknown
SPS-18	2015	n/a
SPS-19	1995	Unknown
SPS-20	2008	n/a

Of the District lift stations, SPS-10 is the most in need of rehabilitation. Originally constructed in 1965, it still utilizes the original pump installed. When rehabbed in 2011, the project installed onsite backup generators only. The pump used by SPS-10 is a vacuum-lift centrifugal system. This is uncommon for lift station design today. In fact, the age of the system is such that specific components of the system are unavailable commercially for purchase and would require custom fabrication if failure occurred. This is especially pressing as SPS-10 is located on the shore of Lake Tahoe.

It is recommended that the District begin a lift station rehabilitation program to address the aging infrastructure. The program would include an initial basis of design report (BDR) that would investigate the issues at each lift station, determine the scope of rehabilitation at each station, and then prioritize the order in which each lift station is addressed.

Additionally, per the analysis in Section 3.3, several capacity issues were identified in the District lift stations. The following sections identify the capacity issues, and the rehabilitation program BDR should ensure that these capacity issues are addressed.

3.4.3.1 SPS Pump Cycle Time

Three lift stations in the system do not maintain a minimum 10-minute cycle time during an average day. Table 53 gives a summary of the lift stations that do not meet this standard, and their current set points and depth. The recommended pump station rehabilitation program will ultimately address this issue; however, it is recommended that the District look into raising the pump on levels at each of these lift stations as a possible short-term solution. A higher pump on level will increase the amount of active storage within the wet well which will increase the fill time of the wet well and therefore increase the pump cycle time.

Table 53: Pump Cycle Time Deficiencies

SPS ID	Pump On/Off Level	Wet Well Depth (ft)
SPS-7	2.0/0.5 2.417/0.5	19
SPS-10	3.74/2.91 4.74/3.94	13.2

While changing the set points will provide immediate resolution, it is not completely necessary. The WTS-14 pump cycle times represent the design criteria are intended to lengthen the life of pumping infrastructure. As these pump stations are in need of rehabilitation, the pump cycle time is not a priority at this time. However, it is recommended that the District ensure all pump stations meet WTS-14 standards after they are rehabilitated.

3.4.3.2 SPS Pump Capacity

With one pump out of service, there is one lift station where pump capacity is limited, and it cannot handle the expected peak flow. SPS-14A pumps directly into the SPS-14B wet well at a flow rate of 55 gpm. The SPS-14B pump operating point is also 55 gpm. During normal, peak flow conditions, where SPS-14A is not pumping into SPS-14B, SPS-14B can handle the peak flow from the surrounding area. However, in the specific cases where SPS-14A is pumping during the same time that SPS-14B is seeing peak flows, the SPS-14B pumps are in exceedance of their capacity. Less than 10 residential customers discharge into the SPS-14A and SPS-14B wet wells. Due to the proximity to Lake Tahoe, the District should monitor the operational conditions at these lift stations to ensure that these lift stations are operating properly and with excess capacity. The pumps at SPS-14B may have to be upsized.

3.4.3.3 SPS Emergency Storage

There are multiple lift stations that do not meet the required emergency storage standard as described within WTS-14. Table 54 is summary of the lift stations that have deficiencies and whether these have backup power sources readily available. No action is necessary at the lift stations that available emergency power with automatic switch over.

Table 54: Emergency Power at Pump Stations with Storage Deficiency

SPS ID	Emergency Power Type	Automatic Switch Over?
SPS-1	Onsite Generators	Yes
SPS-2	Onsite Generators	Yes
SPS-6	Portable Generator w/ Cannon Plug	No
SPS-7	Onsite Generators	Yes
SPS-8	n/a	n/a
SPS-10	Onsite Generators	Yes
SPS-12	Offsite Generator WPS-2	Yes
SPS-18	n/a	No

SPS-8 is able to overflow into the SPS-1 sewershed during times of high flow or during power outages. As SPS-8 sees the second largest peak flows, behind SPS-1, it may be beneficial to install an onsite generator with an automatic transfer switch at this lift station. It can add another layer of redundancy to the system if SPS-1 is ever taken offline.

Although less than 10 residential customers discharge to the SPS-6 and SPS-18 lift stations; due to the close proximity to Lake Tahoe, permanent emergency power sources, or additional emergency storage, should be made available at these locations.

SPS-12 is the main lift station that collects flows from Crystal Bay and the surrounding area before discharging into the State Road 28/Lakeshore Drive combined force main. The SPS-12 lift station has an automatic bypass constructed into the wet well and it is a key overflow in the system that is consistently monitored. The SPS-12 overflow will convey flow to the NTPUD in California. Due to the regulatory concerns of moving sewer to another state, it would benefit the District to investigate installing a permanent emergency power source at this lift station.

3.5 Recommendations

Several deficiencies have been identified in the District gravity collection system and lift stations. Table 55 is a list of recommended projects to address those deficiencies. Project cost estimates and a prioritized CIP can be found in Section 6.0.

Table 55: Recommended Gravity Sewer and Lift Station Projects

Project	Project Description
Gravity Sewer Main Investigation	Investigate capacity deficiencies within SPS-1 and SPS-8 with field survey and field observation.
Gravity System Rehab/Replacement Program	Rehab or replace at risk sewer mains and manholes within the system as identified in the condition assessment.
Sewer Pump Station Condition Assessment and BDR	Perform a condition assessment of all lift stations and draft BDR to scope the rehab effort and prepare preliminary designs.
Sewer Pump Station Rehabilitation Program	Enact the Sewer Pump Station Condition Assessment and BDR by rehabilitating the pump stations in the recommended order.

4.0 WATER RESOURCE RECOVERY FACILITY (WRRF)

This WRRF condition assessment includes evaluations to identify the current condition, capacity, performance issues and safety status of the facility components along with the capability to meet the District's needs into the near and long-term future.

An overview of the WRRF is provided in Figure 25.

The purpose and general description of each unit process is described in each section, and Figure 26 illustrates the location of the unit process within the overall treatment train. The Condition Assessment identifies the design criteria objectives of each process and whether the facility satisfies State of Nevada Division of Environmental Protection (NDEP) criteria as well as other typical design guidelines or best practices, as applicable.

4.1 Facility History

The facility has undergone multiple expansions and modifications since construction of the original mechanical plant in 1962 developed by the Crystal Bay Development Company that included a circular treatment tank structure that included an aeration basin, a sludge retention tank, an aerobic digester, and a center clarifier. Other facilities included a chlorine contact basin and effluent pump system and an effluent filter after disinfection for water discharged to percolation drain field trenches. Effluent was also pumped to spray irrigation. Solids handling facilities included several sludge drying beds.

- 1971 improvements included construction of the influent bar screen, comminutor, aerated grit chamber, aeration basins with 12 surface aerators, secondary sludge pump station for RAS and WAS pumping, two secondary clarifiers and the 500,000-gallon effluent storage reservoir. The pre-existing plant from the 1962 project was converted into a two-stage aerobic digester with a center sludge thickener. Digested sludge was conditioned with polymer pumped to an air floatation system and sludge storage tank. Sludge from the storage tank was treated with lime and ferric chloride and dewatered via a vacuum filter.
- In 1978 facility modifications included addition of a new rotating influent screen, addition of a control building, digester modifications including aeration improvements and a dome cover over the tank, solids handling and chemical storage building modifications including a corrosion inhibitor chemical addition and ferric chlorine storage. This project included demolition of the chlorine contact basin and effluent pump structure.
- 1986 improvements included demolition of the sludge air floatation and vacuum filter system and installation of the elevated platforms, dewatering centrifuges, centrifuge feed pump systems polymer feed systems, sludge cake conveyor and sludge storage hopper. Additional modifications to the aerobic digester tank were also included to remove the center clarifier rotating skimmer and scraper and add coarse bubble aeration into the center sludge storage tank along with various piping changes at the digester tank facility.
- 1988 improvements included installation of a packaged fiberglass metering manhole ahead of the facility headworks.

- In 1991, major modifications included replacement of the surface aerators in the aeration basins with jet aeration circulation pumps and positive displacement blowers, construction of the recirculation pumps and buildings, blower building, modifications to the digesters including removal of the dome cover, addition of chemical scrubber and carbon filter odor control facilities, HVAC improvements, recoating of clarifier mechanisms, installation of density current baffles in the clarifiers, replacement of the ferric chloride and corrosion inhibitor tank and feed systems with liquid hypochlorite feed storage tanks and feed pumps, new generator and fuel storage, and electrical improvements in support of these modifications.
- 1994 improvements included replacement of aeration piping and diffusers in the digester tanks.
- In 1997, clarifier mechanisms (2) and return activated sludge (RAS) pumps (2) were replaced. The clarifier replacements included a new WesTech drive assembly, walkway and operating platform, spiral sludge transport rakes, scum collection arms, scum troughs, and appurtenances. The existing weirs and scum baffles remained and were not replaced. The RAS pump replacement included two new, Wemco-Hidrostal, 5 horsepower, screw centrifugal pumps.
- Year 2004 improvements included modifications to the solids dewatering systems, headworks solids handling equipment and installation of baffle walls in aeration basins S2 and N2. The work included modification of the existing centrifuge support platforms, installation of two new centrifuges, new polymer feed system, cake conveyance, headworks screenings washer/compactor, grit washer, a new PLC to operate the new centrifuges and the existing centrifuge #3 and installation of fiberglass baffle walls in the noted aeration basins.
- Year 2010 improvements included replacement of the chemical and carbon odor control scrubber system with a dry media air filtration and odor control system.
- Year 2015 improvements included rehabilitation work in the facility headworks.
- Year 2020 improvements included upgrades to the aeration basis air system including structural mods, blower replacement, air supply piping, electrical, instrumentation and controls for aeration.

4.2 Facility Description

The current treatment process train includes raw influent flow metering, fine screening, grit removal, secondary biological treatment, clarification, and disinfection along with waste sludge digestion and mechanical sludge dewatering. Figure 25 includes a site overview of the overall WRRF.

Figure 25: Water Resource Recovery Facility–Site Overview

Figure 26 provides a current process flow diagram and includes the processes and components listed in Table 56.

Table 56: WRRF Processes and Components

WRRF Process	Components
Preliminary Treatment	Influent Wastewater Screening Grit Removal
Biological Treatment Process	Aeration Basins (North & South) Secondary Clarifiers (North & South) Secondary Sludge Pump Station
Return Activated Sludge Pump System	-
Waste Activated Sludge Pump System	-
Facility Support Systems	Plant Waste – Return Lift Station Non-Potable Water Pump Station Odor Control System
Effluent Disinfection	Chemical Building Effluent Storage Tank Effluent Pump Station
Solids Handling Facilities	Waste Sludge Holding Tanks Sludge Pump Station Sludge Dewatering Equipment

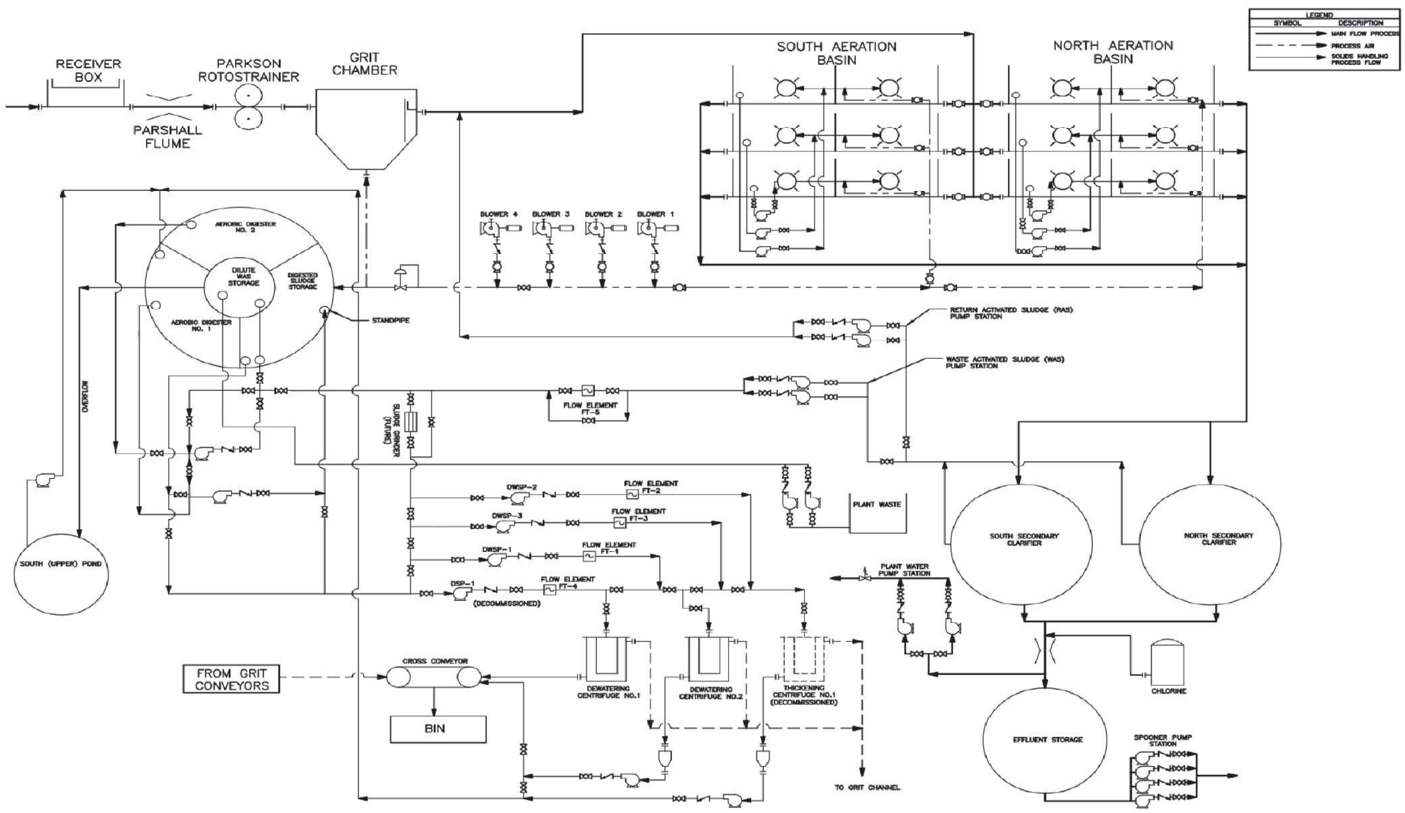

Figure 26: Water Resource Recovery Facility - Process Flow Diagram

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4.3 Historic Flows and Organic Loads

4.3.1 Historic Wastewater Flows

The tables and figures below document the last three years of flow monitoring of both the influent flow meter flume at the front of the plant and the effluent flow meter following the clarifiers.

The key metrics are the annual average daily flow, the maximum monthly flow and the peak daily flow defined as follows:

> Annual Average Daily Flow (AADF): The average of the daily flows occurring over a 24-hour period. This is determined from the total annual flow divided by the 365 days, or the averages of the monthly values found by the total monthly volume divided by the number of days in the month. This metric must be established in light of potential wet weather and dry weather implications. The wet weather or high infiltration period may need to be used as the average daily flows if and when there is a significant difference between wet and dry weather seasons. Average daily flows plotted are for the full year, as there is not a significant influence of infiltration or extended wet weather to otherwise account for in establishing this value. Treatment facility “design capacity” typically refers to the average daily flow capacity (AADF) and the average design BOD_5, though individual unit processes are typically sized for higher hydraulic and/or solids loading rates than this average.
>
> Maximum Monthly Flow: This value is the average of the daily flows occurring during the month of the highest total flows for the month. The maximum month flows have occurred in the month of July over the last three years.
>
> Peak (Max) Daily Flow: This value is the average of the peak flows sustained for a 24-hour period. A three-day running average during the peak month can provide a solid basis of design and avoid designing around an extreme outlier in the data set. A three-day average is not used in the chart for this metric, as the data was consistent with no major outliers to distort the data.

Figure 27 Illustrates the average monthly flows derived from the total volume for the month divided by the number of days. These are influent flows, as recorded by the influent flow meter.

Figure 28 illustrates the observed peak daily influent flows for each month over the period of record.

Table 57 provides a tabulation of the average monthly flows for the influent flow meter, and Table 58 provides the same tabulation for the effluent flow meter. Note, the average difference between the average monthly influent flows and average monthly effluent flows is 13.3% over the entire period of record with variations from approximately 1% to over 30% different with the effluent flows generally trending higher than the recorded influent flows. The differences are inconsistent month to month and even comparing the same months in different years. For reference, the average of 13.3% would be approximately 115,000 gpd additional flow recorded at the effluent meters as compared to the influent flow meter. The majority of this discrepancy is associated with use of potable water at the facility for various utility water needs.

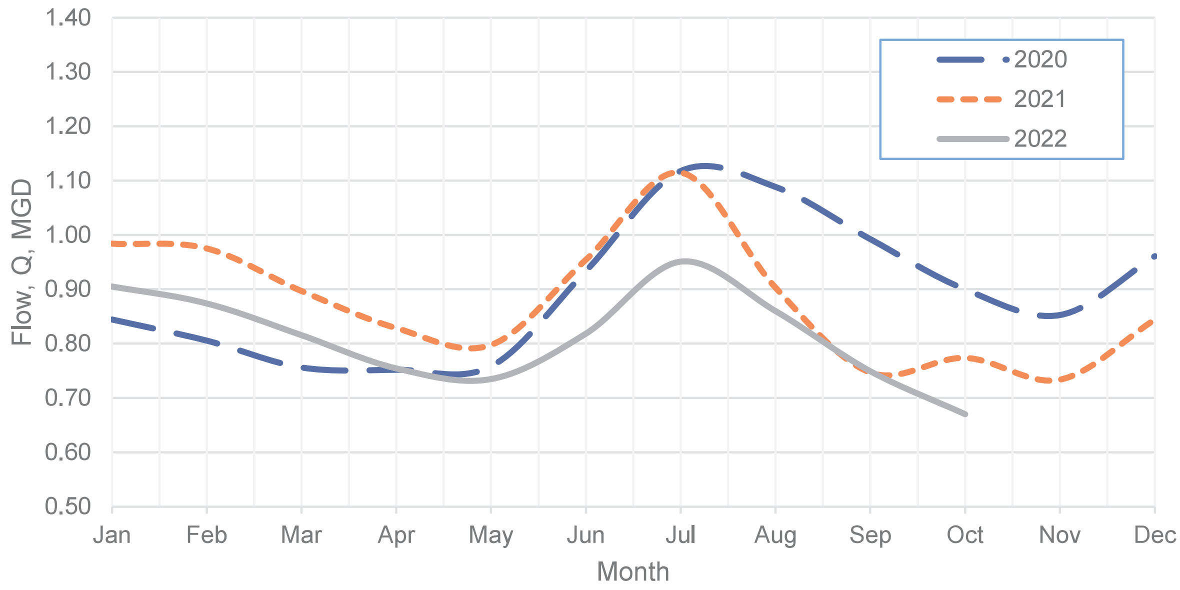

Figure 27: Ave. Monthly Influent Flows (2020 – 2022)

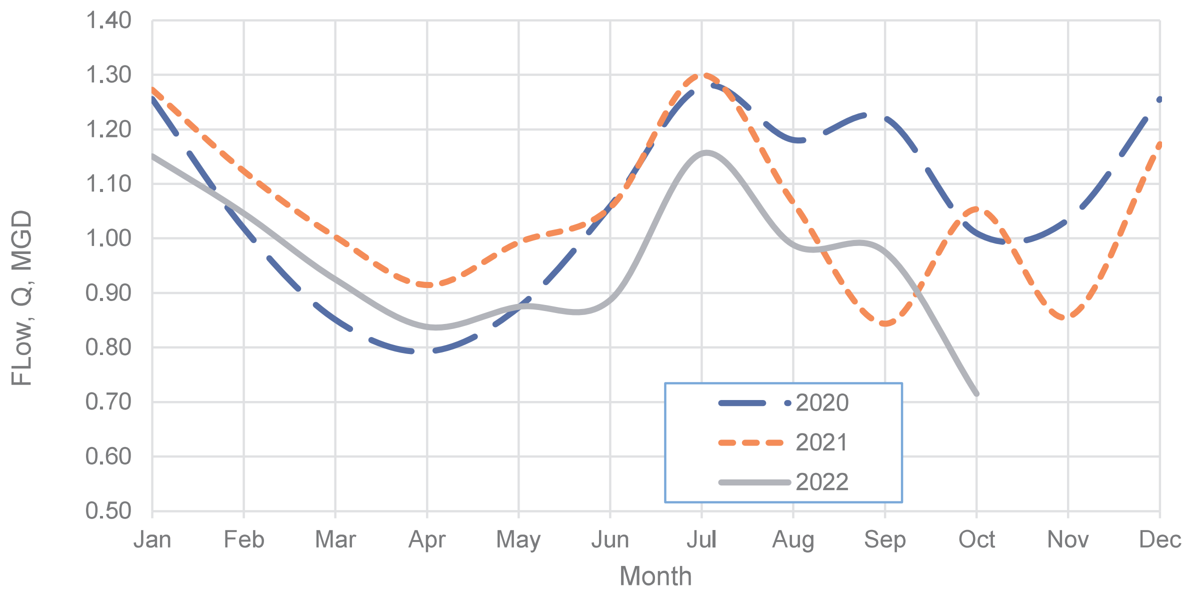

Figure 28: Peak Daily Influent Flows (2020 – 2023)

Table 57: Last Three Years of Monthly Flows (Influent Flow Meter)

Date	Average Day (MGD)	Peak Day (MGD)	Monthly Volume (MG/month)
Jan-20	0.844	1.256	26.177
Feb-20	0.806	1.018	23.37
Mar-20	0.756	0.851	23.436
Apr-20	0.752	0.793	22.563
May-20	0.759	0.874	23.516
Jun-20	0.934	1.059	28.022
Jul-20	1.118	1.279	34.66
Aug-20	1.089	1.181	33.749
Sept-20	0.992	1.221	29.765
Oct-20	0.900	1.010	27.903
Nov-20	0.853	1.035	25.5832
Dec-20	0.961	1.256	29.789
Jan-21	0.984	1.273	30.501
Feb-21	0.975	1.123	27.308
Mar-21	0.897	1.003	27.805
Apr-21	0.829	0.915	24.861
May-21	0.798	0.993	24.734
Jun-21	0.954	1.057	28.632
Jul-21	1.115	1.300	34.558
Aug-21	0.903	1.065	27.979
Sept-21	0.747	0.844	22.408
Oct-21	0.774	1.054	23.997
Nov-21	0.734	0.856	22.01
Dec-21	0.844	1.173	26.174
Jan-22	0.905	1.151	28.068
Feb-22	0.874	1.046	24.478
Mar-22	0.816	0.925	25.284
Apr-22	0.755	0.838	22.638
May-22	0.735	0.875	22.781
Jun-22	0.819	0.887	24.565
Jul-22	0.951	1.156	28.529
Aug-22	0.860	0.988	26.655
Sept-22	0.749	0.975	22.478
Oct-22	0.670	0.715	20.772

Table 58: Last Three Years of Monthly Flows (Effluent Flow Meter)

Date	Average Day (MGD)	Peak Day (MGD)	Monthly Volume (MG/month)
Jan-20	0.967	1.301	29.9881
Feb-20	0.937	1.108	27.183
Mar-20	0.869	0.968	26.952
Apr-20	0.862	0.974	25.866
May-20	0.822	0.921	25.487
Jun-20	1.018	1.197	30.539
Jul-20	1.207	1.360	37.412
Aug-20	1.153	1.334	35.743
Sept-20	1.046	1.294	31.384
Oct-20	0.946	1.019	29.326
Nov-20	0.920	1.059	27.609
Dec-20	1.010	1.323	31.325
Jan-21	1.016	1.307	31.497
Feb-21	1.248	1.458	34.957
Mar-21	1.204	1.324	37.31
Apr-21	1.099	1.296	32.96
May-21	0.853	1.210	26.439
Jun-21	0.916	1.080	27.48
Jul-21	1.096	1.611	33.9745
Aug-21	0.869	1.104	26.9428
Sept-21	0.650	0.747	19.497
Oct-21	0.954	1.299	29.562
Nov-21	1.026	1.175	30.794
Dec-21	0.924	1.109	28.645
Jan-22	0.895	1.132	27.744
Feb-22	0.900	1.071	25.196
Mar-22	0.887	1.508	27.496
Apr-22	1.070	1.740	32.11
May-22	0.939	1.109	29.107
Jun-22	1.026	1.131	30.775
Jul-22	1.099	1.350	34.073
Aug-22	1.035	1.211	25.87
Sept-22	0.906	1.063	27.189
Oct-22	0.793	1.000	24.582

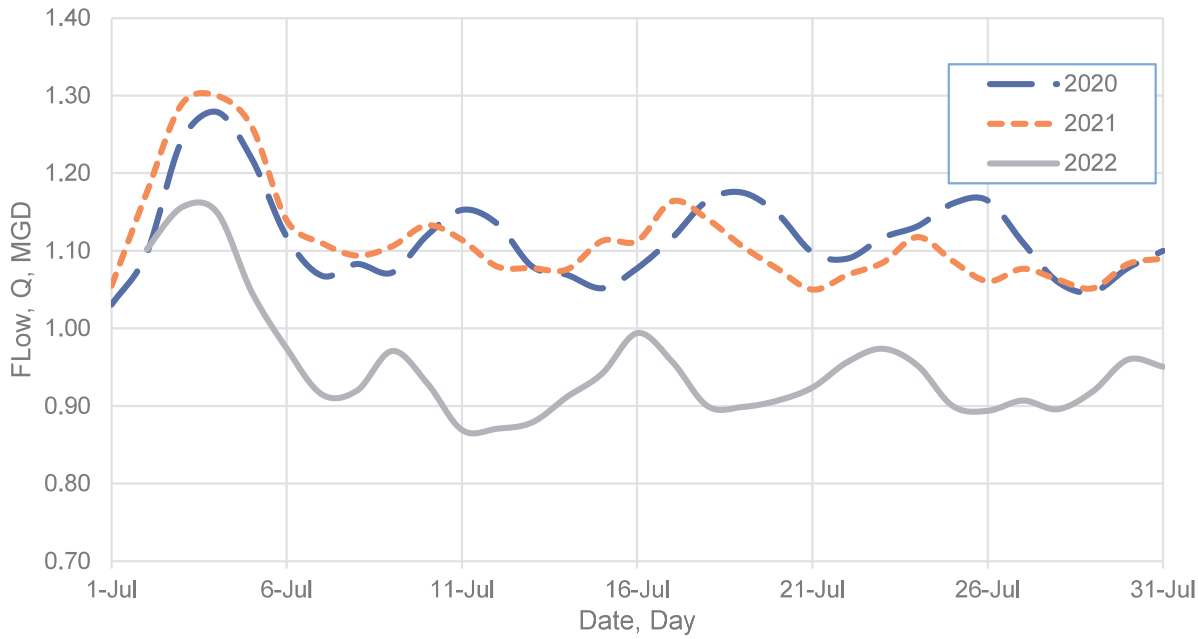

Figure 29: Daily Flows During Peak Month

As noted in Section 1.2, the peak flow months are July and August. Of further note is the significant increase in flows in years 2020 and 2021 resulting from historic low vacancy rates within the District as people worked remotely or chose to stay in the Tahoe area during the COVID pandemic lockdown restrictions.

4.3.2 Historic Wastewater Organic Loads

Roughly the last three years of average monthly influent BOD_5, CBOD, TSS are summarized in the following tables and charts.

The organic concentrations of the influent are representative of higher strength wastewater with low influence of groundwater infiltration or surface water inflow mixing with the domestic wastewater. Original design values for the plant BOD and TSS were approximately 240 mg/L and 264 mg/L in comparison. As shown in the tables and charts, the influent BOD is rarely less than 400 mg/L and more typically two or more times this original design value. The TSS trends generally the same and always significantly greater than the original design values. Fortunately, the flows are generally about one third of the design flow of 3.0 MGD such that the total organic load in pounds per day remains within the conceptual treatment capacity of the plant.

Table 59: Ave Monthly BOD, TSS for Three Years

Date	Average Monthly BOD5	Average Monthly TSS
Jan-20	389.40	386.23
Feb-20	438.75	399.45
Mar-20	466.00	496.35
Apr-20	366.40	487.70
May-20	402.50	490.13
Jun-20	445.00	543.67
Jul-20	519.50	525.10
Aug-20	493.50	478.35
Sept-20	414.75	441.07
Oct-20	450.20	395.87
Nov-20	396.00	423.13
Dec-20	482.00	442.74
Jan-21	439.50	408.06
Feb-21	465.00	407.36
Mar-21	437.50	381.23
Apr-21	335.00	423.23
May-21	444.00	459.81
Jun-21	593.75	450.17
Jul-21	489.20	469.13
Aug-21	525.25	443.61
Sept-21	655.00	464.13
Oct-21	675.00	400.13
Nov-21	861.25	439.97
Dec-21	733.40	449.68
Jan-22	585.80	402.03
Feb-22	906.50	397.79
Mar-22	584.60	418.16
Apr-22	362.25	241.77
May-22	335.50	420.58
Jun-22	540.00	447.57
Jul-22	343.33	425.35
Aug-22	312.00	932.00
Sept-22	637.50	667.50
Oct-22	480.00	-

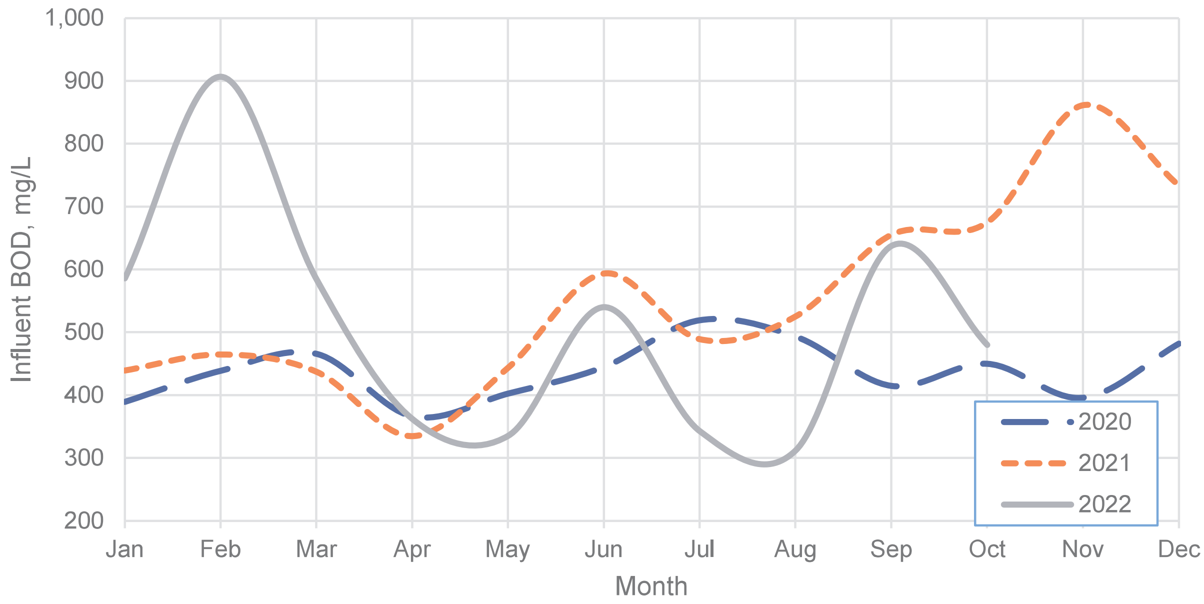

Figure 30: Ave. Monthly Influent BOD_5; (2020 – 2022)

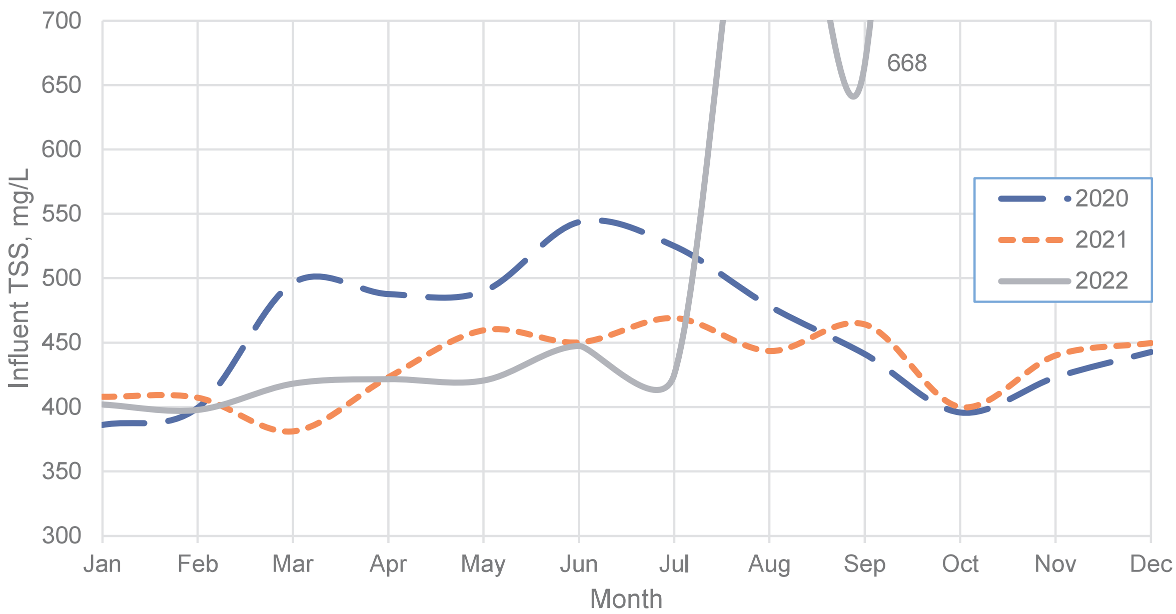

Figure 31: Ave. Monthly Influent TSS (2020 – 2022)

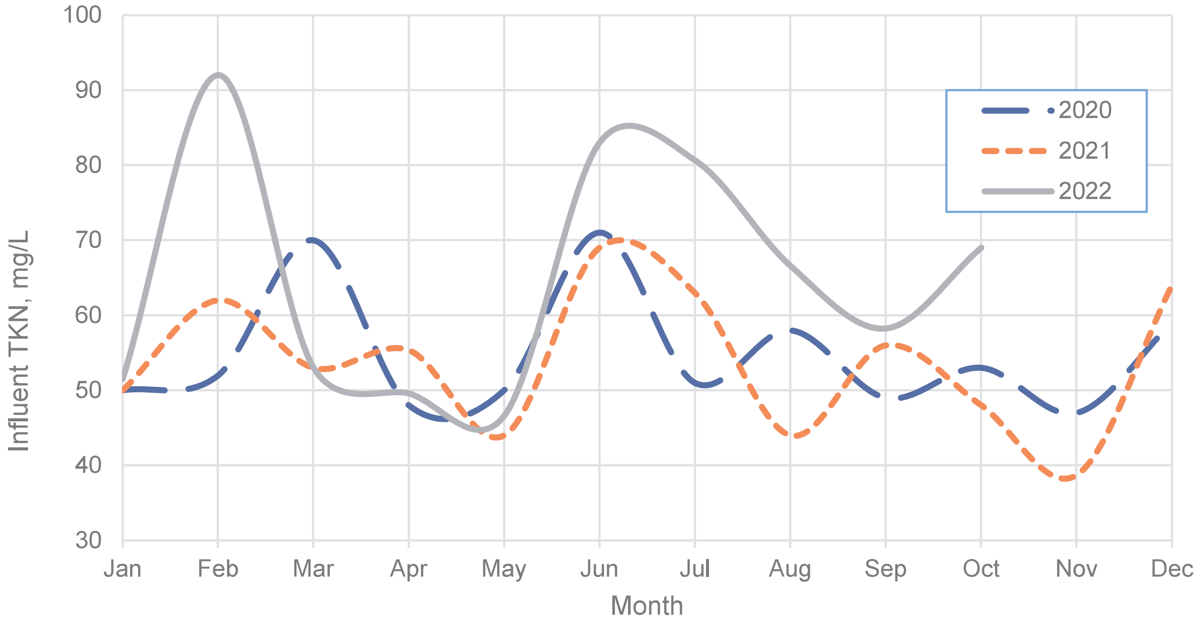

Figure 32: Ave. Monthly Influent TKN (2020 – 2022)

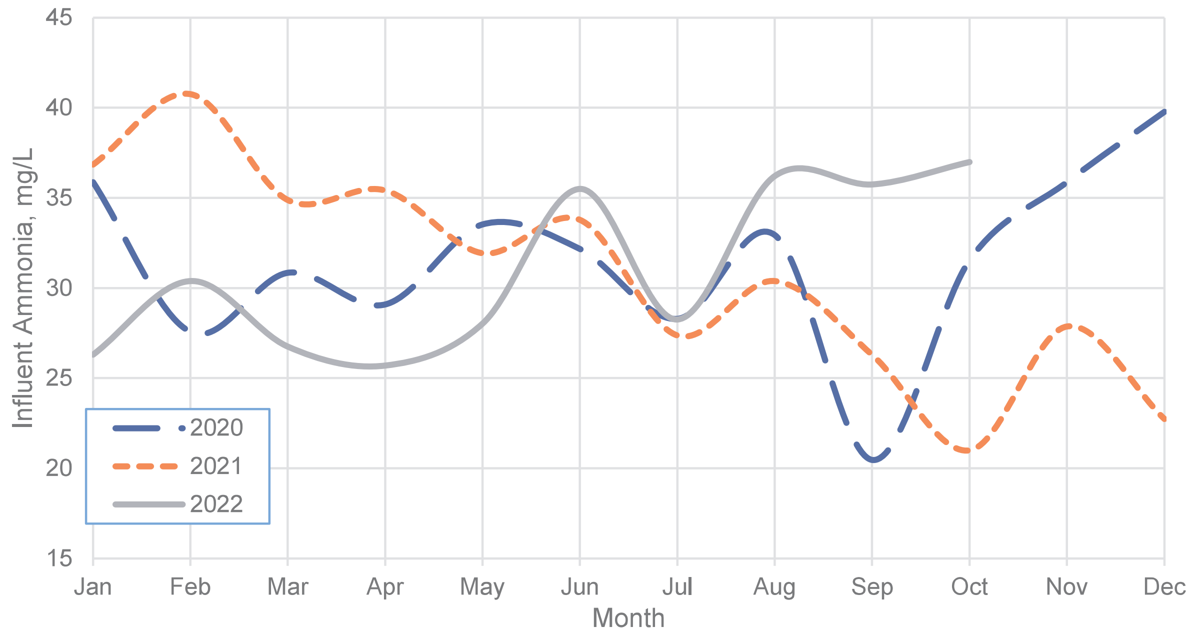

Figure 33: Ave. Monthly Influent Ammonia (NH_3-N) (2020 – 2022)

4.3.3 Summary of Flows and Loads

In the absence of daily sampling data to determine peaking factors for the organic loading the peaking factors in Table 60 are estimated with guidance included in the Water Environment

Federation, Manual of Practice 8, Table 3.5. The peaking factors are based on the statistical analysis of a large data set of activated sludge treatment facilities to develop the peaking factors based on the average daily flow of the facility.

Table 60: Summary of Influent Flows and Organic Loads

Parameter	Value
Average Annual Flow, MGD	0.866
Maximum Monthly Flow, MGD	1.06
Maximum Daily Flow, MGD	1.25
Average Influent BOD5, mg/L	502
Max Month Peak Factor	1.38
Max Monthly BOD5, mg/L	693
Peak Day Factor for BOD5	1.62
Max Daily BOD5, mg/L	812
Average Influent TSS, mg/L	445
Max Month Peak Factor	1.38
Max Monthly TSS, mg/L	616
Peak Day Factor for TSS	1.81
Max Daily TSS, mg/L	808
Average Influent TKN, mg/L	59
Max Month Peak Factor	1.13
Maximum Monthly TKN, mg/L	67
Peak Day Factor for TKN	1.27
Maximum Day TKN, mg/L	75

The actual design values for the organic loading of the plant were substantially lower than these values. Applicable design values versus recent actual data are compared in Table 61. Of note is the typical concentration of BOD_5 and TSS. The observed values are approximately double the facility design criteria. The design assumed influent BOD and TSS concentrations of approximately 240 mg/L and 260 mg/L, respectively. These are very typical domestic wastewater concentrations. Actual observed average values are approximately 500 mg/L and 445 mg/L, respectively. These values are considered high-strength wastewater and well above the typical range for domestic wastewater. The higher concentrations are indicative of very low sewer collection system infiltration and overall conservative water use.

Typically, these much higher influent loading values would present significant treatment challenges for a treatment facility if the influent flows were also approaching the design flow capacity, as the aeration system, tanks and clarifiers would not be able to maintain treatment performance. The recent influent flows, however, are substantially less than expected with a hydraulic design average day of 3.0 MGD versus 0.87 MGD observed average daily flows. Therefore, though the organic concentration is substantially higher, the total daily organic loading in total pounds per day is still well below the assumed design loading.

Table 61: Design v/s Recent Flows and Organic Loads

Parameter	Design Values	Average Actual Values (2020-2023)
Average Annual Flow, MGD	3.0	0.87
Maximum Daily Flow, MGD	4.5	1.25
Average Influent BOD5, mg/L	240	**502**
Average Influent BOD5, lb/day	6,000	3,640
Average Influent TSS, mg/L	264	445
Average Influent TSS, lb/day	6,600	3,230

The influent flows to the WRRF are not expected to increase over time. Thus, the overall capacity of the plant is significantly higher than the current and projected flows and loads in the future, and the facility has adequate tank and process capacity to treat the incoming loads.

4.3.4 Historic Treated Effluent Water Quality

The following figures provide summary representations of the measured treated water quality for the years 2020-2022. Effluent BOD_5 and TSS are consistently below 20 mg/L with values dropping below 10 mg/L in the warmer treatment months. The CBOD values are a measurement of the oxygen uptake of the sample with inhibitors added to the sample to inhibit any nitrifying bacteria from imparting an oxygen demand on the sample. The BOD5 and CBOD values tend to trend very closely together tending to indicate there generally are not significant nitrifying bacteria present in the effluent.

Disinfection: The disinfection efficiency is measured at the Spooner Pump Station. Values are frequently near non-detect at values reported at 1 CFU/100 ml. Some excursions have occurred potentially from equipment malfunctions, rapid spikes in the chlorine demand, or sampling/testing anomalies.

Nitrification: The effluent Total Kjeldahl Nitrogen (TKN) remaining in the effluent is predominantly an indication of the amount of influent ammonia converted (i.e., nitrified) to nitrite and then nitrate. From June through December the warmer temperatures enable nitrification, and the effluent TKN is very low, indicating most of the nitrogen in the wastewater has been converted from ammonia to nitrite/nitrate. Total effluent nitrogen values are not tracked at the facility to indicate how much of nitrate is removed through denitrification to release from the wastewater as nitrogen gas. The facility is not designed for nutrient (i.e., nitrogen and phosphorus) removal and not expected to significantly reduce the total nitrogen or phosphorous. As such, the effluent total phosphorus values suggest very little phosphorous is converted such that it can be removed in the sludge leaving the WRRF.

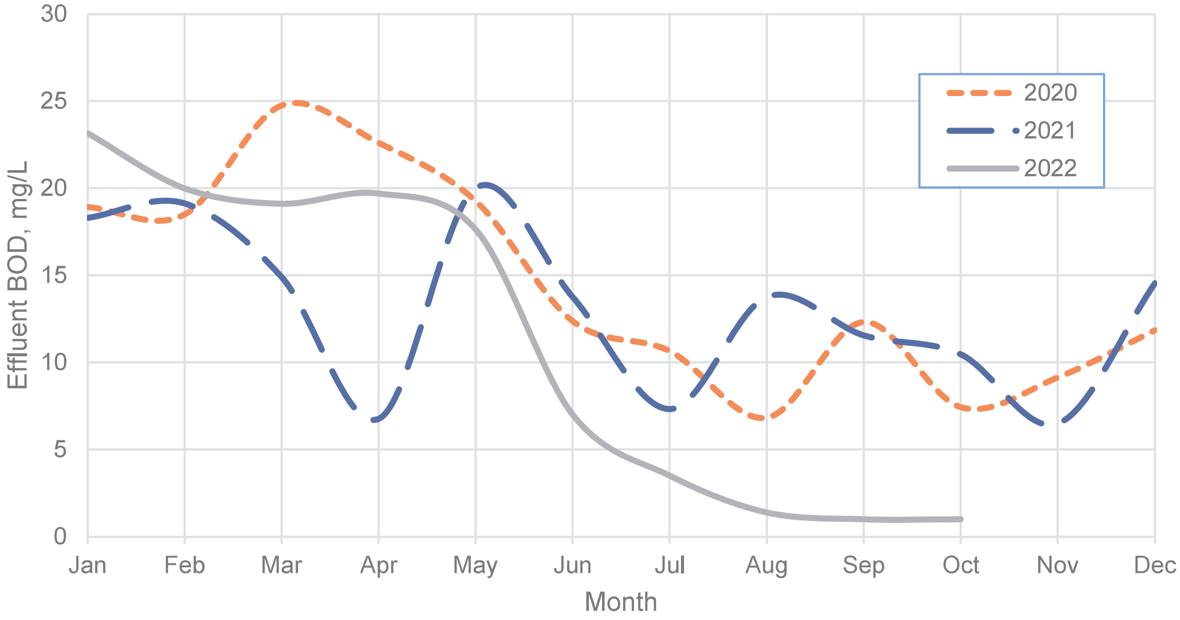

Figure 34: Ave. Monthly Effluent BOD_5 (2020 – 2022)

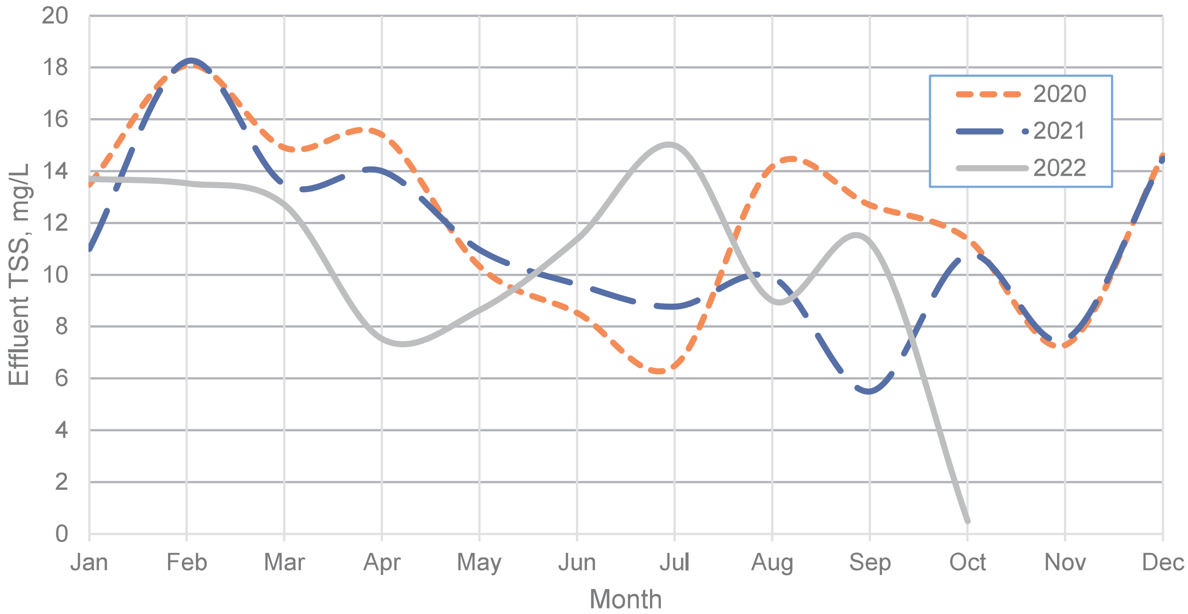

Figure 35: Ave. Monthly Effluent TSS (2020 – 2022)

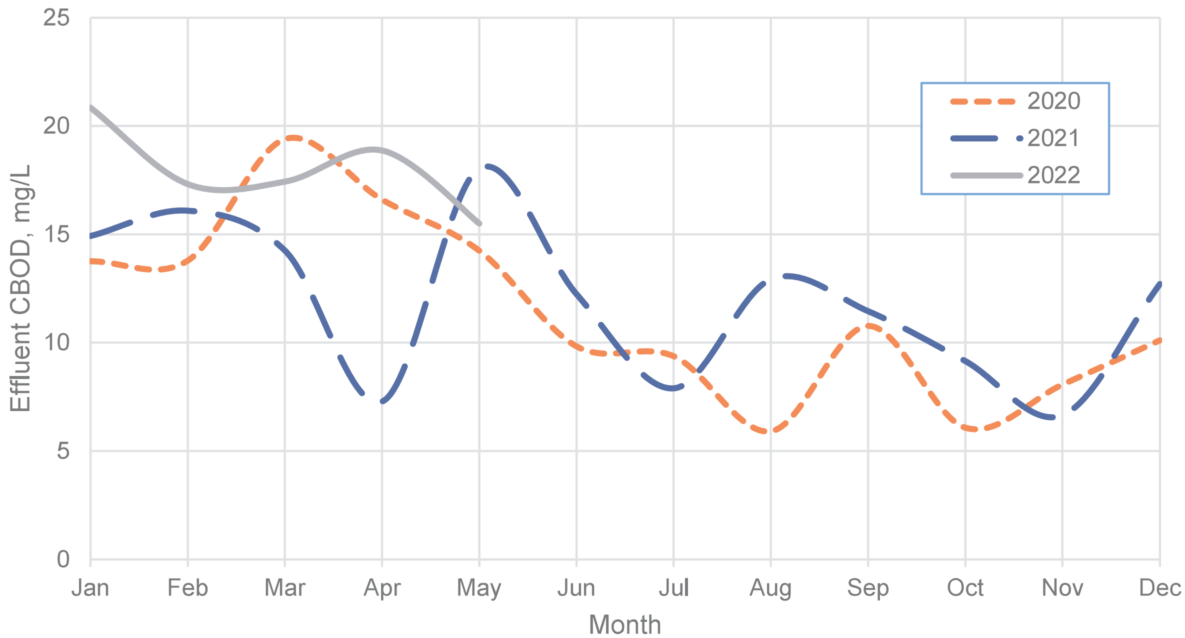

Figure 36: Ave. Monthly Influent CBOD (2020 – 2022)

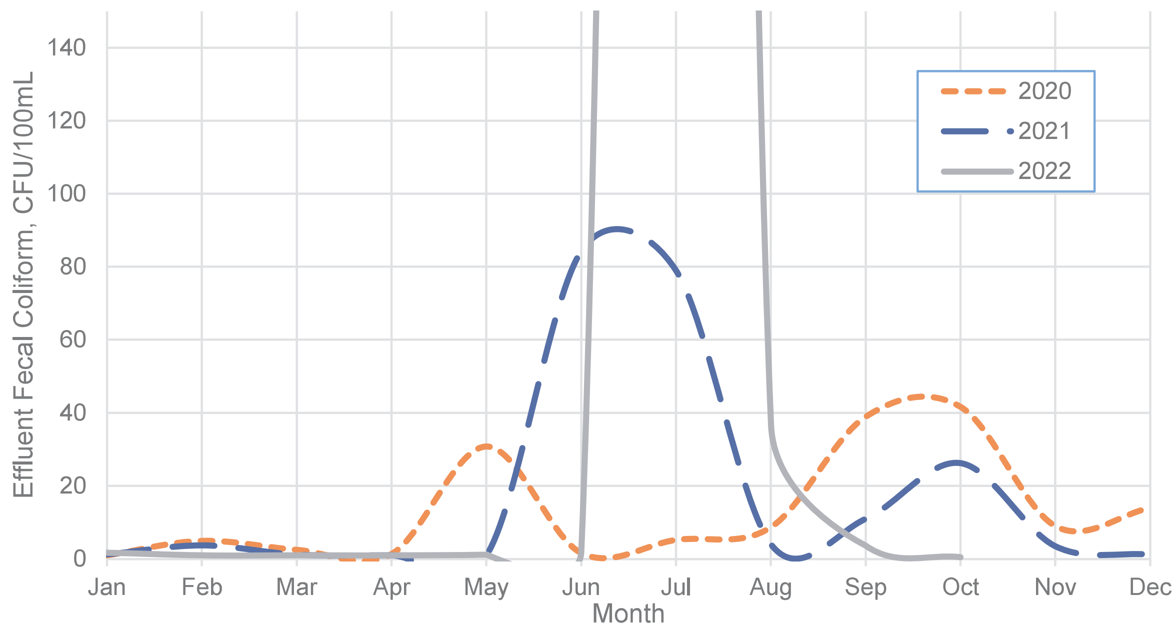

Figure 37: Ave. Monthly Fecal Coliform (2020 – 2022)

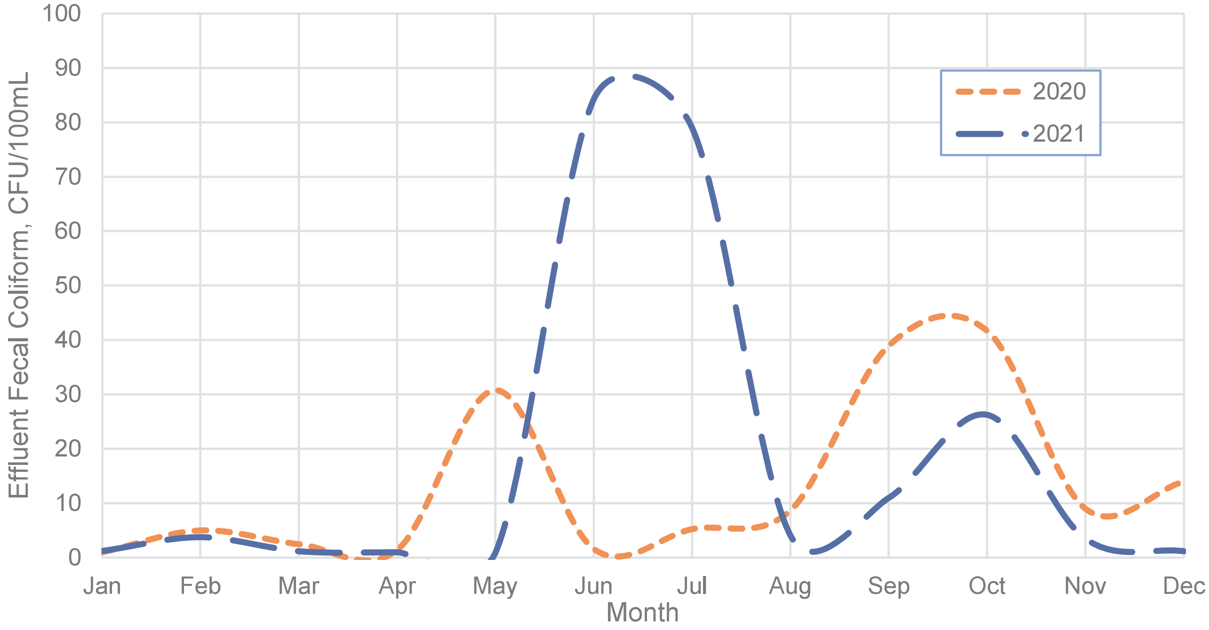

Figure 38: Ave. Monthly Fecal Coliform (2020 – 2021)

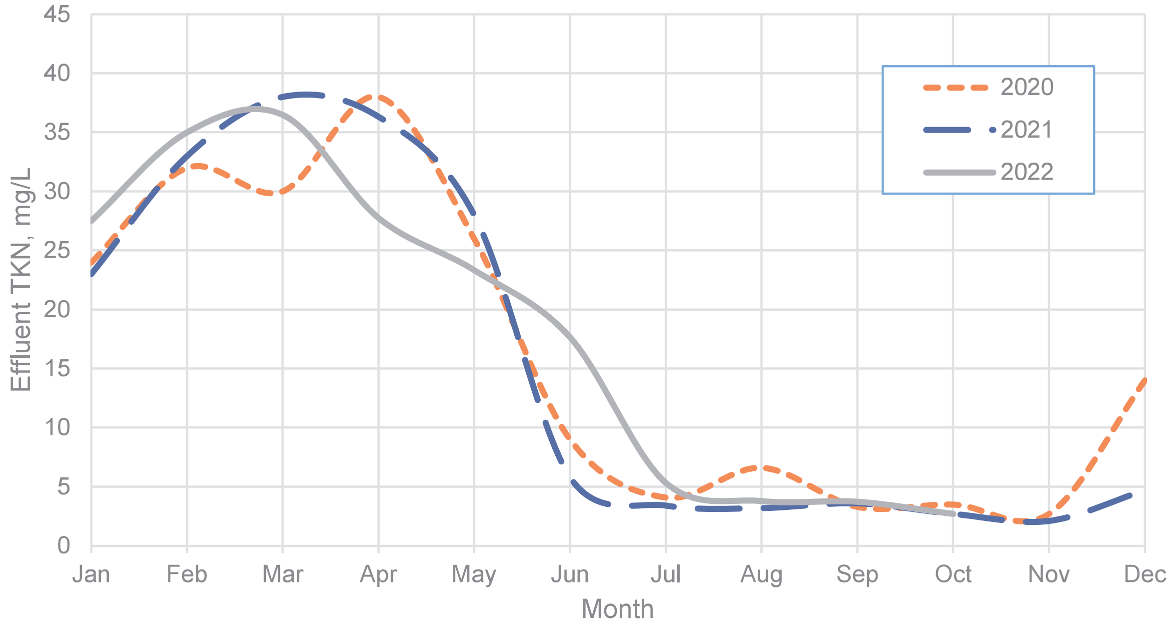

Figure 39: Ave. Monthly Effluent TKN (2020 – 2022)

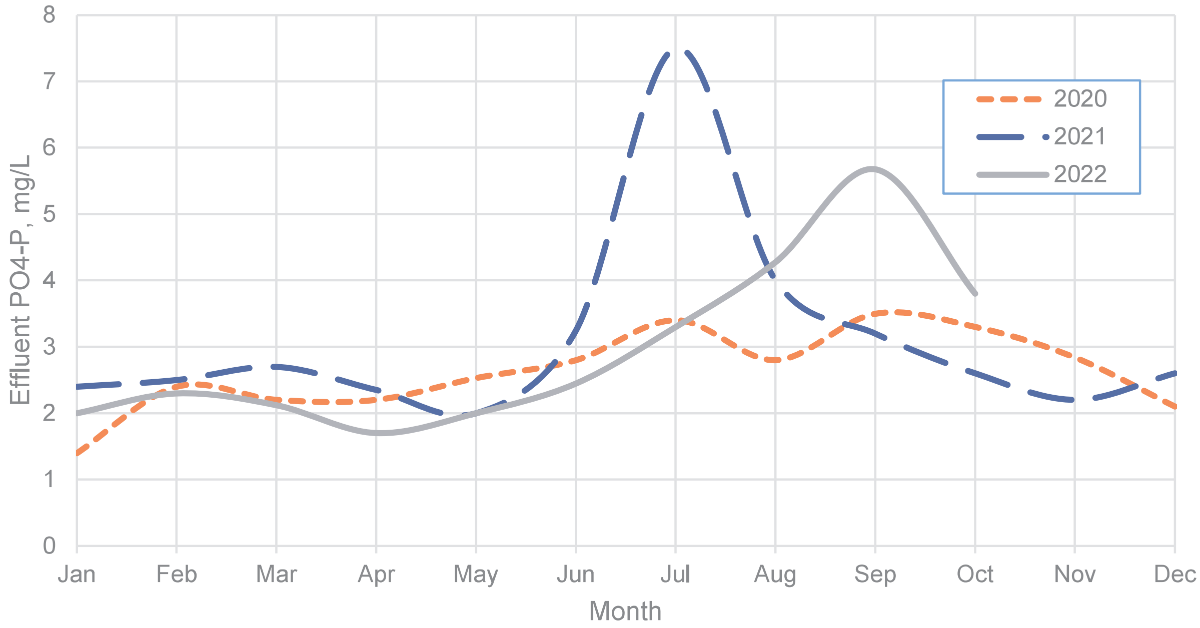

Figure 40: Ave. Monthly Effluent Phosphorous (PO_4-P) (2020 – 2022)

4.4 Effluent Discharge Water Quality Requirements

The secondary treated effluent from the Incline Village WRRF is pumped out of the Lake Tahoe Basin via a 21-mile pipeline over the Sierra Nevada Mountains to the Carson Valley. The treated effluent discharges to the Wetlands Enhancement Facility near Hot Springs Mountain in northern Douglas County. Irrigation use with the effluent also occurs at the Schneider Ranch and the new Clear Creek Tahoe Country Club.

The District received a new discharge permit in June of 2023 for the period of June 1, 2023, to May 31, 2028. Three separate outfall locations are included in the discharge permit including the constructed wetland and irrigation outfalls to Schneider Ranch and Clear Creek Tahoe Golf Course. A total of 13 sampling locations are identified in the permit with periodic monitoring requirements, seven (7) of which are monitoring wells around the wetlands enhancement facility. The key discharge limitations for outfall to the wetlands include the following parameters in Table 62, and Table 63 includes the limits for the monitor well samples. The full discharge permit is included in the Appendix D.

Table 62: Discharge Permit Limits – Wetlands Outfall

Parameter	Limits
BOD5	30 mg/L Monthly Average
BOD5	45 mg/L; Daily Maximum
BOD5	>=85% removal; Monthly Ave Min.
TSS	30 mg/L Monthly Average
TSS	45 mg/L; Daily Maximum
TSS	>=85% removal; Monthly Ave Min.
Coliform, Fecal	<=23 MPN/100 mL; Monthly Geometric Mean
Coliform, Fecal	<= 240 MPN/100 mL; Daily Max

Table 63: Discharge Permit Limits – Monitoring Wells

Parameter	Limits
Chloride (as Cl)	< = 400 mg/L
Nitrogen, total	< = 10 mg/L (Daily Maximum)

4.5 Water Resource Recovery Facility (WRRF) Condition Assessment

This WRRF condition assessment includes evaluations to identify the current condition, capacity, performance issues and safety status of the facility components along with the capability to meet IVGID's needs into the near and long-term future.

4.5.1 Influent Collection Facilities

4.5.1.1 Facility Purpose and Design Criteria

The influent sewer collection facilities at the WRRF conveys raw wastewater from three different sources in the District, combines the flows, and then directs it to the preliminary treatment facilities. Flow monitoring and sample collection facilities are two typical purposes of the influent sewer collection facilities. No specific design criteria apply to how influent enters the WRRF other than having adequate capacity for the peak expected flows.

4.5.1.2 Process Description, Condition and Capacity

Wastewater arrives at the WRRF through a sewer force main from the SPS-01 and SPS-08 lift stations and a gravity sewer inverted siphon from the Sewershed 000. After combining at a manhole at the edge of the property the wastewater passes through a Parshall flume flow measuring device shown as in Figure 41 and then through concrete flow channels en route to the single automatic influent screening equipment. A composite sampler is also in place to pull influent wastewater samples from just below the flume structure, and a total suspended solids (TSS) sensor is also located in the influent channel.

Figure 41: Influent Parshall Flume

The concrete influent channels split downstream of the flume to direct the flow either to the automatic screen or to a bypass channel with a fixed bar rack. The bypass channel directs flow directly to the grit chamber, bypassing the automatic fine screen equipment. See Figure 42. Flows can be directed to bypass the screen and go directly to grit or bypass both the automatic screen and the grit chamber. Flows cannot go through the screen and subsequently bypass the grit chamber. The influent channel toward the fine screen terminates and transitions to a closed pipe before connecting to the fine screen. The discharge from the screen routes the wastewater back to the grit chamber.

Ahead of this transition the channel tends to accumulate oil and grease, see Figure 43. Rather than attempt to remove the material from this location, the general practice is to break up the grease with a hose stream of water and get the material to pass downstream where some can be picked up on the screen, but most will pass through potentially causing downstream issues or make it all the way to the effluent. Ideally, the FOG is broken down farther in the aeration basins and incorporated into the sludge wasted from the facility, as there is not another way to isolate and remove the FOG. This condition/practice is not atypical of treatment facilities, and, fortunately, FOG has not been excessive to the point of causing significant foaming issues in the aeration basins or issues in the fine screen or downstream in the final clarifiers. IVGID has installed a spray system in this location to alleviate the buildup of FOG for eight to nine months out of the year.

Figure 42: Influent Channels, Composite Sampler Weather Enclosure

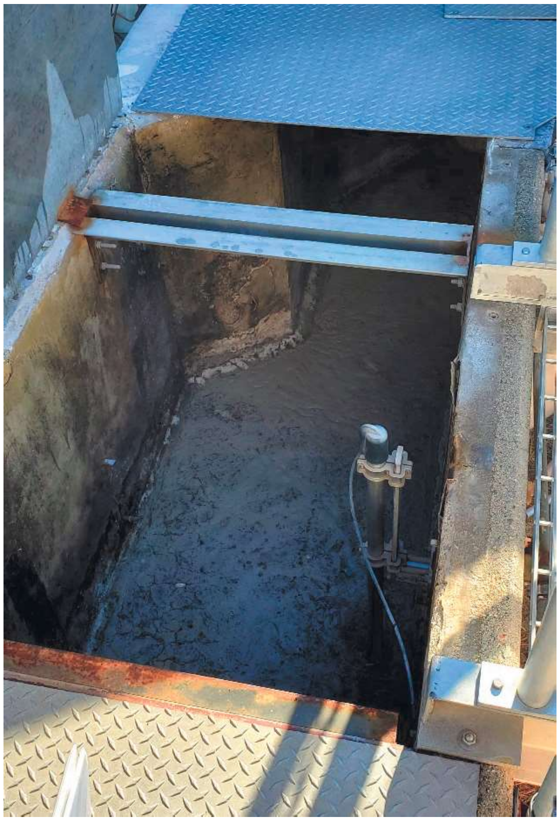

Figure 43: Influent Channel, Grease Accumulation Location

The influent collection facilities, the headworks screen, grit chamber and conveyance pipeline from the grit chamber to the aeration basins must all be sized to accommodate the peak expected influent flows. In this case, with the two pump stations, it should be assumed the pump stations are pumping at the maximum rate, and the peak expected flows in the gravity siphon main added to the total. Because the pump stations are capable of pumping well in excess of the peak hourly flow, the actual peaks into the plant could be a fair amount higher than the maximum flow received over an hour, though the peaks may be in short duration. Capacity of the existing influent structures is sufficient for the historical flows received at the facility.

4.5.1.2.1 Structural

The concrete influent channels show signs of concrete degradation on both the inside and outside concrete surfaces. The exterior degradation (Figure 42) may be from freeze-thaw cycles, whereas the interior concrete degradation is likely from exposure to hydrogen sulfide and other compounds in the wastewater, along with general erosion from the flow of wastewater and debris across the concrete surfaces.

The influent structure includes several different levels of grating, sidewalk and platforms. The grating is intended to be removed/opened for access. Once open there is little room to maneuver around the channels with open hatches. The area can be a tripping, slipping and fall hazard area just by the multiple elevations of the different areas and slippery steel diamond plate covers over the channels. Add snow cover over the area, and access can be more awkward and unsafe to navigate. A few areas over slide gate frames cast into the channel walls are wide enough to get a foot partially through, presenting further hazards in this area. Not much can be done to easily improve upon the access by adding handrails, etc. that would just be in the way of performing maintenance with the hatches opened. The evolution of the facility modifications created a complex area that warrants particular care and attention when operations staff are working in this area.

4.5.1.3 Recommendations

Concrete: Preserving and repairing the degraded concrete to slow the rate of decay will protect the reinforcement rebar from exposure and subsequent corrosion. As the concrete spalling extends and reaches depths of ½-inch plans should be made for isolating, cleaning, and application of a cementitious and/or epoxy, polyurethane or polyurea coating to extend the life of the concrete. The later synthetic coatings tend to require very dry and well-prepared surfaces whereas sprayed or troweled on cementitious and grout repair materials do not require completely dry surfaces for effective bonding and curing.

FOG Management: One option to assist with the FOG management is incorporation of a mixer or chopper pump assembly into the influent channel. The mixer could operate periodically to stir, chop, and reincorporate the FOG into the flow to allow the material to pass downstream. Possible applications include a permanent installation and timer operation, or a portable system periodically placed in the channel and manually operated to mix, chop, and scour the influent channel, in place of the manual hosing and flushing. Example system would include a Vaughan conditioning pump specifically configured for this purpose in lift stations, tanks, or channels.

The recommended FOG management project has been identified in Section 6.2 to be completed in 2026.

4.5.2 Headworks Screening Facility

4.5.2.1 Facility Purpose and Design Criteria

The purpose of the headworks screening facility is removal of non-biodegradable debris, rags, and garbage from the influent wastewater to prevent this material from plugging downstream equipment, accumulating in process tanks, and passing through the treatment system into the effluent discharged from the facility.

Chapter 60 of the Recommended Standards for Wastewater Facilities include the following applicable criteria:

- Where a single mechanically cleaned screen is used, an auxiliary, manually cleaned screen shall be provided. Where two or more mechanically cleaned screens are used, the design shall provide for taking any unit out of service without sacrificing the capability to handle the design peak instantaneous flows.
- Fine screens as discussed here have openings of approximately 1/16 inch (2 mm). The amount of material removed by fine screens is dependent on the waste stream being treated and screen opening size.
- A minimum of two fine screens shall be provided, each unit being capable of independent operation. Capacity shall be provided to treat design peak instantaneous flow with one unit out of service.
- Fine screens shall be preceded by a coarse bar screening device. Fine screens shall be protected from freezing and located to facilitate maintenance.
- Mechanically cleaned screen channels shall be protected by guard railings and deck gratings. Consideration should also be given to temporary access arrangements to facilitate maintenance and repair.
- Fresh air shall be forced into enclosed screening device areas or into open pits more than 4 feet (1.2 m) deep. Dampers should not be used on exhaust or fresh air ducts and fine screens or other obstructions should be avoided to prevent clogging. Where continuous ventilation is required at least 12 complete air changes per hour shall be provided. Where continuous ventilation would cause excessive heat loss, intermittent ventilation of at least 30 complete air changes per hour shall be provided when personnel enter the area. The air change requirements shall be based on 100 percent fresh air.

The preferred wastewater screening arrangement is for the automatic screening equipment to have 100% peak flow capacity. An acceptable alternate arrangement is to provide a bypass channel with a manual screen for when the automatic screen is out of service, such that the remaining channel and screen can still pass 100% peak flow.

The manually cleaned bar screen use is intended to be short term to minimize the passing of debris into the downstream processes until the automatic screen is returned to service. The manual screen has much larger openings than automatic screens to prevent screen blinding between manual cleanings. This allows significantly larger quantities of debris to pass the screening operation to potentially impact downstream processes. Thus, the duration of any use or bypass through the bar screen should be very limited.

Lastly, the downstream processes may dictate use of only mechanically cleaned screens with 100% redundancy, such is the case with downstream membrane or tertiary filtration processes where debris would damage or plug the equipment and bypass of debris for any duration is not acceptable. This is not the case with the existing IVGID WRRF.

Any manual cleaning of debris from equipment in the headworks facility is a laborious process and exposes operations staff to multiple safety hazards including exposure to a host of pathogens in wastewater and risk of puncture from hypodermic needles and other debris. Best design and operations practices include provisions to minimize opportunity for operations staff to be in contact with the debris screened from facility influent.

4.5.2.2 Process Description, Condition and Capacity

The existing screening facility currently includes a single automatic fine screen, Parkson Rotostrainer, accompanied by a manual coarse bar screen to bypass flow while performing maintenance on automatic equipment. The 10-State Standards specify preceding a fine screen with a larger screen for purposes of both minimizing potential to suddenly blind off the screen in the case of load of debris entering the plant such that the automatic cleaning mechanism could not keep up and pass sufficient flow and/or for the purpose of protecting the fine screen from larger debris that might make it to the fine screen and damage the equipment. This arrangement with a coarse screen ahead of a fine screen is not in place at the IVGID WRRF but, to date, the single screen has not had issues screening the incoming flows and it has not been damaged by any larger debris. The upstream sewer lift stations may actually serve to either break up or prevent passage of larger debris that could damage a screen.

Preceding the fine screen with a manual coarse screen would require frequent manual cleaning. Thus, adding a coarser screen would mean an additional mechanical screen in front of the manual screen to meet this criterion.

If or when the automatic screen is out of service the debris removal efficiency is reduced, thereby, increasing negative impacts to downstream processes and requiring manual cleaning of a manual bar screen and operator exposure to the hazardous debris.

The lack of redundancy poses a risk to downstream equipment and an operations challenge for any period with the automatic screen out of service. Further, when bypass is required, the downstream processes accumulate debris requiring manual effort to clean equipment and remove the debris, exposing operations staff to the host of safety issues handling such debris. These efforts to collect and dispose of bypassed debris and effort of repairing equipment come with a cost and associated safety risks, where a second, redundant screen would eliminate the same. The automatic screen is installed in a small, enclosed room, making removing the screen for maintenance difficult. Figure 44 and Figure 45 illustrate the automatic fine screen, and Figure 46 illustrates the coarse screen in the bypass flow channel.

Figure 44: Headworks, Rotary Drum Screen (inlet/outlet)

Figure 45: Headworks, Rotary Drum Screen

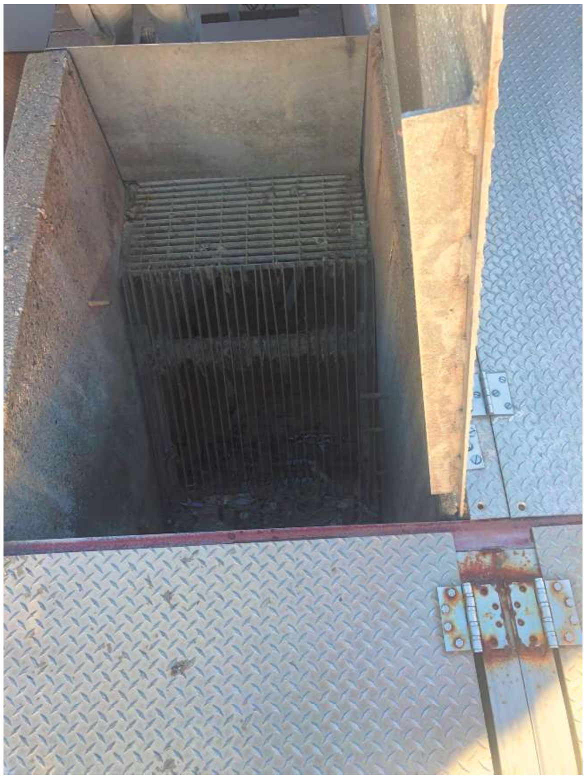

Figure 46: Headworks, Manual Bar Screen (Bypass)

The size of the room would also make it exceptionally difficult or impossible to install a second screen of the same style in the same area. Otherwise, access to the current screen is acceptable and safe with appropriate handrails on the upper level.

Table 64: Headworks Fine Screen

Equipment Name	Make/Model	Flow Capacity	Screen Opening Size	HP	Volts	Phase
Headworks Fine Screen	Parkson; Rotostrainer		2.5 mm (max)	0.75	460 V	3P
Headworks Bypass Screen	NA	NA	1”	NA	NA	NA

4.5.2.2.1 HVAC

Ventilation provisions in the combined screen and grit removal room includes adequate ventilation and odor control provisions for the air exhausted from the room. There are three ventilation ducts which draw air and gasses out of the headworks building for proper air exchange.

Gas monitors (Figure 47) for hydrogen sulfide (floor level), oxygen concentration (eye level) and hydrocarbons (ceiling level) are included in the headworks/grit room for purposes of alerting operations staff of hazardous air conditions. Operators reported a past event of a flow of gas or oil reaching the plant and filling this space with potentially volatile and explosive gas. This event illustrates the exact purpose of the gas monitoring system and the need for adequate ventilation and monitoring equipment to keep operators and the minimize the potential for ignition or explosion of accumulated volatile gases. It should be noted that the gas monitoring system was recently replaced in December of 2023.

Figure 47: Hazardous Gas Monitor System Sensors (H_2S & Low Oxygen)

4.5.2.3 Recommendations

Fine Screen: Addition of a second fine screen may not be possible in the space around this corner of the building. To mitigate risk of having the screen out of service for more than a few hours or days, stocking key spare parts for this machine would be a prudent approach. Thus, it is not recommended to pursue a major project to add a second mechanical screen of the same style as the existing screen.

One option for future consideration would be installation of an automatic screen in the location of the existing manual bar screen. Various styles of automatic chain and rake or step screens are available to install into a rectangular channel as illustrated in the image below taken from the Huber Technology website. Any such screen would need to be protected from freezing for the collected debris and the wash water spray system. Thus, in addition to installation of the screen and an additional debris washer compactor, some form of enclosure would be needed to protect the system from below freezing temperatures.

As identified in Section 6.2 it is recommended that a basis of design report for the second screen be completed in 2026, with a target to complete the second screen installation in 2027.

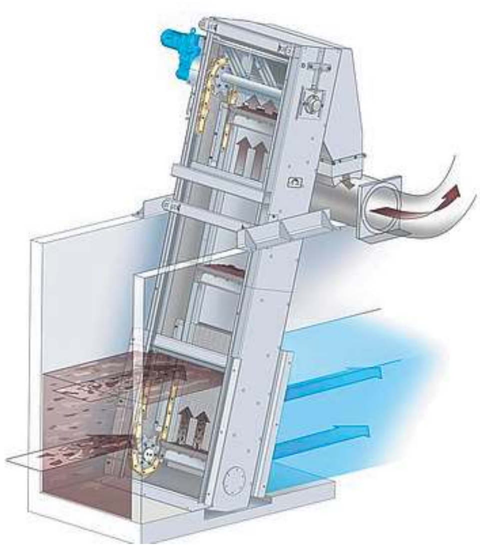

Figure 48: Example Screen
(Retrofit to Existing Bar Screen Channel)

Figure 49: Example Screen
(Debris Washer/Compactor)

4.5.3 Headworks Grit Removal

4.5.3.1 Facility Purpose and Design Criteria

The grit removal process captures sand, gravel, glass, and heavier debris before it enters downstream tanks, pipes, and process equipment. Accumulation of grit in tanks and pipelines eventually reduces the functional capacity, while pumping of grit through pumps and process equipment significantly increases the wear on the equipment. Removal of grit from downstream tanks requires the tank to be empty, and the only practical way to remove it may be with hand shovels and hours of manual labor. Where process tanks cannot be readily removed from service for extended periods of time, the grit will just accumulate and eventually influence the treatment capacity of the unit process. Thus, it is imperative grit removal facilities are included and effective for a mechanical treatment facility.

Chapter 60 of the Recommended Standards for Wastewater Facilities include the following applicable criteria:

Plants treating waste from combined sewers should have at least two mechanically cleaned grit removal units, with provisions for bypassing. A single manually cleaned or mechanically cleaned grit chamber with bypass is acceptable for small wastewater treatment plants serving separate sanitary sewer systems. Minimum facilities for larger plants serving separate sanitary sewers should be at least one mechanically cleaned unit with a bypass. Facilities other than channel-type shall be provided with adequate and flexible controls for velocity and/or air supply devices and with grit collection and removal equipment. Aerated grit chambers should have air rates adjustable in the range of 3 to 8 cubic feet per minute per foot [4.7 L/(m·s) to 12.4 L/(m·s)] of *tank length. Detention time in the tank should be in the range of 3 to 5 minutes at design peak hourly flows.*

- The design effectiveness of the grit removal system shall be commensurate with the requirements of the subsequent process units.
- Fresh air shall be introduced continuously at a rate of at least 12 air changes per hour, or intermittently at a rate of at least 30 air changes per hour.
- Adequate stairway access to above or below grade facilities shall be provided.
- All aerated grit removal facilities should be provided with adequate control devices to regulate air supply and agitation.
- The need for grit washing should be determined by the method of grit handling and final disposal.
- Impervious, non-slip, working surfaces with adequate drainage shall be provided for grit handling areas. Grit transportation facilities shall be provided with protection against freezing and loss of material.

4.5.3.2 Process Description, Condition and Capacity

The grit removal facility includes one aerated grit chamber. Influent that passes through the microscreen flows into an aerated, sloped bottom grit chamber. Air is added and controlled to gently roll the influent to settle out the grit. An air lift system, on timers, conveys the grit from the bottom of the grit chamber to a settling hopper, the separated grit is conveyed up an incline auger then into a cart destined for disposal to a secured landfill. The compressed air for the air lift pump function is provided from a side stream from the blower also providing air to the sludge holding tanks. Thus, there are no other grit pumps in the system to remove grit from the grit chamber. The air lift system pulls grit from the chamber hopper solely through the air lift system.

Figure 50: Grit Chamber & Outlet to Aeration Basins

4.5.3.2.1 Structural

The grit chamber shows some concrete degradation at the waterline, but the remainder of the tank was not visible below the water level to assess any more significant concrete concerns. Monitoring of the concrete condition is recommended to look for indications that it is time to complete restoration work to protect the reinforcement from corrosion that would result from inadequate concrete cover over and around the steel reinforcement.

4.5.3.3 Recommendations

As noted, on at least an annual basis the grit chamber should be bypassed and drained to monitor the concrete for more significant concrete degradation and plan for repairs if erosion or spalling exceeds ½" or any signs of rebar exposure or rust stains on the walls are present.

4.5.4 Odor Control Systems

4.5.4.1 Facility Purpose and Design Criteria

The odor control facility provides scrubbing of odors collected predominantly from the headworks screen and grit room and the solids handling room before exhausting the ventilation air to atmosphere. The specific criteria for odor control systems are specific to the type of equipment or facilities implemented. Biofiltration, versus chemical scrubbing, versus packed media towers all have different criteria associated with sizing the equipment and accommodating the target air flow.

4.5.4.2 Process Description, Condition, and Capacity

The existing deep bed scrubber (DBS) adsorption system manufactured by Purafil Filtration Group replaced a previous wet scrubber odor control facility. The existing system uses no chemical and relies on manufactured and proprietary media for adsorption of odors prior to discharge to atmosphere. Aside from periodic replacement of the media, the system requires little maintenance, is much safer to operate than a chemical scrubber, and performs for the purpose it serves. The system is in good condition and not specific deficiencies are apparent with the system overall. Further, the media was replaced in July 2023

Figure 51: Odor Scrubber

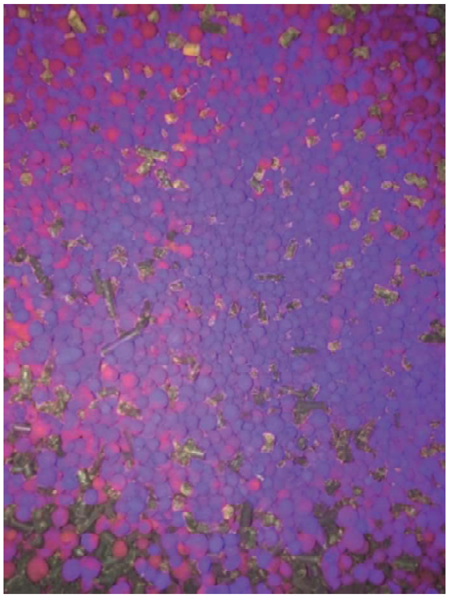

Figure 52: Odor Scrubber Media

4.5.5 Aeration Basins

4.5.5.1 Facility Purpose and Design Criteria

Chapter 90 of the Recommended Standards for Wastewater Facilities include the following applicable criteria:

- Ordinarily, liquid depths should not be less than 10 feet or more than 30 feet except in special design cases.
- Total aeration tank volume shall be divided among two or more units, capable of independent operation.
- Aeration equipment shall be capable of maintaining a minimum of 2.0 mg/L of dissolved oxygen in the mixed liquor at all times and provide thorough mixing of the mixed liquor. In the absence of experimentally determined values, the design oxygen requirements for all activated sludge processes shall be 1.1 lb O2/lb design peak hourly BOD5.
- Normal air requirements for all activated sludge processes except extended aeration shall be considered to be 1,500 cubic feet at standard conditions of pressure, temperature, and humidity per pound of BOD5 tank loading (94 m3/kg BOD5). For the extended aeration process the value shall be 2,050 cubic feet per pound of BOD5 (128 m3/kg of BOD5).
- Diffuser systems shall be capable of providing for 200 percent of the designed average day oxygen demand.
- Aeration Tank Organic Loading – 40 lb BOD5/d/1000ft3
- F/M Ratio – 0.2-0.6 lb BOD5/d/(lb MLSS)
- Maintain a minimum of 2.0 mg/L of dissolved oxygen in the mixed liquor at all times throughout the basins.
- Mechanisms and associated structure shall be protected from freezing.
- Meet maximum oxygen demand and maintain process performance with the largest unit out of service.
- Provide for varying the amount of oxygen transferred in proportion to the load demand on the plant.
- Provide that motors, gear housing, bearings, grease fitting, etc. be easily accessible and protected from inundation and spray as necessary for proper functioning of the unit.

4.5.5.2 Process Description, Condition, and Capacity

The biological treatment process used in the plant is activated sludge aeration designed to meet secondary treatment standards for BOD5 and TSS removal. The purpose of the activated sludge in the aeration basins is to remove biochemical oxygen demand (BOD) and to break down certain organic materials such as Phosphorous and Ammonia Nitrogen in the wastewater. The biological activity is assisted by maintaining a controlled environment of concentrated activated sludge (mixed liquor) and supplying dissolved oxygen (DO) by injecting air. There are six available aeration basins, two to four basins are normally run in parallel and in plug flow mode. The basins receive plant influent from the headworks facility and return activated sludge (RAS) via the clarifier underflow. The influent and the RAS mix together at the head of the “center channel” between the north and south aeration basins and flow through the drop in slide gates of the desired aeration basins.

Figure 53: North Aeration Basins, Under Aeration (N-2, N-3)

Figure 54: North Aeration Basin & “Jet Cluster” Diffuser, Empty (N-1)

Figure 55: Aeration Bain, Anoxic Zone & Mixer (Basin S-2)

The center aeration basins (N-2, S-2) include a baffle wall in the inlet corner of the basins to separate this area of the reactor from the aeration system. The isolated section, along with a submersible mixer, provides an anoxic zone ahead of the aerobic zone, effectively reducing the volume of the tank under aeration and the total time under aeration. The baffle acts to reduce residence time and inhibit nitrification.

4.5.5.2.1 Aeration

Rotary lobe positive displacement blowers provide oxygen to the jet aeration diffusers in the bottom of each aeration basin. The aeration system controls are part of the Aerzon blower control system installed in year 2020. The AERtronic control system incorporates dissolved oxygen and blower discharge pressure control to modulate the blower operating speed and to distribute air among the aeration basins in operation through mass air flow meters and automated butterfly valves on each respective tank aeration header.

Aeration control to each aeration basin in operation is based on the dissolved oxygen in the aeration basin, as measured through dissolved oxygen probes (YSI/Xylem) suspended in the aeration basins. The air input is modulated through both the variable speed drives on the blowers and modulating flow control valves on the individual air supply headers to each aeration basin. Each aeration basin header includes a mass flow meter to measure the air input and throttle or open the control valve according to the target dissolved oxygen concentration in the respective aeration basin. The overall blower speed is modulated based upon the discharge pressure on the blower(s) in operation. As an individual aeration basin modulating valve throttles to reduce the air input into the basin based on the D.O. probe, the pressure at the blower increases, allowing the blower speed to slow down to meet the reduced air flow demand. The rotary lobe positive displacement blowers are shown in Figure 56 and Figure 57, and the blower capacities are show in Table 65.

Figure 56: Aeration Basin Blowers and Blower

Figure 57: Aeration Basin Blowers/Enclosures

Table 65: Aeration Blowers

Equipment Name	Make/Model	Capacity	Max Discharge Pressure	HP	Volts	Phase
Blower 1	Aerzen	1700 CFM	6.5	125	480	3
Blower 2	Aerzen	1700 CFM	6.5	125	480	3
Blower 3	Aerzen	1700 CFM	6.5	125	480	3
Blower 4	Kaeser	1236 CFM	7	60	480	3

Figure 58: Dissolved Oxygen Monitors (3) and Mass Air Flow Meter Displays (6)

The target dissolved oxygen concentration is generally around 1.9 to 2.0 mg/L though at times when the oxygen demand is higher during warmer process water temperatures in the summer months, the blowers and aeration system have trouble maintaining 2.0 mg/l. The surface of the aeration basins become very violent, and the water surface nearly reaches the above walkways, as softball size bubbles become prevalent as the blower discharge air flow rate increases and the open control valves open farther.

The larger bubbles are not as efficient in oxygen transfer as compared to many small bubbles with greater aggregate surface area for transfer to occur into the water. Generally, the violent surface agitation and aeration is an indication something is not right in the process, as the aeration pattern should not result in significant agitation at the surface in any condition. A few items possible contributors to this condition are as follows:

- Influent organic loading peaks and inadequate volume under aeration to dilute the load to minimize the increased oxygen demand: Significant increases in the influent BOD concentration in the basins in service may exceed the aeration capacity of the respective basin, not necessarily the system as a whole. If only a couple aeration basins are in service and the organic load into the plant increases sharply during the day, the total volume under aeration may not be sufficient to distribute the organic load and the corresponding oxygen demand. The aeration system response to push more air into the basins may be too much air for the circulation pumps and jet aeration clusters to effectively diffuse into the normal fine bubble aeration pattern.
- The larger bubbles are indicative of a Non-Ideal combination of mixing flow and air input in the jet aeration system. If the air flow is excessive, the pumped flow cannot effectively diffuse the air through the jet nozzles to create the fine bubble aeration pattern.
- The DO drops resulting in the air control valves opening farther to let more air into the basin. The blower senses the drop in pressure from the valve opening and increase in speed. At some breakpoint, the air flow exceeds the capacity of the

recirculation pumps to provide adequate flow to produce a fine bubble aeration pattern. The large bubbles do not transfer oxygen as efficiently, thereby allowing the dissolved oxygen to continue dropping. The valves continue opening and the blowers continue ramping without improving the oxygen transfer and resulting in the violent water surface pattern and splashing. In summary, there is too little recirculation flow to match the air flow provided from the blowers.

- In evaluating the existing jet aeration, the original air flow and recirculation rates were not readily located. Instead, a conceptual jet aeration design was generated for the actual flow rate and organic strength observed in the peak loading summer months. In short, the flow rate necessary with a modern jet aeration system to meet the peak loading conditions is closer to 4,000 gpm rather than the stated flow capacity of the existing recirculation pumps at approximately 2,900 gpm. This further confirms the suspicion the existing jet aeration system is not able to provide the correct combination of air flow and recirculation flow to maintain the intended fine to medium bubble aeration pattern.
- In some jet aeration systems, the pump can become air locked, resulting in insufficient flow of water, larger bubbles and a similar violent surface aeration and splashing pattern.
- Pump Impeller Wear: Thirty years of use may have contributed to some impeller wear and loss of pumping capacity in the recirculation pumps. If the actual recirculation flow is significantly reduced, this would limit the aeration volume that could be effectively diffused to maintain a fine bubble aeration pattern. However, the District indicated all of the pumps were rebuilt in approximately 2017, including replacement of the impellers. Presumably, any excessive wear of impellers would have been addressed with this rebuild, if observed. Impeller wear alone would not explain the shortage of flow resulting in the aeration pattern. Estimates of the needed flow rate exceed the rated flow rate of the existing pumps.
- If there was anything substantially wrong with the jet clusters, the issue would likely be more common. The jet nozzles appear in good condition and appear to be replaceable apart from the overall cluster assembly.

Aeration piping from the blowers and into the basins is a combination of fiberglass from the year 1991 project and stainless-steel pipe from the aeration system improvements project in year 2020. The in-basin piping is fiberglass, but the air pipe transitions to stainless steel above the water line. Part of the main header on the south end of the aeration basins is fiberglass before transitioning to stainless steel back to the blower building. The stainless-steel air piping is in like new condition, and the fiberglass is not showing obvious signs of delamination from a somewhat remote observation done from outside of the tanks.

The aeration piping generates a loud vibration/resonance sound in the main aeration header between the blowers and the aeration basins. The exact cause and cure of this noise has been illusive and remains a significant environmental nuisance at the facility where other sources of noise outside the buildings are mostly absent.

The aeration basins have adequate capacity and flexibility in the number of tanks in operation to maintain appropriate loading rates and aeration supply to meet the noted typical design criteria. The summer swings in flow at generally the same high BOD and TSS concentration may warrant placing additional aeration basins in service to maintain the same operating metrics for the operating food to mass concentrations but there is ample flexibility in the system to readily adjust for the seasonal variations in loading and performance. However, IVGID has indicated they prefer to keep two of the six basins available for emergency storage in the event there are issues with the effluent storage, pump station or export pipeline. The District can stop discharging and store roughly 200,000 gallons of influent in each of the two of the basins to buy time getting the effluent export system back online. Therefore, until the new effluent storage tank currently in design is constructed, use of the other two basins to potentially assist with spreading the peak loads and reducing the aeration system challenges is not an option.

4.5.5.2.2 Process Control and Instrumentation

In addition to the dissolved oxygen meters in the aeration basins, the following additional instrumentation is in place at the facility:

Total Suspended Solids (TSS) Meters: TSS meters (YSI/Xylem) are installed in the headworks influent channel, each aeration basin, on the return activated sludge recycle pipeline, on the waste activated sludge (WAS) pipeline and the effluent collection channel downstream of the clarifier. At the time of a site visit in November of 2023 the corresponding values at each of these meters were as follows:

Table 66: Observed Operational Parameters (TSS)

TSS Meter Location	Observed Value
Aeration Basin	1,836 mg/L
Return Activated Pipeline	10,590 mg/L (1.05%)
Waste Activated Sludge Pipeline	5,059 mg/L (0.51%)
Clarifier Effluent Channel	11.5 mg/L

Note, the WAS and RAS pumps pull clarifier underflow water from the same manifold and these TSS numbers should normally be the same. The WAS pumps were likely not operating at the time this reading was recorded. The concentration of the mixed liquor in the aeration basin is used to calculate the total mass of biosolids under aeration. Dividing this mass by the total daily pounds of solids wasted in the WAS pumping provides the total solids retention time (SRT) of the facility at that point in time. Operators noted a low SRT of 3-4 days is targeted to minimize the nitrification in the system. Based on the TKN numbers in the effluent, it appears the low SRT does not completely inhibit the nitrification in the warmer months of June to November.

4.5.5.2.3 Mixing

The mixing and diffusing energy is provided by large recirculation pumps recycling mixed liquor from each individual tank back to the respective “jet cluster” diffuser assembly where the return flow mixes with the air from the blowers. There are a total of six jet aeration pumps also referred to as sludge recirculation pumps. Two sets of three pumps are housed in each of the north and south pump buildings. The pumps serve their respective aeration basins (north and south) by pulling mixed liquor out of the basins and recirculating it through the aeration basins continuously. The general specifications for each pump are outlined in Table 67. To provide the necessary mixing energy, these pumps are operated at a continuous speed. The dissolved oxygen concentration is thus controlled only through variation of the air flow into each tank.

Table 67: Jet/Recirculation Pumps

Pumps	Make/ Model	Capacity	Max Head	HP	Speed	Volts	Phase
North Recirc. Pumps (3)	Fairbanks-Morse T40-B5445	2,930	24.0	25	580 rpm	480	3
South Recirc. Pumps (3)	Fairbanks-Morse T40-B5445	2,930	24.0	25	580 rpm	480	3

The facility has ample capacity to allow one or more basins to be removed from service. It is, therefore, unlikely losing one of the jet mixing pumps would cause a substantial upset in the overall process or compromise the ability of the facility to adequately process the wastewater. The actual pumping performance of the pumps is not easy to measure as there are no flow meters on the respective pump discharge pipes. Visual inspection of the pump impellers and general observation of the pump operation are the only means to monitor the performance and condition of these pumps.

The recirculation piping between the recirc pumps and the jet cluster diffusers is showing signs of surface corrosion. It is not known if the pipe was ductile iron or steel and whether it had a black coal tar epoxy coating on the exterior and any form of either cement mortar or other pipelining material. The exterior of the visible recirc piping is showing signs of rust from failure of whatever coating may have been applied to this piping in the first place. The piping should be monitored annually for signs of deeper corrosion and material flaking, pitting or pinholes that would be a definitive sign of the time to replace this process piping.

Similarly, the aeration jet clusters are a combination of a steel “pot” with either steel or fiberglass/composite jet nozzles. Some minor signs of rusting are present on the outside of the visible jet clusters but for 30 years of service the coating on the jet cluster appears to be holding up well.

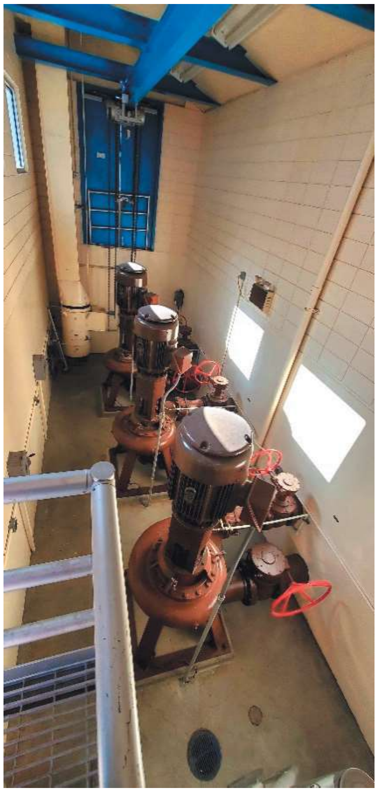

Figure 59: South Aeration Basin Jet/Recirculation Pumps

4.5.5.2.4 Structural

The aeration basin concrete walls have a significant amount of surface degradation resulting in exposed aggregate throughout the basins and below the normal operating water level.

A visual and non-destructive assessment was performed of exterior structures. The plant was operating so a full assessment of structures or tanks filled with liquid was not feasible. Several aeration basins were not in use used and were visually observed. The aeration basins have a lack of cement paste matrix in areas as the aggregate is exposed below the normal liquid level as seen in the figures below. Exposed reinforcement, large cracks or delaminations or spalling were not observed; however, some cracks were observed. The minor cracks in the basin walls were unable to be accessed for measurement and the concrete was not sounded with a hammer. As the surface of the concrete continues to deteriorate or crack, concrete spalling and exposed and rusting rebar could initiate. Once the rebar begins to rust, the concrete near the rebar will continue to spall leading to more exposed rebar.

Figure 60: Aeration Basin Concrete Erosion

Figure 61: Aeration Basin Concrete

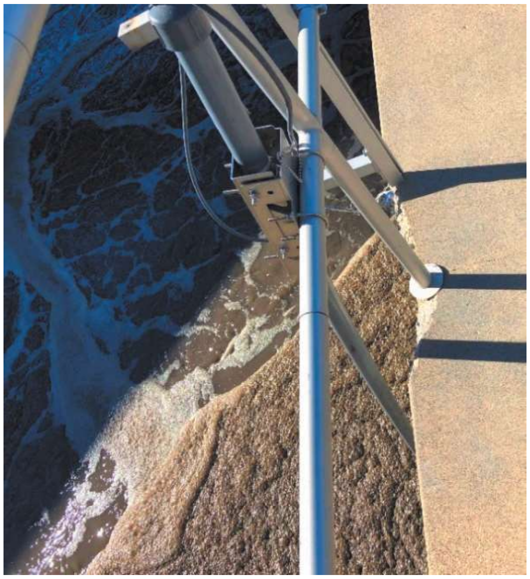

Figure 62: Walkway Concrete Damage

Figure 63: Walkway Concrete Damage

4.5.5.2.5 Biological Process Loading/Capacity

Table 68 provides a summary of key biological treatment process operation parameters as compared to the typical criteria stated at the beginning of this section.

Table 68: Biological Process Compared to Typical Design Criteria

Ordinarily, liquid depths should not be less than 10 feet or more than 30 feet except in special design cases.	OK
Total aeration tank volume shall be divided among two or more units, capable of independent operation.	OK
Aeration equipment shall be capable of maintaining a minimum of 2.0 mg/L of dissolved oxygen in the mixed liquor at all times and provide thorough mixing of the mixed liquor. In the absence of experimentally determined values, the design oxygen requirements for all activated sludge processes shall be 1.1 lb O_2/lb design peak hourly BOD_5	Not met. As noted, the facility aeration system does not keep up with the peak organic loading, and the aeration pattern is indicative of aeration and/or mixing system deficiencies in the jet aeration system. The exact reason for the apparent deficiency warrants further investigation.
Aeration Tank Organic Loading – 40 lb BOD_5/d/1000ft^3	At July flows of 1.1 MGD and Influent BOD_5 of 500 mg/L the loading is 43 lb/BOD_5/1000 ft^3; Loading appears excessive with just four basins online. The actual flows into the plant are approximately 50% of design capacity of 2.25 MGD but the organic loading is typically at least 200% of the design values.
F/M Ratio – 0.2-0.6 lb BOD_5/d/(lb MLSS)	At noted July flows and influent loading and MLSS of 1,800 mg/L the F:M=0.38
Solids Retention Time (SRT)	Assuming 4 basins online; WAS volume of 25,000 gpd and WAS concentration of 7,500 mg/L (0.75%) the SRT is approximately 7.7 days. With 3 basins online, the SRT would be 5.8 days. Neither value is grossly out of range for this type of plant and secondary treatment objectives.

4.5.5.2.6 Blower Building/Room

The blower building is adequately isolated from the other buildings and rooms, and the sound attenuation enclosures sufficiently dampen the sound of the notoriously loud rotary lobe blowers. In hotter months, the room does get overly warm and could use additional ventilation to help remove the hot air from the room. IVGID has installed some security screens allowing the roll-up doors to remain open approximately 50%. This helps but additional air movement is needed to maintain lower temperatures in this room. More detailed evaluation would be required to assess if outside air cooling is adequate or if mechanical cooling would be required.

Generator Room: The emergency generator exercises periodically. The discharge air from the generator cooling system discharges to the adjacent storage room and can interfere with the fire protection system in this room.

4.5.5.3 Recommendations

4.5.5.3.1 Structural

Reinforcement corrosion can adversely affect the service life and durability of a concrete structure. As the reinforcement corrodes, it can lead to spalling, cracking, and leaks in the confinement structure if present. Especially in wastewater applications, the damaged layer of concrete (cracking and spalling) allows deleterious elements to penetrate the concrete and damage the integrity of the structure. Corrosion of the reinforcement is the most common cause of deterioration of concrete tanks and basins. The steel reinforcement corrodes in the presence of oxygen and moisture. In the treatment facilities, chlorides in the contained water enter through the concrete and damage the passivating layer. Hydrogen Sulfide present in dissolved gases in the liquid can also rapidly deteriorate concrete.

A damaged layer of concrete accelerates the deterioration process as corrosive elements easily find a way into the concrete. Abrasion, freeze-thaw, and chemical attack are the main mechanisms to prevent for that reduce durability in your concrete assuming the original construction was relatively crack-free, dense with low-permeability and resistance to freeze-thaw damage. The quality of the concrete is essential in protecting the reinforcement from an aggressive environment.

Once concrete is badly damaged, it may not be economical to repair; so, timely preventative maintenance of reinforced concrete tanks and basins is always prudent. Usually, aging, or damaged concrete is coated with a sealer, polymer coating or latex-modified coating; however, because traditional materials and construction techniques cannot adequately address the aggressive environment and corrosive agents. Once the tank is badly damaged, it is not economical to repair; so, timely maintenance of reinforced concrete tanks and basins is always prudent. Spalls or deterioration larger than 6” diameter x 1" deep are poor condition and would affect the structural rating and need a coating. The prominent signs are if there are any active corrosion or rust staining. It is recommended to annually monitor the concrete deterioration specifically for increasing decomposition in the surface of ½” or more and any evidence of rust staining. At that time and evaluation of coating and repair options should be evaluated and pursued.

As identified in Section 6.2, it is recommended that lining the aeration basin be completed in 2027.

4.5.5.3.2 Aeration Equipment

The aeration piping and jet cluster aeration assemblies are showing some signs of aging. Close inspection of the piping and diffusers from inside the tank was not completed as part of this Plan, and the equipment may have many years of life remaining. It is recommended to visually inspect the aeration piping and supports annually for signs of deeper corrosion, flaking, pin holes, etc. Similarly, inspect the jet cluster diffusers for signs of wear, erosion to the jet nozzles and corrosion inside the cluster assemblies.

The Jet Cluster aeration diffusers, recirculation pumps and blowers do not appear to be appropriately paired or are otherwise unable to meet the oxygen demands of the incoming loads. Further investigation is needed to determine the cause, or the jet aeration diffuser system needs replaced with a modern system.

The aeration tank organic loading appears to be excessive with just four basins online. The organic loading to flow ratio suggests that utilizing the additional basins will bring organic loading more in line with the typical design values.

The recommended aeration system improvements project has been identified in Section 6.2 to be completed in 2025.

4.5.6 Main Pump Building and Return Activated Sludge (RAS) Pumps & Equipment

4.5.6.1 Facility Purpose and Design Criteria

The return activated sludge (RAS) pumps return biologically active micro-organisms to the aeration basins for further wastewater treatment. After separation from the treated water within the secondary clarifier, a portion of the sludge is conveyed back to the aeration basins as return activated sludge. The rate of return flow through the RAS pumps impacts the clarifier operation, the MLSS concentration, the solids retention time (SRT) and the food to mass ratio (F/M). Thus, the RAS recirculation rate, combined with the sludge wasting rate and the dissolved oxygen set point in the aeration basins are the key process control parameters available for optimization of the activated sludge process at this facility.

Chapter 90 of the Recommended Standards for Wastewater Facilities include the following applicable criteria:

- Rates. Sludge recirculation from the secondary settling basin to the aeration basin shall be variable within 15 (minimum) to 100 (maximum) percent of the average design flow.
- Pumps should have a least 3-inch suction and discharge openings.
- Discharge piping should be at least 4 inches in diameter and should be designed to maintain a velocity of no less than 2 feet per second when facilities are operating at normal return sludge rates.

4.5.6.2 Process Description, Condition and Capacity

The plant has two RAS pumps that discharge through a mud valve into the raw wastewater influent channel between the aeration basins and mixes with influent ahead of the aeration basins. The RAS pumps are located on the middle floor or the mechanical building and are powered by variable frequency drives (VFDs). The VFDs are controlled by set points based on the percent of influent flow from the plant control system. The pumps can also be run manually if needed. The specifications of each Return Activated Sludge Pump are listed below.

A chlorine injection line is tapped into the RAS header in the RAS pump room, and for use in managing filamentous bacteria. This is not normally in use. As noted, the RAS discharge header includes a TSS meter to monitor the concentration of the mixed liquor recycle.

Figure 64: Return Activated Sludge (RAS) Pumps

The RAS pumps were modified with a project in 1997 to replace the original pumps installed in the 1971 project with two new Wemco Hydrostat screw centrifugal pump. The return sludge pipe from the clarifiers connects into a manifold in the pump room such that either pump could recycle sludge from both clarifiers though the general operation is to allow one of the RAS pumps to work directly with the corresponding clarifier. Operating in an alternative configuration is reported to not provide for appropriate flow splitting between the two clarifiers. The pumps and equipment are in good condition. Adequate isolation valves, check valves, pressure gauges are in place along with a TSS meter on the common manifold.

Operators report the pumps can turn down very low on the VFDs but at or below 25 Hz, the pumps accumulate air and have issues with air lock. IVGID installed new air release valves in August of 2023 to help alleviate the air locking of the pumps. The general consensus is the pumps do not allow an adequate turndown to address the low flow periods during the night when the RAS flow rate is targeted to maintain a RAS flow in proportion to the incoming flows.

Operating the RAS proportional to influent flow is the recommended method. Once influent flows drop very low, the RAS rate may tend to be high relative to incoming flow. The main impact of this from a process perspective is use of more energy and pumping rate than is necessary for the biological process and clarifier operation. The effect on the biological process should be minimal. Thus, to prevent the issues with the low flow rate, the controls could be set to never allow the pumps to operate below a speed that does not cause the air lock problems. The only downside should be the use of slightly more energy from this relatively small pump at 5 horsepower.

Table 69: Return Activated Sludge (RAS) Pumps

Pumps	Make/Model	Capacity	Max Head	HP	Speed	Volts	Phase
RAS Pumps (2)	Wemco Hydrostal/ ESK-M-E2W HVP	475 - 700 GPM	9-13	5	1750	480	3

4.5.6.2.1 Structural

The new process piping for the utility water includes seismic bracing where much of the other piping in the plant does not include the lateral bracing. No other specific structural concerns were identified in the pump building, and the structure appears in excellent condition.

4.5.6.2.2 HVAC

The basement levels of the main pump building warrant adequate ventilation to prevent accumulation of any potentially hazardous gas and ensure there is adequate fresh air and oxygen. Both the intermediate building level with the RAS pumps and the lower level with the WAS pumps, plant waste pumps and utility water pumps have ductwork extended to a few feet off the floor level to force the exchange of air from these rooms. The air handling and heating unit, intake louvers and exhaust fans are located the upper level electrical and controls room. The equipment is operational and in good condition, providing suitable ventilation to the entire pump building.

4.5.6.3 Recommendations

The suction header in the RAS pump room includes a tee and blind flange on the discharge manifold for connection of a third RAS pump. A third suction pipe is stubbed out of the building and intended for future connection to a third, secondary clarifier. Installation of a third pump of smaller capacity on the available piping connections would not provide for adequate flow split from the two clarifiers. Thus, the recommendations for addressing the air lock issue with the pumps is to operate at a higher recycle rate. When the pumps are due for a major rebuild or replacement, slightly smaller pumps or even different impellers could be installed to reduce the capacity and allow greater turndown more in line with the peak flows of 1.5 MGD versus the higher original design flows of 3.0 MGD.

4.5.7 Waste Activated Sludge (WAS) Pumps

4.5.7.1 Facility Purpose and Design Criteria

Chapter 90 of the Recommended Standards for Wastewater Facilities include the following applicable criteria:

- Waste sludge control facilities should have a capacity of at least 25 percent of the design average rate of wastewater flow and function satisfactorily at rates of 0.5

percent of design average wastewater flow or a minimum of 10 gallons per minute, whichever is larger.

- Means for observing, measuring, sampling, and controlling waste activated sludge flow shall be provided.

4.5.7.2 Process Description, Condition and Capacity

The Waste Activated Sludge (WAS) Pumps convey sludge out of the bottom of the clarifier and discharges to the WAS storage tanks. The WAS pumps are operated intermittently to remove excess activated sludge in order to achieve the desired volume of sludge in the mixed liquor based on plant process needs. The plant has two WAS pumps that are located on the bottom floor of the mechanical building. The pumps are controlled by a timer that is controlled by calculated figure based on the target solids retention time for the facility. The general specifications of each pump are outlined below.

Figure 65: Waste Activated Sludge (WAS) Pumps

Table 70: Waste Activated Sludge (WAS) Pumps

Pumps	Make/Model	Capacity	Max Head	HP	Speed	Volts	Phase
WAS Pumps (2)	Hydrostal/D3K-3-DOH HVP	100 GPM	13.0	3	1170	480	3

The WAS discharge header includes flow meter and a total suspended solids meter to measure flow rates and the concentration of the waste sludge, respectively. This concentration and the total daily volume pumped provides the total amount of solids leaving the process for use in determining the total solids retention time in the treatment process.

4.5.7.3 Recommendations

None.

4.5.8 Plant Utility Water Pumps

4.5.8.1 Facility Purpose and Design Criteria

The plant utility water pumps deliver non-potable water throughout the plant. Other than having adequate capacity, redundancy and general pumping system best practices for isolation valves, check valves, gauges, and flow meters there are not specific regulatory criteria for plant water supply system.

4.5.8.2 Process Description, Condition and Capacity

The two plant water pumps are newer end suction centrifugal that replaced two horizontal split case pumps. The combined discharge pipe includes pressure gauges and a pressure regulating valve to allow regulation of the downstream pressure. These pumps have no VFDs or pressure tank to control stopping and starting or pump operating speed control. One pump or the other runs continuously to supply the continuous water needs throughout the plant. Each pump has isolation valves on the suction and discharge and check valves, all in good condition.

One of the pumps runs continuously to supply water to the point of use in the facility including spray nozzles at the headworks automatic screen, nozzles in the aeration basin mixed liquor outlet channels and spray nozzles in the clarifiers, both for management of foam. This reuse water is not used in the polymer make up water for the centrifuges in the solids handling building, where potable water was required for this equipment. The reuse water is pulled from the clarifier effluent collection channel adjacent to the main pump building and the mixed liquor distribution channel structures preceding the clarifiers. The effluent does not go through any filtration prior to using and the relatively low TSS has not caused issues with the larger nozzle openings plugging such as in the headworks screen.

Figure 66: Plant Utility Water (non-pot) Supply Pumps

Table 71: Plant Utility Water Supply Pumps

Equipment Name	Make/Model	Flow Capacity	Head Capacity	HP	Volts	Phase
#2 Plant Water Pumps (2)	Cornell 1.25Y - CC	160 gpm	275	20	460	3

4.5.8.3 Recommendations

This system was recently upgraded and does not appear to warrant any near-term modifications for performance or longevity.

4.5.9 Plant Waste Pump Station

4.5.9.1 Facility Purpose and Design Criteria

The purpose of the plant waste pump station is to collect various process flows and drain water that cannot flow by gravity into the front of the plant and needs collected and pumped. The pump station is essentially a duplex, dry well, sewer lift station needing the same general duplex redundancy, isolation and check valving, and liquid level control provisions as a typical lift station.

4.5.9.2 Process Description, Condition and Capacity

The plant waste pump station collects water from the facility bathroom, miscellaneous floor, tank, and process drains, and scum from the clarifiers and pumps the flows back to the front of the WRRF, entering the influent channel upstream of the headworks screen. The pump system was constructed with the aeration basins and pump building in 1971. The pumps and piping appear in good condition with no noted deficiencies. This pump system does not have a flow meter included on the discharge line. The total volume relative to the remainder of the influent in small and flow monitoring calculated on pumping rates (i.e., pump curves or wet well draw down tests) multiplied by the pump run times would provide a sufficient indication of the total volumes returned to the front of the plant. The duplex pump system is shown in Figure 67, and Table 72 summarizes the record capacity of this pump system.

Figure 67: Plant Waste Pump Station

Table 72: Plant Waste/Recycle Pumps

Equipment Name	Make/Model	Flow Capacity	Head	HP	Speed	Volts	Phase
1	Wemco Hydrostal 4X3 D3K-S-DOW-HVP	250 gpm	30	5	1750 rpm	460	3
2	Wemco Hydrostal 4X3 D3K-S-DOW-HVP	250 gpm	30	5	1750 rpm	460	3

4.5.9.3 Recommendations

If the pumps have not been replaced or rebuilt since original installation in 1971, it may be time for an evaluation of the pumps. The capacity of the pumps could be measured by a wet well draw down test and compared to the original pump operating point for a measurement of the current performance versus design. Otherwise, the system appears in good condition and does not appear to warrant near term improvements.

4.5.10 Secondary Clarifiers

4.5.10.1 Facility Purpose and Design Criteria

Secondary clarifiers provide settling of the MLSS to separate the solids from the effluent. The overflow from the clarifiers is effluent intended to meet BOD, TSS and applicable nutrient removal criteria prior to discharging to the disinfection process. In secondary clarification, MLSS is fed to the clarifier and the MLSS settles, forming a solids blanket with an overlying clear-water zone. Within the clear-water zone, discrete flow particles settle, resulting in a clarified effluent. In the lower zone, the sludge blanket of MLSS thickens prior to withdrawal as clarifier underflow. The microorganisms in the clarifier underflow, or return activated sludge (RAS), is returned to the aeration process. Some of the microorganisms are also removed or wasted from the system (i.e., waste activated sludge or WAS) and are pumped to the aerobic digesters for further treatment.

Chapter 70 of the Recommended Standards for Wastewater Facilities include the following applicable criteria:

- Multiple units capable of independent operation are desirable and shall be provided in all plants where design average flows exceed 100,000 gallons/day (380 m3/d).
- Minimum side water depth of 12 feet.
- Surface Overflow Rates at Design Peak Hourly Flow (gpd/ft2) – 1,200. (Based on influent flow only, excluding RAS flow)
- Peak Solids Loading Rate of 50 lb/day/ft2 (base on the design maximum day flow rate plus the design maximum return sludge rate requirement and the design MLSS under aeration.)
- Overflow Weir Loading Rate – 30,000 gpd/lineal ft at peak hourly flow.

- Full surface mechanical scum collection and removal facilities, including baffling, shall be provided for all settling tanks.
- The design shall provide for convenient and safe access to routine maintenance items such as gear boxes, scum removal mechanisms, baffles, weirs, inlet stilling baffle areas, and effluent channels.

4.5.10.2 Process Description, Condition, and Capacity

The original clarifier mechanisms installed in 1971 were recoated in 1991 and then were replaced in 1997 with all new equipment except for the density current baffles around the inside perimeter of the tank installed with the major facility improvements in 1991. The two (2) 60-foot diameter secondary clarifiers are spiral blade, center feed and center sludge collection clarifiers manufactured by WesTech.

Mixed liquor exits the aeration basins through tank perimeter overflow weirs, enters a conveyance trough and converges between the two pump buildings before dropping into one of the two center feed clarifier influent pipes. A third pipe is available for a future, third clarifier but is currently plugged. Mixed liquor enters the center energy dissipation ring before flowing downward and under the bottom of the energy dissipation ring approximately 4.5 feet below the water surface. The solids settle and the clarified water exits over the perimeter overflow weir to the effluent trough. The clarified water returns to the area between the two recirc pump buildings before converging in a channel and passing through a Parshall flume for flow measurement. Near this location, the plant utility water pumps pull water for distribution around the facility. The effluent also passes through a screen installed in the effluent channels to capture any larger debris before the effluent routes to the effluent storage tank and transfer pipeline to one of three permitted outfalls: the Wetlands Enhancement Facility in the Carson Valley, Clear Creek at Tahoe, or Schneider Ranch.

The secondary clarifiers are illustrated in Figure 68, Figure 69, and Figure 70. Table 73 includes a summary of the clarifier equipment.

Figure 68: Secondary Clarifiers

Figure 69: Secondary Clarifier

Figure 70: Secondary Clarifier Mechanism (example similar to IVGID)

Table 73: Secondary Clarifier Criteria

Equipment Name	Make/Model	Scraper Tip Speed	Dia	HP	Speed	Volts	Phase
1 & 2	WesTech COP (Spiral)	12 feet/min	60’	1	1800 rpm	460	3

The clarifiers are adequately sized to meet the noted criteria of Chapter 70 of the Ten State Standards.

Common challenges noted in clarification in colder climates can include ice problems, denitrification resulting in sludge rising with the nitrogen bubbles, temperature currents and sludge bulking, most all of which cannot be mitigated in the clarifier itself but must be addressed in the activated sludge recycle and wasting processes. Few specific issues with sludge settleability were noted, and effluent quality is consistently low in Total Suspended Solids.

The effluent weirs have the typical evidence of growth of algae, but the system is well maintained to minimize the accumulation. Overall, the clarifiers are in good condition, of abundant capacity with minimal noted operational or maintenance issues. The south clarifier mechanism is noted to have some observed play in the gear box that briefly delays movement of the mechanism when the motor starts. It is noticeably different than the north clarifier, but further investigation is warranted to understand if it is a problem or could potentially create one in the future.

4.5.10.2.1 Structural

No specific structural concerns were noted with the clarifiers. They were both operational at the time and most of the concrete was not visible. However, IVGID staff has noted the weir channels in both clarifiers have some exposed aggregate. As with the aeration basin concrete, it is recommended to monitor this deterioration and evaluate rehabilitation options for preserving this concrete before the deterioration reaches a point of no repair.

4.5.10.3 Recommendations

As another tool toward the operation of the clarifier we recommend use of the state point analysis tool to evaluate clarifier performance at different mixed liquor and RAS recycle rates. This is a simple spreadsheet tool used to help understand if the clarifier loading and RAS recycle rates are within the ideal operating range for settling, thickening and energy consumption. The tool may be most helpful in conditions with high influence of storm water and rapidly changing flows. However, it is one more tool for use in optimizing performance of the facility and for use in trouble shooting when the process experiences performance changes.

Continued semi-annual inspection of clarifiers when empty is recommended to monitor the coatings on the center column structures scraper mechanism and effluent weirs. The original clarifiers lasted from 1971 to 1997 but had some coating applications done in 1991. Therefore, the original clarifiers were rehabilitated after 25 years but this only lasted 5 years before they were completely replaced. Thus, the existing clarifiers are at roughly the same age (26 years) as the original clarifiers when a rehabilitation project was completed. Within the 20-year planning period of this report it is likely the mechanism will reach the end of its useful life and either need recoated or replaced.

The recommended clarifier mechanism replacement has been identified in Section 6.2 to be completed in 2030.

4.5.11 Effluent Disinfection Facilities

4.5.11.1 Facility Purpose and Design Criteria

Chapter 100 of the Recommended Standards for Wastewater Facilities include the following applicable criteria:

- For a chlorination system, a minimum contact period of 15 minutes at design peak hourly flow or maximum rate of pumpage shall be provided after thorough mixing.
- Piping systems should be as simple as possible, specifically selected and manufactured to be suitable for chlorine service, with a minimum number of joints.
- Storage containers for hypochlorite solutions shall be of sturdy, non-metallic lined construction and shall be provided with secure tank tops and pressure relief and overflow piping. Storage tanks should either be located or vented outside. Provision shall be made for adequate protection from lights and extreme temperatures. Tanks shall be located where leakage will not cause corrosion or damage to other equipment. A means of secondary containment shall be provided to contain spills and facilitate cleanup. Due to deterioration of hypochlorite solutions over time, it is recommended that containers not be sized to hold more than one month's needs.
- Rooms containing disinfection equipment shall be provided with a means of heating so that a temperature of at least 60°F can be maintained.
- With chlorination systems, forced, mechanical ventilation shall be installed which will provide one complete fresh air change per minute when the room is occupied.

4.5.11.2 Process Description, Condition and Capacity

The facility uses sodium hypochlorite to disinfect treated effluent. The chemical building provides for chemical receiving, storage, and feed equipment for final disinfection. There are two main purposes for the use of chlorine at the IVGID facility; disinfect treated effluent to be safe for the environment once discharged; and control of filamentous bacteria through injection in the RAS pipeline returning from the clarifiers. Historical injection points also included injection into the influent in the headworks and use for odor control, neither of which are currently used.

The plant contains two 4,050-gallon storage tanks located in a spill containment area for safety. Additionally, there are three "Blue-White" peristaltic metering pumps. Two of the pumps treat the effluent from the plant, while the third is designated to dose the RAS line for filamentous bacteria control. All three of the chlorine lines are non-diluted. Chlorine is injected into the effluent water after the final clarifiers.

The chemical tanks, pumps and process piping are all in good condition, though one small leak on the piping had developed on the discharge of one of the tanks and in the spill containment area.

The tanks, pumps and building room enclosure meet the general requirements of the criteria noted from Chapter 100 of Recommended Standards for Wastewater Facilities. The actual ventilation rate from the room, however, was not verified to confirm the noted room exchange rate is actually achieved.

4.5.11.2.1 Chlorine Demand Challenges

The IVGID WRRF is currently designed for BOD and TSS reduction by Activated Sludge Treatment, not to treat for ammonia conversion or full nitrogen removal. However, the facility has experienced significant nitrification during summer months. This is illustrated in Figure 71. In the figure, Total Kjeldahl Nitrogen (TKN), which excludes Nitrites and Nitrates, is plotted for the facility influent and effluent from January 2020 to October 2022. The drop in effluent TKN indicates the transformation of organic nitrogen and ammonia to inorganic nitrites and nitrates. The nitrification process includes biological oxidation from ammonia to nitrite by ammonia-oxidizing bacteria, and then further oxidation of nitrite to nitrate by nitrite-oxidizing bacteria, the later step taking place rapidly.

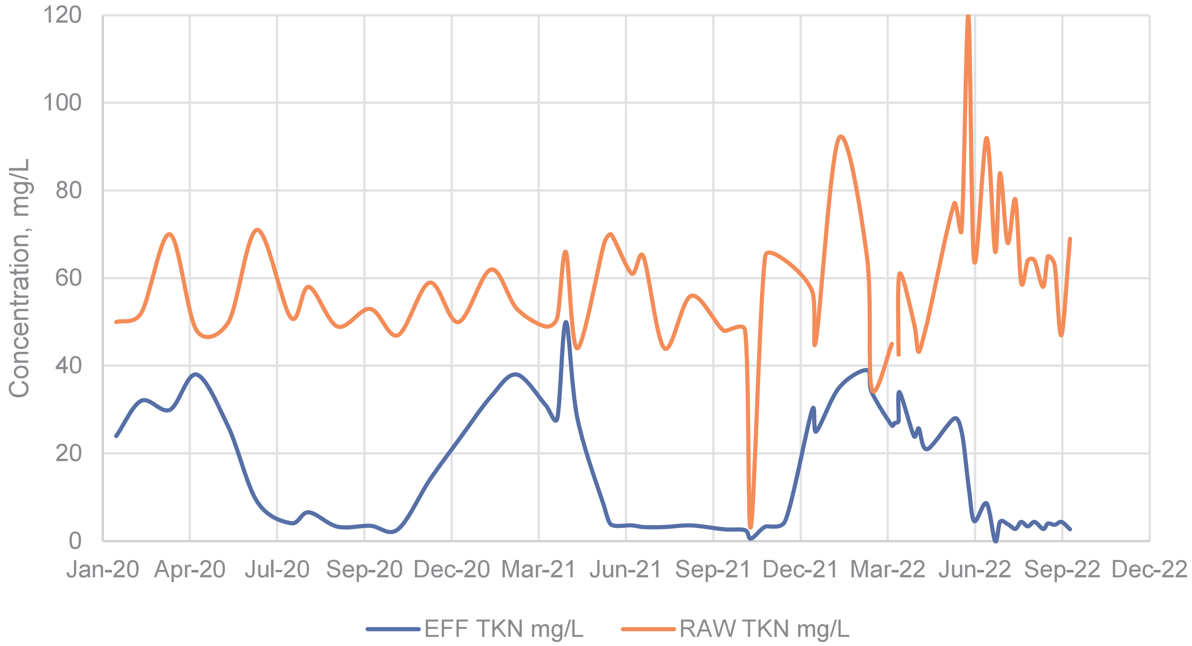

Figure 71: Influent (Raw) and Effluent (Eff) TKN concentrations, 2020-2022

The facility has experienced substantial consumption increases of sodium hypochlorite. This is thought to be the result of incomplete nitrification resulting in elevated nitrite concentrations in the secondary effluent.

- Nitrification is the conversion (oxidation) of ammonia (NH_4^+) to nitrate.
- The two-stage process includes two specific types of bacteria known as Nitroso-bacteria and Nitro-bacteria as follows:
- Step 1: (Nitroso-bacteria) $2NH_4^+ + 3O_2 > 2NO_2^- + 4H^+ + 2H_20$
- Step 2: (Nitro-bacteria) $2NO_2^- + O_2 > 2NO_3^-$
- Total Oxidation Reaction: $NH_4^+ + 2O_2 > NO_3^- + 2H+ + H_2O$
- Further, these autotrophic bacteria need a carbon source for their metabolism and consume alkalinity in the form of bicarbonate as follows, as part of the overall reaction:
- $NH_4^+ + 2HCO_3^- + 2O_2 > NO_3^- + 2CO_2 + 3H_2O$; For each gram of ammonia nitrogen converted, 7.14 grams of alkalinity, as $CaCO_3$ is required.
- The noted bacteria for the nitrification step grow much slower than the heterotrophic bacteria responsible for BOD removal. Thus, processes designed for nitrification require longer hydraulic residence time and solids retention time to grow sufficient "nitrifiers" for nitrification.
- Typically, at temperatures above about 17 deg C, the conversion step from nitrate to nitrite is rapid, leaving little free nitrite in the water.
- As temperatures decrease below 14 deg C, the oxidation of nitrite to nitrate slows and becomes the controlling step in nitrification, allowing nitrites to accumulate.
- Incomplete nitrification (conversion of ammonia NH_3 & NH_4^+ to nitrates, NO_3^-) results in accumulation of nitrite, NO_2^-.
- Nitrite reacts with chlorine at a ratio of 5 mg/L of chlorine to 1 mg/L of nitrite to oxidize the nitrite to nitrate.
- $NO_2^- + HOCL + H_2O > H_3O^+ + NO_3^- + CL^-$

Also, at work with the addition of chlorine to wastewater effluent is the reaction of chlorine with any free ammonia. The combination of ammonia (NH_3) and chlorine forms monochloramine. However, adding more chlorine will continue for form di- and tri- chloramines, both less effective at disinfection. Further addition of chlorine will reach a breakpoint such that any ammonia present has been converted and each part of chlorine dosed thereafter will cause one part of chlorine residual formed.

Thus, both residual ammonia and incomplete nitrification resulting in the presence of nitrites can significantly increase the required chlorine dose.

Figure 72: Hypochlorite Tanks

Figure 73: Hypochlorite Feed Pumps

4.5.11.3 Recommendations

The volume of chlorine required to maintain adequate disinfection is a significant cost to the facility operations. The most common wastewater effluent disinfection method for wastewater effluent is ultraviolet (UV) light, though in this case, UV would not necessarily help with reducing the chlorine demand. If ammonia and nitrites are adding to the chlorine demand, these would not be removed, or the associated chlorine demand reactions prevented through the use of UV.

A residual would still be preferred in the effluent pipeline to minimize biofilm growth in the 20 miles of effluent pipeline.

Alternative forms of disinfection could be considered such as chlorine gas, chlorine dioxide or ozone, but each has safety and cost implications and may still require chlorine use to maintain a residual in the effluent pipeline.

To reduce the hauling of a generally hazardous liquid sodium hypochlorite, this can be generated on site with on-site hypochlorite generation systems that use salt, water, and electricity to create sodium hypochlorite on site. The source material delivered to the facility would be salt instead of liquid bleach. It is recommended to investigate the implications of installing on-site hypochlorite generation system.

As identified in Section 6.2 it is recommended that a basis of design report for an on-site hypochlorite system be completed in 2030, with a target to install the system in 2031.

4.5.12 Effluent Storage Tank

4.5.12.1 Facility Purpose and Design Criteria

Once the effluent has passed through the secondary clarifier it is then disinfected and finally flows to a 500,000-gallon effluent reservoir. The effluent from the facility is exported out of the Lake Tahoe Basin to one of three permitted outfalls. The effluent storage tank provides operational storage for the Spooner Effluent Pump Station and approximately 8 to 12 hours of effluent storage depending on the prevailing influent flow rates into the WRRF. There are no specific design criteria for this facility in the context of an effluent storage facility, but the facility does provide chlorine contact time prior to the point of compliance at the effluent pump station. Typical water storage tank design criteria for pipe inlet/outlet provisions, isolation, venting, overflow, safety, and access, draining, etc. also apply to the storage tank.

4.5.12.2 Condition

The storage tank picture in Figure 74 and Figure 75 is reported to be in poor condition with the interior coating beyond the life of the coating system. Removing the facility from service is complicated in having no alternative storage to provide both the chlorine contact time and the operational storage needed for the effluent pump station operation. Thus, maintenance is very difficult, as removal of the tank from the overall operation effects the effluent pump station, particularly when sewer flows into the facility are lower.

IVGID has plans in development for an additional effluent storage tank to replace or supplement the existing tank and allow rehabilitation of this existing tank at the District’s discretion.

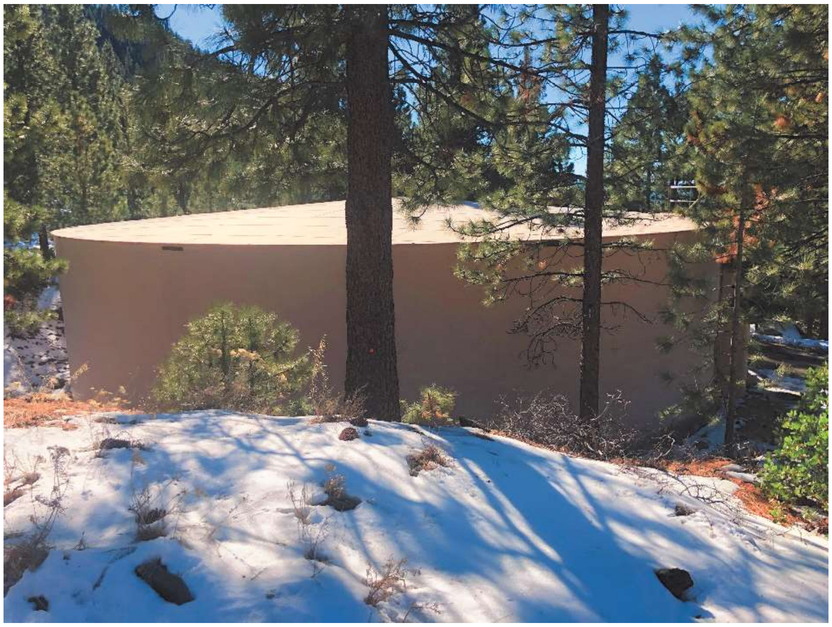

Figure 74: Effluent Storage Tank

Figure 75: Effluent Storage Tank

4.5.12.3 Recommendations

Redundancy and flexibility for operations and maintenance is needed in the effluent storage facilities. It is recommended to retain this tank and rehabilitate the facility with a sandblast, any necessary steel rehabilitation, and application of a modern interior tank coating system upon completion of the additional effluent storage tank. The redundancy would allow for more flexibility in maintaining either storage tank or provide additional shut down time if work needed to happen at the effluent pump station and/or on the effluent export line.

The rehabilitation option may warrant an analysis to assess the impact of modern seismic standards and whether a retrofit is cost effective or if a total replacement would be needed. The range of possible retrofit needs could include external roof anchorages, internal connections enlarging footings, adding foundation anchorages and/or thickening of the lower part of the tank shell.

The proposed 2-million-gallon effluent storage and rehabilitation of the existing storage tank have been identified in Section 6.2 to be completed in 2025 and 2026, respectively.

4.5.13 Effluent Disposal System

The Spooner Pump Station sends effluent out of the Tahoe Basin to the Wetlands Enhancement Facility in the Carson Valley near Hot Springs Mountain in northern Douglas County. In addition to the wetlands disposal area, there are two other irrigation sites permitted for effluent reuse, Clear Creek at Tahoe, and Schneider Ranch. The wetlands facility was completed in 1984 and has served the District since. Prior to that time effluent was used for irrigation of hay fields in the summer and was discharged to the Carson River during the winter months. More stringent effluent criteria prompted development of the wetland facility. The site covers approximately 900 acres, including approximately 770 acres of constructed wetlands, natural warm-water wetlands, seasonal storage/waterfowl areas, effluent storage area and an upland area.

The constructed wetlands are the primary disposal area while the seasonal storage area stores water during low evaporation periods and high rainfall and are typically dry during the summer and fall. The 200-acre upland area was intended to provide for effluent disposal via spray irrigation during extended rainy weather, but this disposal site is not used.

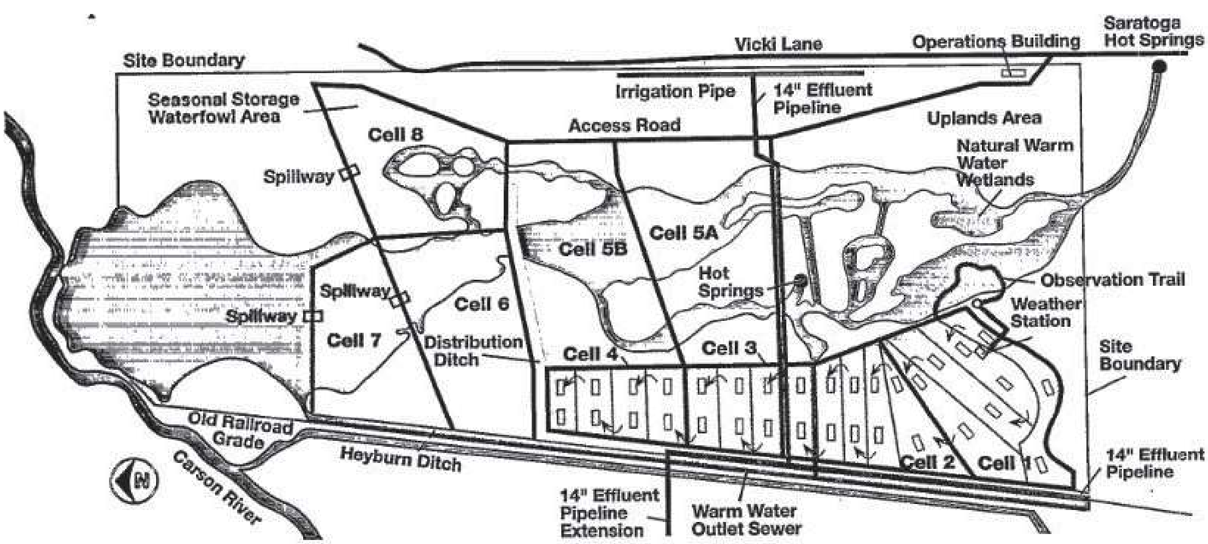

Figure 76: Wetlands Enhancement Facility (Image from EPA832-R-93-005k)

Effluent travels through approximately 390 acres of constructed wetland cells and is subject to evaporation, transpiration through the wetland plants and percolation through the soil, avoiding discharge to the Carson River. Thus, no surface water quality criteria must be met, and the

WRRF does not have to specifically treat for either ammonia conversion or nutrient (nitrogen and phosphorous) removal beyond the limits noted in Table 74 for the quality of the effluent entering the wetland facility.

The design average daily annual flow to the facility is 1.66 MGD with a maximum daily design flow of 2.68 MGD. Other design criteria are noted in Table 74.

Table 74: Wetland Enhancement Facility Design Criteria

Treated Water Quality Parameter(s), mg/L	Value
Total Suspended Solids	20
BOD5	20
Total Dissolved Solids (TDS)	240
Total Phosphorous (TP)	6.5
Total Nitrogen (TN)	25
Wetland Area Sizing (acres)	
Cell 1	37.9
Cell 2	33.2
Cell 3	27.3
Cell 4	23.4
Cell 5 (overflow area)	117.3
Cell 6 & 7 (floodplain area)	105.6
Cell 8 (seasonal storage)	42.5
Wetland Depth (ft)	
Emergent Marsh	0.5
Open Water	2.0-3.0

The District groundwater discharge permit (#NS0030009) requires water quality sampling of influent and effluent at the WRRF and monitoring of the effluent into the wetland facility and at seven monitor well sites around the wetland facility. Monitoring parameters at the wells include those listed in Table 75.

Table 75: Groundwater Monitoring Well (MW) Parameters

MW Sampling Parameters
Chloride (as Cl)
Nitrogen, nitrate total (as N)
Nitrogen, total (as N)
Solids, total dissolved
pH, maximum
pH, minimum

The effluent reuse, land application sites permitted are listed in Table 76.

Table 76: Effluent Reuse Sites

Effluent Reuse Site	Authorized Quantity (ac-ft)
Clear Creek at Tahoe	350
Schneider Ranch	99

4.5.13.1 Recommendations

The District has noted no major deficiencies though some of the headgate structures between various cells are in need of rehabilitation and currently budgeted for work in the near term. Ongoing work maintaining the levees among the cells to address vegetation and rodent holes but otherwise the 40-year-old facility continues to provide stable performance and a long-term solution for the District's effluent disposal.

4.5.14 Solids Handling Facilities

Waste sludge from the biological treatment process is periodically pumped from the clarifier underflow and RAS suction pipeline to the aerated sludge holding tanks. The sludge is stored and aerated and then pumped to two dewatering centrifuges for dewatering. The sludge cake is hauled off site to the Bently Ranch near Minden, NV where four other regional wastewater treatment facilities are also able to dispose of their biosolids.

The current biosolids disposal site at Bently Ranch is approximately a 60-mile round trip. The alternative site at the Lockwood landfill east of Reno would add approximately 40 miles to the round trip.

The final sludge removal provisions, hauling limitations and disposal requirements of the waste solids from the WRRF dictate the level of treatment and dewatering required at the WRRF. Some facilities temporarily store waste solids for removal by a pumper truck for transport of liquid sludge either to another facility for treatment or to a land application site for injection or spreading over agricultural fields. On the other extreme are facilities with biosolids drying facilities where the liquid sludge is dewatered with mechanical dewatering equipment and then processed through a biosolids dryer facility, so the solids leave the facility with zero moisture content and as dried pellets in large sacks for transport and disposal at a landfill.

Some land application sites require digestion of the waste sludge to meet the definition of a Class B biosolid for the percent of volatile solids destruction and pathogen reduction achieved through the sludge treatment. Biosolids reuse applications require a higher level of processing and treatment to establish a Class A classification to allow unrestricted reuse of the material.

IVGID dewaters the waste sludge and delivers the cake to the Bently Ranch for subsequent composting to establish a Class A biosolid. Thus, the goal of solids handling processes at the IVGID facility is optimization of the sludge dewatering process to minimize energy use, conditioning polymer use and the amount of water weight and volume that must be hauled 40 miles to the Bently Ranch. The lower the water content the less weight and cost for hauling the material.

The degree of sludge digestion affects the efficiency and consistency of the dewatering process. The use of dewatering centrifuges for dewatering provides an effective dewatering process requiring less operator attention, dry cake solids and less overall volume produced. The process can be less dependent upon the degree of digestion preceding the dewatering equipment.

Thus, the current biosolids handling process at IVGID includes what is effectively aerated sludge holding tanks followed by direct dewatering of the sludge through the two existing centrifuge sludge dewatering machines. As long as this process is working well and not creating issues dewatering the sludge and complicating transport of the cake, then additional digestion of the waste sludge to meet the Class B biosolids criteria for volatile solids reduction and pathogen reduction is not necessary.

Other criteria for sludge dewatering include energy consumption, polymer use and operator attendance.

4.5.15 WAS Holding Tanks & Digester

4.5.15.1 Facility Purpose and Design Criteria

The existing facility is not designed to meet the criteria for a Class B digested biosolids and serves simply as an aerated sludge holding tank. As long as equipment can effectively dewater the sludge for hauling and disposal there is no specific requirement for how long the sludge is in the holding tanks, and the aeration is provided to minimize odor and prevent anaerobic conditions before the sludge is pumped to the dewatering centrifuges.

4.5.15.2 Process Description, Condition and Capacity

The tanks previously operated as a small wastewater plant with aerated basins around the perimeter of the tank and the center circular tank served as a single secondary clarifier. This facility was repurposed multiple times over the last 30 years, and nearly every improvement project since has included modifications to this facility. One past project included installation of a cover over the tanks, and a few projects later, the covers were removed.

The tanks are currently used to store WAS before entering the Solids Handling Building and going through the dewatering process. There are three separate storage tanks within the holding facility #1, #3 and #4. WAS is pumped from the bottom of the secondary clarifiers into one of the three holding tanks. Air is supplied from Blower #4 in order to keep the sludge from going septic and providing a moderate level of actual digestion.

The WAS Storage Pump Station pumps WAS from the holding tanks into the solids handling building, where it goes through the dewatering process.

Figure 77: Center Sludge Holding Tank

Figure 78: Sludge Holding Tank

In earlier iterations the tanks included a WAS thickening stage. WAS directed to the center tank (old clarifier) and then processed through a centrifuge and then to one of two other sections of the solids holding tanks storage tank termed as Digester #1 and Digester #2. The final of the four (4) tank zones was for storage of the digested sludge for batching to the other two centrifuge units for final sludge dewatering. The purpose was to reduce the volume of water sent through the final dewatering equipment while conditioning and thickening the sludge to closer to 1.5%. This process has not been used for many years. The thickening centrifuge had mechanical issues and has not operated since. Thus, the four areas of the solids holding tanks are all used as aerated sludge holding tanks and sludge at approximately 0.5% solids is pumped to the two other centrifuges for successful dewatering.

A single Kaeser, positive displacement blower provides process air to both the sludge holding tanks and the aerated grit chamber for grit displacement and air lift of the grit into the grit classifier. Operations staff have noted multiple leaks in the buried aeration piping between the blower building and the sludge holding tanks. Air bubbles can be seen during melt periods when there is standing water on the west side of the building over the aeration piping. When several of the sludge holding tanks are under aeration at the same time, the single Kaeser blower does not have adequate capacity to meet the air demand of the sludge holding tanks and the aerated grit chamber. The loss of air may be significant and such that repair of the air lines would make a difference toward meeting the overall air demand. However, there is currently no backup to this smaller blower, and a second blower could be added to both provide redundancy and additional capacity when all the demand is needed to run concurrently. Alternatively, the primary aeration blowers could be evaluated to determine if excess air can be diverted to the digesters.

Several existing valves used to direct air to different tanks are not safely accessible and either need to be relocated, or new access platforms are needed to access. They are currently suspended in several locations above and inside the tanks where safe access is not possible, and operators are placed in precarious conditions if and when these valves warrant any operator attention.

4.5.15.2.1 Structural

The concrete of the sludge holding tanks is the oldest concrete at the facility. A few areas of the walkways show some concrete spauling in need of some cosmetic work. The interior walls actually have less aggregate exposed than the much newer aeration basins and generally look to be in fair condition considering the age and use. IVGID has reported some observed leakage of water between zones 1 and 2. This area warrants further monitoring and mitigation if or when this leakage impacts the process or impedes dewatering and maintenance on either side of the wall where leakage is occurring. Epoxy injection and other similar mitigation methods are available to restore or improve the watertightness at cracks or joints in the concrete structure.

4.5.15.3 Recommendations

Recommendations for the sludge holding tanks include the following:

- Replace the air lines from the blower to the sludge holding tanks with either new buried lines or route the air lines above grade.
- Add a second blower of same or comparable size to provide redundancy and supplement the air supply during peak needs.
- Relocate valves (whether manual or actuated) to locations where they can be readily accessed. If not at ground level, provide access walkways or platforms suitable for safe access and maintenance.
- Specifically monitor the respective tank compartments for leakage and/or increases in leakage. Investigate and complete repairs as needed to maintain operability and maintenance of the facility.

The recommended sludge holding tank improvements has been identified in Section 6.2 to be completed in 2028.

4.5.16 Sludge Dewatering Systems

4.5.16.1 Facility Purpose and Design Criteria

Sludge dewatering criteria are subject to the disposal location requirements and the sludge conditions necessary to economically haul the sludge. With a relatively long transport and requirement for low water content sludge, the District must effectively dewater the sludge to 15-20% solids to minimize the weight and cost of hauling water and meet the requirements of Bently Ranch. The following are a few applicable regulatory criteria for sludge processing facilities.

- Provision shall be made to maintain sufficient continuity of service so that sludge may be dewatered without accumulation beyond storage capacity. The number of vacuum filters, centrifuges, filter presses, belt filters, other mechanical dewatering facilities, or combinations thereof should be sufficient to dewater the sludge produced with the largest unit out of service. Unless other standby wet sludge facilities are available, adequate storage facilities of at least 4 days production volume in addition to any other sludge storage needs shall be provided. Documentation must be submitted justifying the basis of design of mechanical dewatering facilities.
- Adequate facilities shall be provided for ventilation of the dewatering area in accordance with Paragraph 42.75. The exhaust air should be properly conditioned to avoid odor nuisance.
- 42.75 Ventilation, if continuous, shall provide at least 12 complete air changes per hour; if intermittent, at least 30 complete air changes per hour. The air change requirements shall be based on 100 percent fresh air.

4.5.16.2 Process Description, Condition and Capacity

The sludge dewatering includes sludge transfer pumps, polymer feed assemblies, mechanical dewatering centrifuges, sludge cake conveyors and sludge cake collection and transport containers. The sludge pump station inside the solids handling building pumps aerated WAS from the WAS holding tanks to the sludge dewatering centrifuges. As noted, the former WAS thickening process is no longer used so the current process includes only pulling WAS from the holding tanks directly to the centrifuges.

The Sludge Pump Station is designed to transfer sludge from the WAS storage tanks (#1, #3 or #4) into the solids handling building to begin the dewatering process. Specifications for the WAS Sludge Pumps and the centrifuges are outlined below.

The District has recently replaced several of the existing progressive cavity pumps with new Liberty pumps. The process piping and valves are in fair condition no significant pipe coating degradation. The room is well ventilated, minimizing the accumulation of humidity on the colder pipes and equipment in the room.

The system includes two, new polymer makeup and feed systems, one for each of the operating centrifuges. Each unit uses potable water to blend with the polymer before feeding into the liquid sludge stream enroute to the centrifuge. Bulk polymer delivered in 300-gallon totes is transferred to a day tank near the polymer feed skids.

The centrifuges effectively dewater the relatively dilute WAS without any subsequent thickening before reaching the centrifuges. The initial dewatering process with the centrifuges included a thickening stage and a thickened WAS sludge holding tank use to hold the thickened WAS for pumping back to the “digester” before digested WAS fed back to the other two dewatering centrifuges. This thickening step is removed, and the two operable centrifuges discharge the sludge cake directly to a conveyor that fills the cake storage and transport container.

The centrifuges and conveyors are approaching 20 years of age (replaced in 2004) but are generally in good condition. The polymer systems are new within the last few years. With a typical projected or planned life of 25 years for major mechanical equipment, consideration should be given to planning for major rebuild of the centrifuges within 20-year planning period of this report.

All of the dewatering equipment is of sufficient capacity to meet the current and projected flows to the WRRF. Redundancy is generally sufficient in the pumping and dewatering equipment, though loss of one of the centrifuges would impact operations and require more hours of operation.

Table 77: Sludge Dewatering Feed Pump Capacity

Equipment	Capacity, GPM	Pressure Drop, FT TDH	HP	Volts
DSP Pump	250	22.5	5	480/3

Table 78: Sludge Dewatering Centrifuges

Equipment Name	Make/ Mod	Flow Capacity	Head Capacity	HP	Volts	Phase
Centrifuge 1	Centrysis/CS18-42P	110 GPM	N/A	75	480	3
Centrifuge 2	Centrysis/CS18-42P	110 GPM	N/A	75	480	3

Figure 79: Sludge Pumps (Centrifuge Feed)

Figure 80: Dewatering Polymer Makeup and Feed System

Figure 81: Dewatering Centrifuge

Figure 82: Polymer Transfer Pump and Sludge Cake Hopper

4.5.16.3 Recommendations

Loss of a centrifuge for even a few days would start to have a significant effect on operations and eventually plant performance. Thus, the following are recommendations for the sludge dewatering processes.

- Replace the electrical controls and replace the existing controller and operator interface terminal (OIT) with a new Allen-Bradley system.
- To provide additional backup to the existing centrifuges, add a third centrifuge where the existing smaller, inoperable centrifuge is currently located. An identical unit will not fit in this location, but a smaller unit would still provide a key backup provision in

the event either of the existing two centrifuges had a mechanical issue and needed to be shut down for maintenance.

- Purchase at least one spare rotating assembly to have on site for installation into either of the two, existing centrifuges.

The recommended sludge dewatering system improvements has been identified in Section 6.2 to be completed in 2029.

4.5.17 WRRF Electrical Equipment Evaluation

The Incline Village GID (IVGID) has an electrical distribution system at the WRRF that consists of transformers, switchboards, motor control centers, VFDs, automatic transfer switches, generators, and branch circuit panelboards. DOWL performed a site visit to evaluate the existing electrical distribution system from a high-level perspective. The resulting observations, findings, and subsequent evaluation for the WRFF concludes that a significant portion of the existing electrical equipment are antiquated, lack reliance of spare parts, and have a high consequence of failure that could result in prolonged, unexpected outages. The age of the internal wiring and circuit breaker contacts could cause catastrophic failure. For these reasons, this report recommends that all equipment exceeding expected useful and reliable life be planned for replacement.

An onsite electrical field assessment was performed on 4/06/2023. IVGID electrical personnel provided guidance and working knowledge of the electrical facilities during the assessment. Detailed photos of the exterior condition on the distribution equipment were documented and are used as part of this report to convey the extent of the conditions. Interior photos of a few motor control center (MCC) buckets were captured but further investigation of interior conditions, including distribution bussing, will encompass a more detailed evaluation, and is not within the scope of this assessment. Personal protective equipment complying with NFPA 70E and scheduled power outages would be required should IVGID desire a more in-depth investigation of each major piece of distribution equipment.

4.5.17.1 Scope of Work

The scope of work for this assessment was limited to the major electrical distribution equipment within the facility. Observations were made of the existing transformers, panelboards, disconnect switches, MCCs, raceways, and any visible wiring. Recommendations are documented so that IVGID has the information they need to schedule maintenance or plan capital improvement projects that can be taken to management for approval.

4.5.17.2 Equipment Reliability

Anticipated life expectancy of industrial electrical equipment is discussed in the "IEEE Gold Book-Recommended Practice for the Design of Reliable Industrial and Commercial Power Systems," which presents the fundamentals of reliability analysis in industrial and commercial electric power distribution systems. The IEEE recommended years of reliable life expectancy for power distribution equipment is listed in Table 79.

Table 79: Electrical Equipment, Expected Useful Lift

Equipment Type	Expected Useful Life Period (years)[22]
Transformers	25 to 30
Circuit Breakers	15 to 20[23]
Switchboards/Switchgear	30 to 40
MCC/Motor Starters	20 to 30
Panelboards	30
Motors	18 to 25
Generators	5 to 20
UPS	10
Luminaire	20
Capacitors (Power Factor Correction)	17
VFDs	20
Cable/Wire	30 to 40

Furthermore, the 'Bathtub Curve" shown in Figure 83 details the three key periods of a component's life cycle. This curve shows, how, during the life of an electrical component, it goes thru a period of useful life but is bookended with failure in the early stage due to 'infant mortality' or early failure of defective components, and 'wear-out period' of when a component is old and beyond repair due to unavailable parts and warranty.

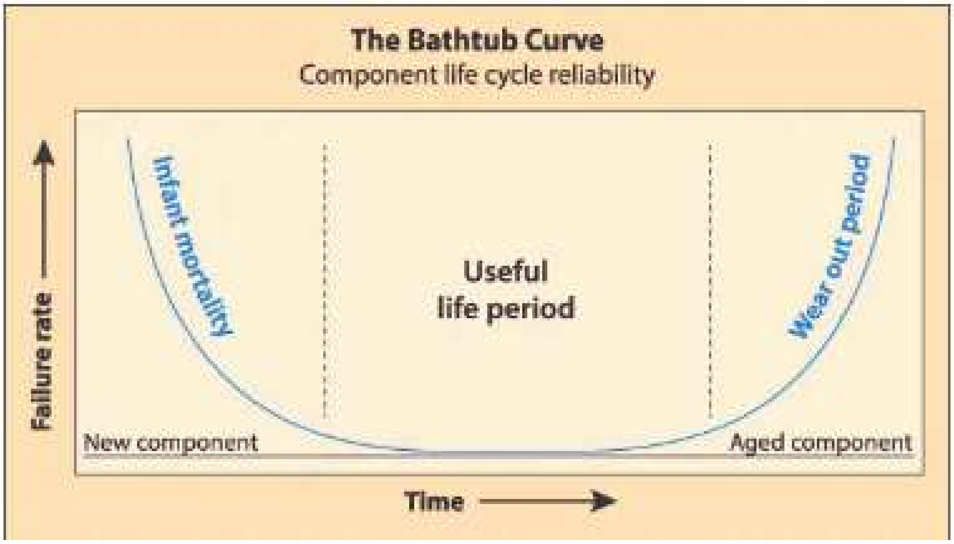

Figure 83: Component Reliability, Bathtub Curve

Electrical distribution equipment can and typically does remain in service long after the recommended time frames. However, being operable does not mean functioning per specs.

[22] *Life expectancy info collected from ABB,CDA, CDM, IEEE Gold Book, and Siemens*

[23] *By year 10, 50% of circuit breakers do not function properly per specs. By year 20, 90% does not function properly.*

Often electrical problems do not present themselves until failure occurs unless a preventative maintenance schedule is implemented.

4.5.17.3 Recommend Strategic Replacement

IVGID has been maintaining and replacing parts as needed but would benefit from a more pro-active approach in equipment replacement to avoid unplanned outages and hazardous situations caused by aged equipment and installations conforming to outdated codes. Reliable and constant operation of the electrical distribution system is crucial to the WRRF. An example that illustrates the need for maintenance and/or replacement are some of the panelboards and dry-type transformers in the facility. The photos below show the corrosion effects after many years in service.

Figure 84: Panel Board Corrosion

Figure 85: Transformer Corrosion

4.5.17.4 Observations

Table 80 lists the condition of the electrical equipment that was observed and is recommended action for remediation.

Table 80: Electrical Equipment Condition Summary

Equipment	Condition Assessment	Recommended Action
Panelboard	Wear out period	Replace
MCC Westinghouse 2100	Wear Out period	Replace
Dry-Type Transformers	Wear Out period	Replace
Raceways	Useful Life	Monitor and maintain
Disconnect Switches	Wear out period	Monitor and maintain
Wiring (where visible)	Useful Life	Monitor and maintain
Control Cabinets	Wear out period	Replace

4.5.17.5 Conclusion

Based on the 'wear out period' assessment of some of the equipment in the WRRF there is a high consequence of future failure due to the lack of spare parts, expiration of any warranty, and the lead time to obtain such equipment. It is recommended that this equipment be replaced. Replacement can occur over a period as budget and scheduling permits. It must be emphasized that the internal parts and lack of serviceable components will result in failures and long-term outages of critical plant loads. A systematic approach is recommended to minimize down time as well, so that all the equipment is up to date with warranties and spare parts. Scheduled outages of critical loads can be kept to a minimum by coordinating the replacement effort with the plant operations.

The typical, approximate costs of equipment recommended for replacement are shown in Table 81. Keep in mind that these are budget-level equipment cost estimates based on previous experience and do not include labor and incidentals. Also note that not all the equipment in the plant may need to be replaced. A more thorough examination will be required to ascertain a more exact condition and prioritization of proposed equipment improvements or replacements.

Table 81: Electrical Equipment, Expected Useful Life

Item	Description	Unit Cost
1	VFD	$30,000
2	MCC	$120,000
3	RTU, Control Cabinets	$50,000
4	Dry Type XFMR	$8,000
5	480V Panel	$5,000
6	120V Panel	$3,500
7	Disconnect Switches	$1,500

4.5.17.5.1 Centrifuge Equipment Electrical

Operations staff have noted the electrical and controls equipment for the centrifuges are in need of replacement. The controls hardware and control program are obsolete and in need of replacement with updated and non-proprietary hardware to facilitate ongoing support of the equipment.

4.6 Recommendations

Several deficiencies have been identified in the WRRF. Table 82 is a list of recommended projects to address those deficiencies. Project cost estimates and a prioritized CIP can be found in Section 6.0.

Table 82: Recommended WRRF Projects

Project	Project Description
New Effluent Storage Tank	Design and construct a new 2-million-gallon effluent storage tank.
Aeration System Improvements	Increase the capacity of the existing jet aeration system to meet the peak organic influent loading.
Effluent Storage Tank Rehab	Recoat the interior and exterior of the existing effluent storage tank, perform miscellaneous repairs, and a seismic upgrade retrofit.
Headworks Improvements	Install recirculation/chopper pump in influent channel.
Headworks Second Screen BDR	Evaluate the mechanical and structural modifications necessary to install a second screen.
Headworks Second Screen Installation	Install the second influent channel screen.
Aeration Basin Lining	Preserve the existing aeration basin concrete by lining the basin.
Aerated Sludge Holding Tanks	Miscellaneous improvements to the system blower, addition of redundant blower, and improve access to key air control valves.
Sludge Dewatering Improvements	Replacement of electrical controls, purchase of spare rotating assembly, and install of third centrifuge.
Secondary Clarifier Mechanism Replacement	Replacement of the clarifier center column, center feed well, walkway, motor and gear box, sludge scraper, scum scraper arm, sludge collection ring, scum box, and overflow weir.
Onsite Hypochlorite System BDR	Evaluate the feasibility of install an on-site hypochlorite system.
Onsite Hypochlorite System Installation	Installation of the recommended hypochlorite system.

5.0 EFFLUENT EXPORT SYSTEM

5.1 System Overview

The effluent export system begins at the WRRF and consists of a storage tank, pump station, surge tank, and approximately 20 miles of transmission main. The export system is used to convey the effluent out of the Lake Tahoe Basin and into one of three permitted outfalls: the Wetlands Enhancement Facility south of Carson City, Clear Creek at Tahoe, and Schneider Ranch. The wetlands are the primary outfall of effluent and is comprised of approximately 770 acres of the 900-acre site. The effluent travels through approximately 390 acres of constructed wetland cells before it evaporates, transpires through the wetland plants, or percolates through the soil. Clear Creek at Tahoe and Schneider Ranch utilize the effluent for irrigation.

5.1.1 Storage Tank

The existing 500,000-gallon effluent storage tank houses the treated and disinfected wastewater and acts as the head tank for the Spooner Effluent Pump Station. The primary purpose of the storage tank is to provide additional operational storage for the Spooner Effluent Pump Station. This amounts to approximately 8 to 12 hours of effluent storage depending on the influent flow rates into the WRRF. The tank also provides additional chlorine contact time before effluent leaves the WRRF into the export system. The storage tank is reported to be in poor condition, but IVGID has plans in development for an additional effluent storage tank to replace or supplement the existing tank and allow for rehabilitation or replacement of the existing storage tank.

5.1.2 Pump Station

SPS-16 was constructed in 1974 and is located approximately 3.75 miles to the south of the WRRF. The facility was most recently improved in 2012, which included the construction of the generator building, the replacement of four pumps, and the installation of a recirculation/bypass line. Table 83 summarize the pump station pumping capacity. The SPS-16 sits approximately 207 feet below the water surface level of the head tank and pumps effluent at a rate of 1,050 to 2,800 gpm through 40,100 linear feet of 16-inch main to the high point of the effluent export transmission main. The high point of the transmission main alignment is 648 feet above the minimum water surface elevation of the head tank. Figure 86 depicts the hydraulic profile of the effluent export system and was prepared by HDR for the 2003 memorandum titled, Hydraulic Operation Evaluation (HDR, 2003). SPS-16 was last assessed in 2015.

Table 83: Spooner Lift Station Summary

# Pumps	Size (HP)	Pump Flow (gpm)	Pump Head (ft)
1	250	1,050	1,010
2	250	1,100	1,010
3	350	1,350	1,010
4	350	1,390	1,010
Recirc	20	335	206
Firm Capacity	-	2,800	unknown

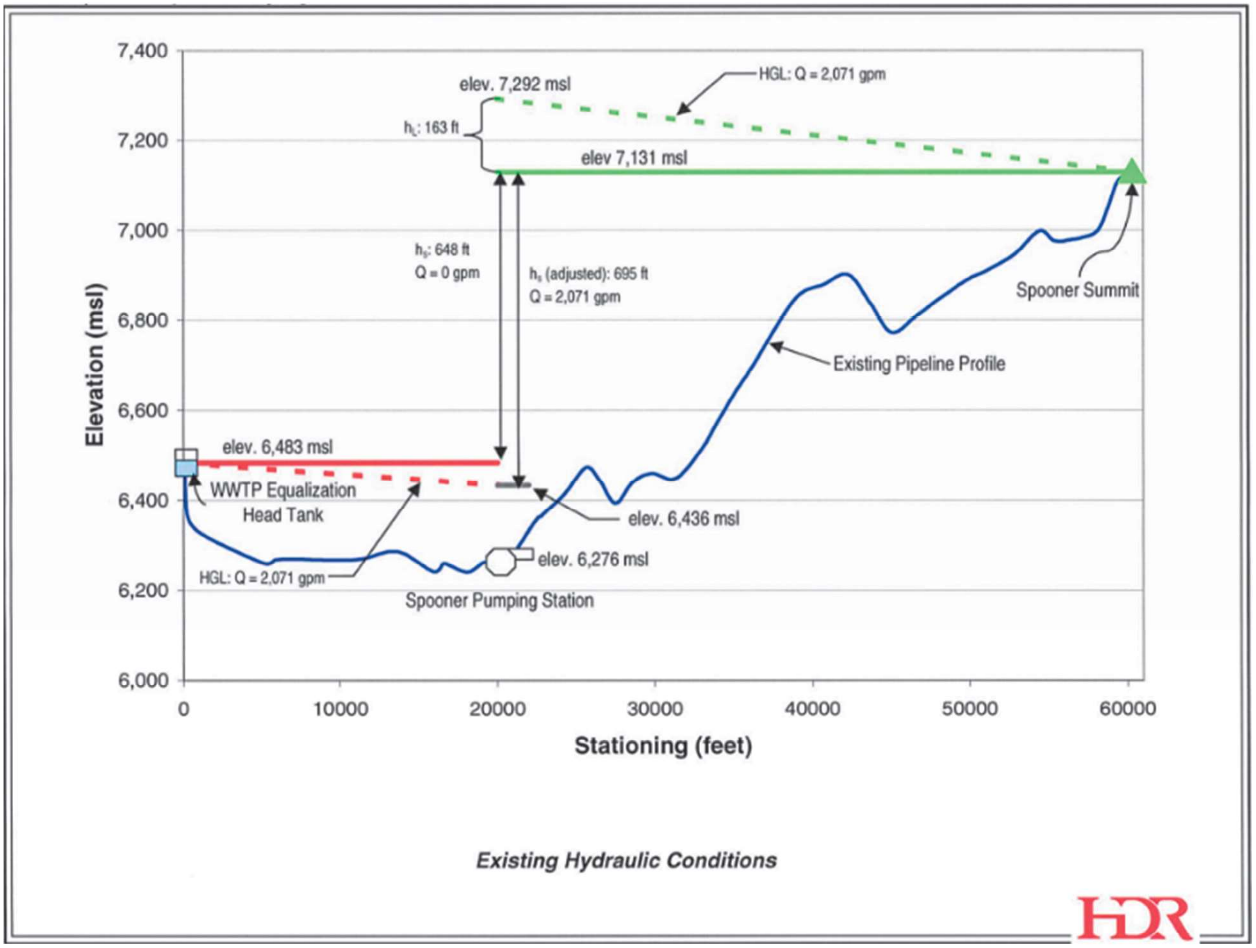

Figure 86: Effluent Export System Hydraulic Profile

5.1.3 Surge Tank

SPS-16 is equipped with a closed surge tank to mitigate the impacts of transient events. Little is known about the tank except that it was installed with the original improvements in 1974, is made from steel, and was originally designed for petroleum applications. It is recommended that a surge tank replacement basis of design report be completed to understand the size and properties of a new device based on the improvements to the effluent transmission main, current pump configuration, and current footprint of the facility. Once the preferred surge protection alternative is known it should be determined if the existing tank can be taken offline and potentially removed.

5.1.4 Transmission Main

The effluent distribution system contains over 20 miles of transmission main from the effluent storage tank to SPS-16 and terminating at one of the three permitted outfalls. The transmission main is fairly uniform in diameter and material but has a wide range of ages. Table 84, Table 85, and Table 86 give summaries of the distribution main diameters, materials, and year of install, respectively.

Table 84: Effluent Pipeline Diameter Summary

Pipe Diameter (in)	Length (ft)
14	8,027
16	99,898

Table 85: Effluent Pipeline Material Summary

Pipe Material	Length (ft)
Cement Lined Steel, CL ST	14,483
Ductile Iron Pipe, DIP	36,441
Steel, ST	57,001

Table 86: Effluent Pipeline Install Year Summary

Pipe Age	Length (ft)
1970	101,955
2006	823
2009	5,147

The export pipeline has had a number of issues and leaks over its lifetime. However, the majority of the pressurized pipeline will have been replaced within the last 20 years following the 2026 construction season. Table 87 is a summary of the leaks/issues recorded by IVGID staff over the past 3 years and Figure 87 is a map showing these approximate locations, as interpreted from notes and general location information. The locations are identified by their work order number on Figure 87.

Table 87: Effluent Pipeline Leaks/Issues Summary

Year	# Of Leaks/Issues
2022	3
2021	5
2020	2

5.2 End Users

The export system is used to convey the effluent out of the Lake Tahoe Basin and into the Wetlands Enhancement Facility, located to the south of Carson City. In addition to the wetlands disposal area, two irrigation sites are permitted for effluent reuse: Clear Creek at Tahoe and Schneider Ranch. The irrigation effluent reuse sites and their permitted quantities are listed in Table 88. Figure 88 is a map showing the location of the end users.

Table 88: Effluent Reuse Irrigation Sites and Quantities

Effluent Reuse Site	Authorized Quantity (Ac-Ft)
Clear Creek at Tahoe	350
Schneider Ranch	99

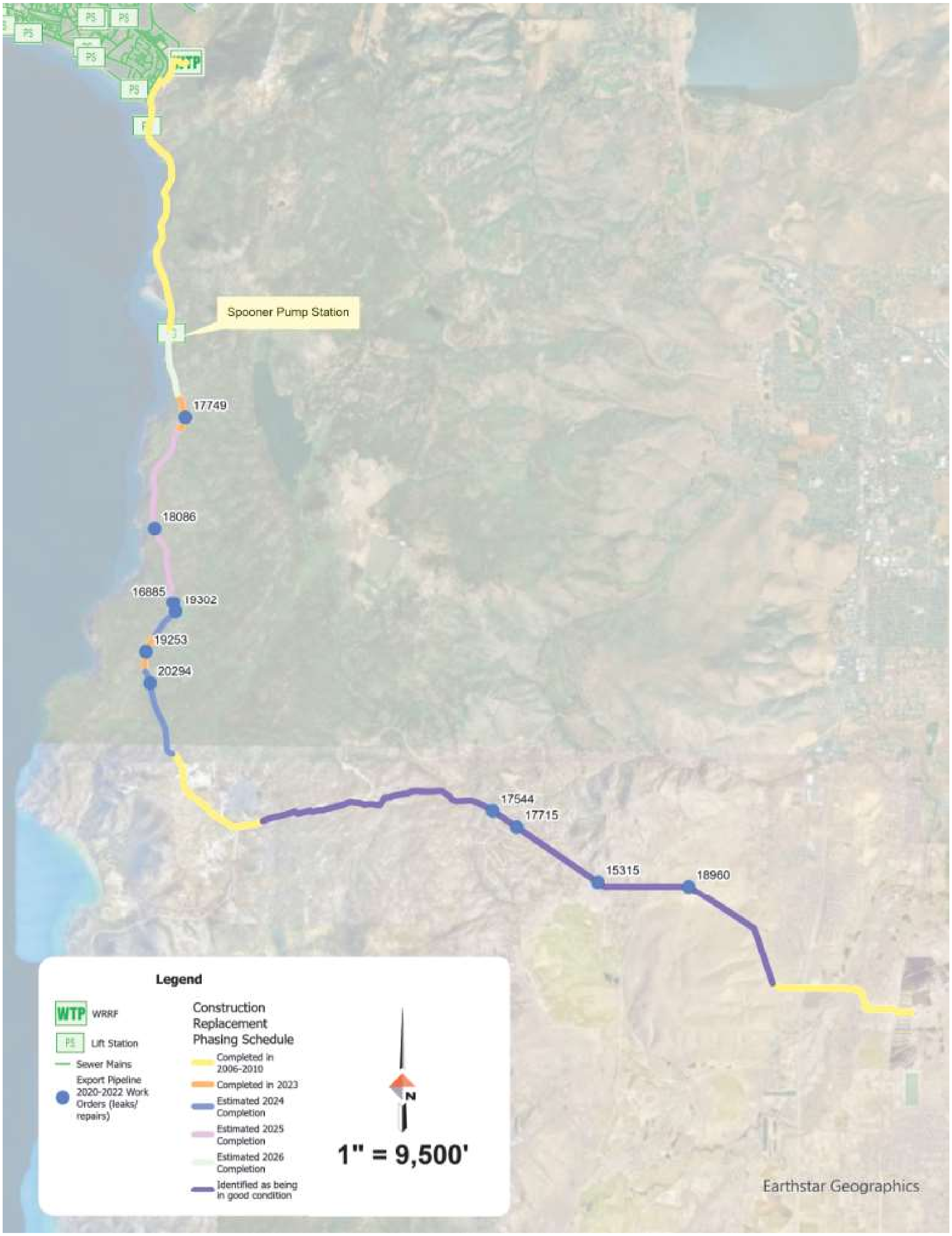

Figure 87: Export Pipeline Leaks /Issues (2020-2022)

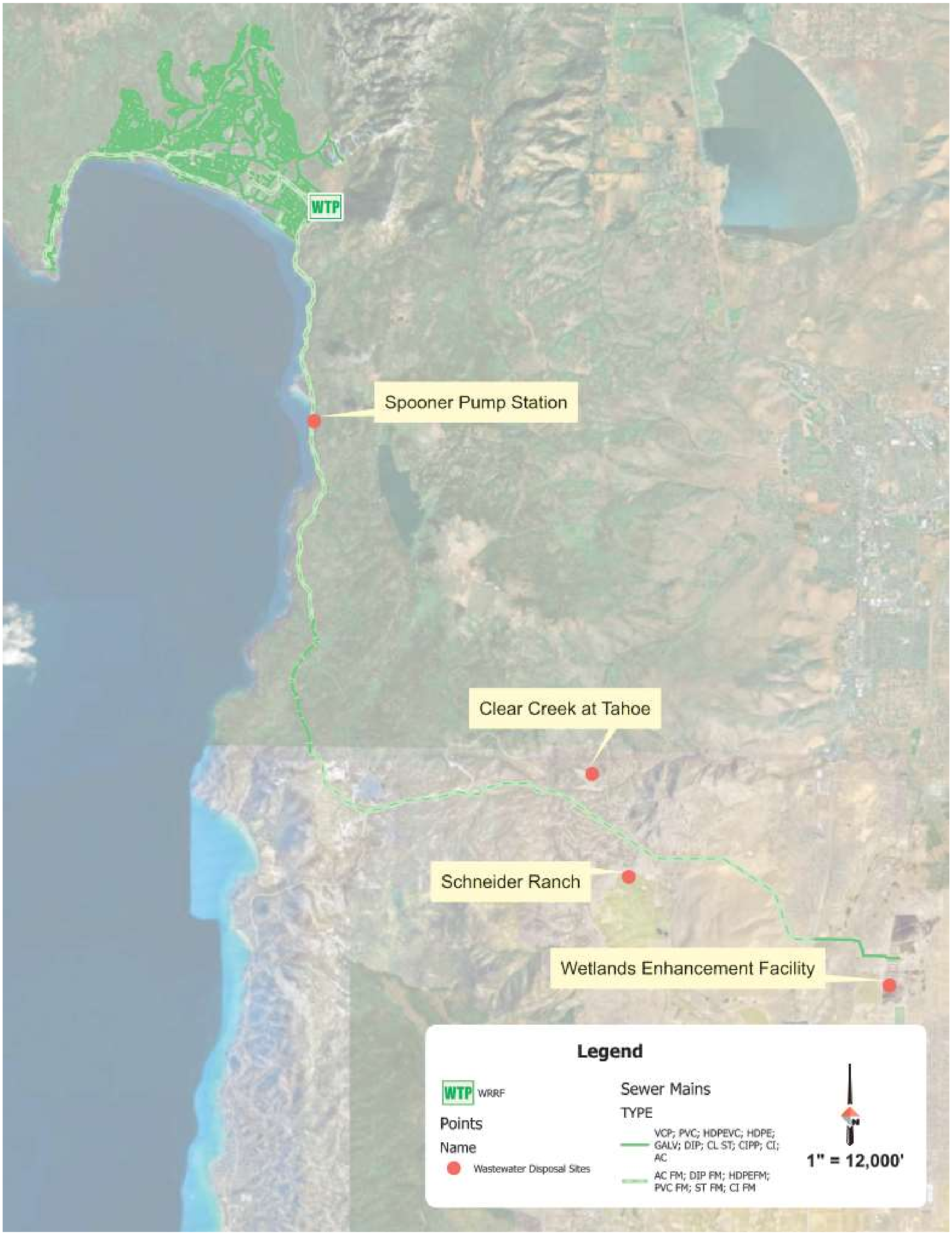

Figure 88: Effluent End Users

5.3 System Deficiencies, Operational Challenges, and Recommendations

Overall, the effluent export system infrastructure has the capacity, and facilities to provide reliable exportation of treated effluent outside the Lake Tahoe basin. However, some improvement projects are currently underway or planned to ensure that safe and reliable exportation continues long into the future. Table 89 is a list of the recommended projects to address the system deficiencies, and its current status. Project cost estimates and a prioritized CIP for the newly recommended projects can be found in Section 6.0.

Table 89: Effluent Export System Deficiencies

Project	Status	Expected Year of Completion
High Pressure Transmission Main Replacement	Underway	2026
New Effluent Storage Tank	Underway	2025
Effluent Storage Tank Rehab	Recommended	2026
SPS-16 Surge Protection Improvements	Recommended	2026

6.0 CAPITAL IMPROVEMENT PROGRAM

This section presents improvement projects, studies, investigations, and maintenance activities IVGID should consider over the next 20 years. Projects will be sorted by the recommended fiscal year of implementation and will be assigned a project type as follows:

- **Repair/Replacement** – Projects that replace failing infrastructure or assets which are beyond their useful life
- **Capacity Added** – Projects that install infrastructure or assets that resolve a system capacity pinch point
- **Maintenance** – Projects that extend the useful life of an asset
- **Condition Assessment** – Projects that update asset condition or overall risk ratings
- **Inspection** – Projects that investigate or test existing assets to document their condition or status
- **Study/Planning** – Projects that document system needs, evaluate alternatives, provide a basis of design, and update project cost estimates prior to commencing engineering design activities

6.1 Basis of Estimate

All costs shown in in this section are class 5 estimates as defined by the Association for the Advancement of Cost Estimating International (AACE International) which are conceptual (0 to 2% level of maturity) and have an expected accuracy range of -50% to +100% of the cost listed. Additionally, all costs were estimated in 2023 dollars and then projected forward to the appropriate year by an inflationary rate of 3.8%. In the following project description sections and in the CIP, the costs presented have been projected forward to the recommended fiscal year. A more thorough basis of estimate memo, and individual cost estimate breakdowns can be found in Appendix E.

6.2 WRRF Projects

Several improvement projects have been recommended for the WRRF and are summarized in Table 90.

Table 90: Recommended WRRF Projects

WRRF Project	Fiscal Year	Estimated Cost
New Effluent Storage Tank	2025	$7,172,900
Aeration System Improvements	2025	$3,858,900
Effluent Storage Tank Rehab	2026	$1,217,000
Headworks Improvements	2026	$49,000
Headworks Second Screen BDR	2026	$33,600
Headworks Second Screen Installation	2027	$694,300
Aeration Basin Lining	2027	$2,579,000
Aerated Sludge Holding Tanks	2028	$347,700
Sludge Dewatering Improvements	2029	$1,649,200
Secondary Clarifier Mechanism Replacement	2030	$2,447,400
Onsite Hypochlorite System BDR	2030	$97,400
Onsite Hypochlorite System Installation	2031	$1,090,300
Total		**$21,236,700**

6.2.1 Effluent Storage Tank Replacement and Rehabilitation

The existing effluent storage tank is in poor condition and in need of a major rehabilitation effort and possible structural upgrades to meet current seismic codes. The District has initiated the construction of a new effluent storage tank which will be completed in 2025. It is recommended that a budget of **$7,172,900** be allocated in **FY 25** for the new 2-million-gallon effluent storage tank.

Once new storage tank is online, the existing tank could be taken out of service for rehabilitation. This rehabilitation would include recoating the interior and exterior of the tank, performing any miscellaneous repairs or installation of relevant appurtenances, and performing a seismic upgrade retrofit. A budget of **$1,217,000** is recommended to be allocated in **FY 26** for the replacement of the existing tank.

6.2.2 Aeration System Improvements

As described in Section 4.5.5, the existing jet aeration system does not have sufficient capacity to meet the peak organic influent loading. In short, the ratio of recirculation flow to the air flow is not suitable for the existing jet aeration system to maintain the intended fine to medium bubble aeration pattern.

Options for improving or modifying the aeration system could include the following:

- Option 1: Remove the jet aeration system and replace with fine bubble diffusion system.
  - The fine bubble diffuser grids are typically connected to the floor of the aeration basins.
  - Alternatively, the aeration grids can be constructed on retractable grids to allow replacement of the aeration membranes without taking a basin out of service.
  - Initial clean water oxygen transfer efficiency with a fine bubble system can be higher than a jet aeration system. However, the differences are not substantial and need to take into account the potential for diminishing diffuser efficiency over time on account of aging and plugging of the typical membrane diffuser system.
  - A fine bubble system would likely require a higher blower discharge pressure, thereby requiring replacement of the existing blowers.
  - The typical fine bubble diffuser discs have a useful life of approximately eight (8) years.
  - Depending on treatment objectives, separate mixers are typically included to maintain adequate mixing in the basins, especially if it is desired to modulate the aeration for possible denitrification.
  - Note, the ideal depth of aeration basins is closer to 18-20 feet rather than the 12.5 feet of operating depth typical in the existing aeration basins. The tanks were originally designed with turbine surface aerators where this depth was near the limitations of such equipment. Thus, the shallower depth inhibits the efficiency of the fine bubble diffusion systems.
- Option 2: Replace and upgrade the existing jet aeration system with a modern jet aeration system.
  - Provide a design with the appropriate sizing of the blowers, recirculation pumps and jets to maintain a very efficient, fine-bubble aeration pattern. The existing blowers would likely remain in place.
  - Maintain the design with no moving parts in the basin.
  - Mixing is provided through the recirculation/motive pumps with no separate mixers required in the basins.
  - The jet aeration headers and nozzles would be expected to last 20-30 years or beyond with no maintenance or parts to replace.

As part of the effort to evaluate the cause of the current aeration system challenges, and based on the suspicion the blowers, recirculation pumps and jet cluster aeration modules were not all adequately matched to meet the peak aeration demands, a conceptual new jet aeration system was developed. An updated system would resemble the illustration in Figure 89 and include the following revisions to the existing system:

- Each of the “jet cluster” aeration assemblies would be removed and replaced with linear jet aeration assemblies consisting of a water recirculation header, aeration header, jet aeration nozzles, and a header backflush assembly for use in backflushing the headers.

- The existing recirculation/motive pumps are not large enough to provide sufficient flow. Each of these pumps would be replaced with pumps sized for closer to 4,000 gpm and approximate motors sizes of 40 HP. This would require replacement of the existing motor starters and may trigger additional electrical improvements on account of the increased load from six, 25 HP pumps to six, 40 HP pumps, for an additional load of 90 HP if all the pumps were operating.
- The existing blowers appear to have adequate flow and discharge pressure to match a new jet aeration system.
- The existing aeration piping from the blowers to each of the control valves to each tank is relatively new and would stay in place. New sections of drop pipe from the valves down to the jet aeration header would be included.

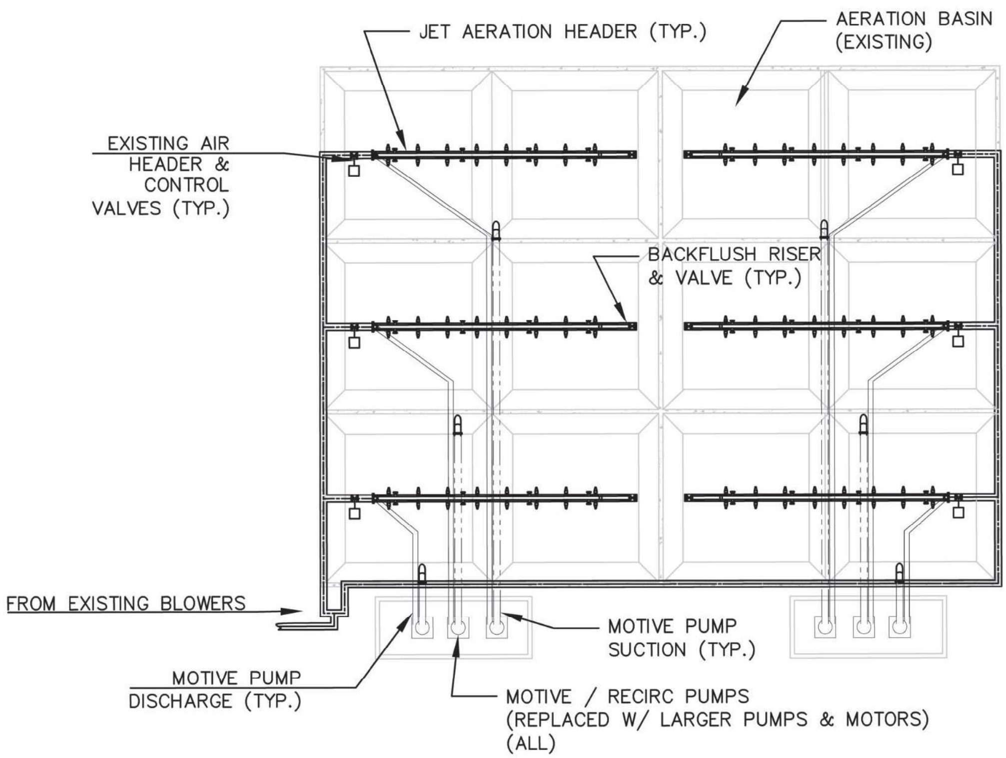

Figure 89: Conceptual Jet Aeration System Replacement

A budget of **$3,858,900** is recommended to be allocated in **FY 25** for the aeration system improvements.

6.2.3 Headworks Improvements

6.2.3.1 Install Recirculation/Chopper Pump in Influent Channel

The proposed submersible chopper pump would function to pull FOG off the surface of the influent flow in the influent channel and resuspend and incorporate the FOG to minimize accumulation in the influent channels and reduce the labor associated with periodically washing out the channels to push the FOG downstream to the aeration basins. The chopper pump would operate on a timer, floats or manually. The oil and grease would not be removed just transferred downstream to the aeration basins and then the clarifiers where the scum scraper mechanisms would skim off the scum and route the scum to the solids handling facilities.

A budget of **$49,000** is recommended to be allocated in **FY 26** for the installation of a recirculation/chopper pump in the influent channel.

6.2.3.2 Add Additional Screen in Second Channel

The concrete influent channels could be modified to accommodate a vertical band screen, or similar, to provide a redundant fine screen in the process. In addition to the band screen to remove debris from the influent, a second piece of equipment for screenings treatment would be required to collect, wash, compact and dewater the collected screenings and direct the material into a garbage receptacle for periodic removal. Considerations of this potential additional would include:

- Consider if a third channel or overflow/bypass provisions should be in place in the event either power was unavailable or both screens had mechanical problems.
- The screen and screenings washer/compactor would need to be enclosed in a heated enclosure to protect the equipment and the utility wash water nozzles and equipment from freezing. The section of the concrete channel that could accommodate the screen is in a difficult location to construct an additional building enclosure.
- The channel may need modified to fit the appropriately sized band or other type of screen.

The preliminary engineering phase will evaluate the mechanical and structural modifications necessary and determine how feasible it would be to incorporate a redundant screen, as described. A budget of **$33,600** is recommended to be allocated in **FY 26** for a BDR of the second influent channel screen, and a budget of **$694,300** in **FY 27** for the installation of the second influent channel screen.

6.2.4 Aeration Basin Concrete Tank Wall Preservation

Capital improvement recommendations for the facility biological aeration basins include monitoring and eventual concrete preservation work to protect the existing concrete. Various options are available for applying a protective coating over the existing concrete to stop further degradation of the walls and floors. Coating options include various epoxy, polyurethane or polyurea coating systems. Each would include a heavy pressure washing or light sand blasting to prepare the surface. Some then incorporate a cementitious mortar material spray or trowel applied to a damp surface in a thickness of up to ½-inch. Following curing of the mortar a layer of epoxy is spray applied to a thickness of 125 mils (+/- 1/8-inch).

An alternative system available includes similar surface preparation followed by application of a multi-layer, 100% solids polyurea spray (OBIC 1000) applied to the concrete surface.

A budget of **$2,579,000** is recommended to be allocated in **FY 27** for the Aeration Basin Lining project.

6.2.5 Aerated Sludge Holding Tanks

The aerated sludge holding tanks (i.e., digesters) are in fair condition but some key improvements are needed. The recommended improvements include replacement or repair of the leaking air line from the blower to the tanks, addition of a second, 60-HP blower for both redundancy and additional capacity to supply the combined aeration demand from this blower to the holding tanks and the grit chamber, and either construction of access platforms or relocation of key air control valves to improve access and operator safety.

A budget of **$347,700** is recommended to be allocated in **FY 28** for the improvements to the aerated sludge holding tanks.

6.2.6 Sludge Dewatering Improvements

The sludge dewatering provisions perform as needed for the facility, but some improvements are needed to maintain reliable operation of the systems. The recommended improvements in order of priority include replacement of the electrical controls, purchase of an available spare rotating assembly and eventual purchase and installation of a new, third centrifuge.

A budget of **$1,649,200** is recommended to be allocated in **FY 29** for the improvements to the sludge dewatering facilities.

6.2.7 Secondary Clarifier Mechanism Replacement

The clarifier mechanisms are at 26 years of life on an expected life of approximately 30 years. The original clarifier mechanisms were recoated after 20 years in 1991 and then replaced at 26 years of age in 1997. Thus, the current mechanisms have reached the end of the previous life of the original clarifiers.

Replacement of the clarifier mechanisms would include removal of the center column, center feed well, walkway, motor and gear box, sludge scraper, scum scraper arm, sludge collection ring, scum box, and overflow weir. The existing clarifiers also have density current baffles to improve settling and reduce washout. These were added in 1991 and could be evaluated for continued use.

A budget of **$2,447,400** is recommended to be allocated in **FY 30** for the Secondary Clarifier Mechanism Replacement.

6.2.8 Effluent Disinfection System

Evaluation of the existing sodium hypochlorite feed system included consideration of the use of on-site hypochlorite generation equipment in place of the liquid chlorine delivered in bulk. This alternative is a tradeoff between the cost of delivery of 12% strength bulk hypochlorite with delivery of salt and use of electricity to generate either 0.8% hypochlorite through typical hypochlorite generation systems or 12% chlorine through a proprietary DeNora system.

Either type of system would require periodic delivery of high-quality salt. Both of systems would also consume approximately the same amount of salt since each pound of salt can only produce approximately one third pound of available chlorine. To provide the same level of redundancy as the existing system, dual hypochlorite generation systems would be required.

The recommended approach is to complete a basis of design report (BDR) first so that the preferred installation alternative could be identified prior to allocating capital budget for the improvements. The BDR will also include a 15% engineering design for the preferred alternative so the improvement and material storage footprints can be better understood, and a class 3 opinion of probable cost can be prepared. A budget of **$97,400** is recommended to be allocated in **FY 30** for the BDR and a placeholder budget of **$1,090,300** is suggested for **FY 31**. This budget should be replaced with the class 3 cost estimate prepared as part of the BDR.

The following information describes what is currently known for the two installation alternatives.

6.2.8.1 Hypochlorite Generation System (0.8% Strength Solution)

Several manufacturers can provide a hypochlorite generation system to produce 0.8% hypochlorite solution. Many systems are in operation in the region and are readily supported through regional equipment representatives. This technology has been in operation for several decades in the United States. This discussion is based upon quotation from Evoqua Water Technologies. The system would include the following major equipment items:

- Brine Tank: Bulk salt would be periodically delivered to the site and blown directly into the brine tank. No other salt storage or provisions to load into the brine tank would be included. If additional “storage” were desired, a second brine tank could be incorporated and rotated between deliveries.
- Brine and potable water would be fed to the sodium hypochlorite generator cell to produce 0.8% hypochlorite. Note, to provide full redundancy, two separate generators are included, both capable of meeting the peak required production capacity of the facility. Similar facilities typically rotate two or more generator systems on a weekly or otherwise periodic basis to ensure either system is ready to operate when needed.
- A hypochlorite storage tank would receive and store the solution.
- Chemical feed pumps would inject the solution into the facility effluent at the same location as the current injection points. Because the solution is lower in strength more volume of the solution must be pumped to the injection point. Thus, the chemical feed pumps would be replaced with larger pumps to provide the same volume of available chlorine to the injection points.
- Hydrogen is produced as part of the generation process. Without appropriate tank venting provisions, the hydrogen would accumulate in the storage tank. Thus,

provisions are included to continuously ventilate the tank and remove the hydrogen to the atmosphere outside of the enclosed chemical feed room space.

- Additional components include level controls and monitoring equipment on the tanks, water softener for equipment cooling water, various drains, ducting provisions for ventilation equipment, control valves, electrical provisions, and control panels.

Figure 84, from Evoqua Water Technologies, illustrates the general equipment arrangement and the required footprint to accommodate the equipment. It would be possible to install the brine tank outside with available insulation and heat tracing provisions. The tank(s) would need to be on the north side of the building for easy access for salt delivery and loading into the brine tank.

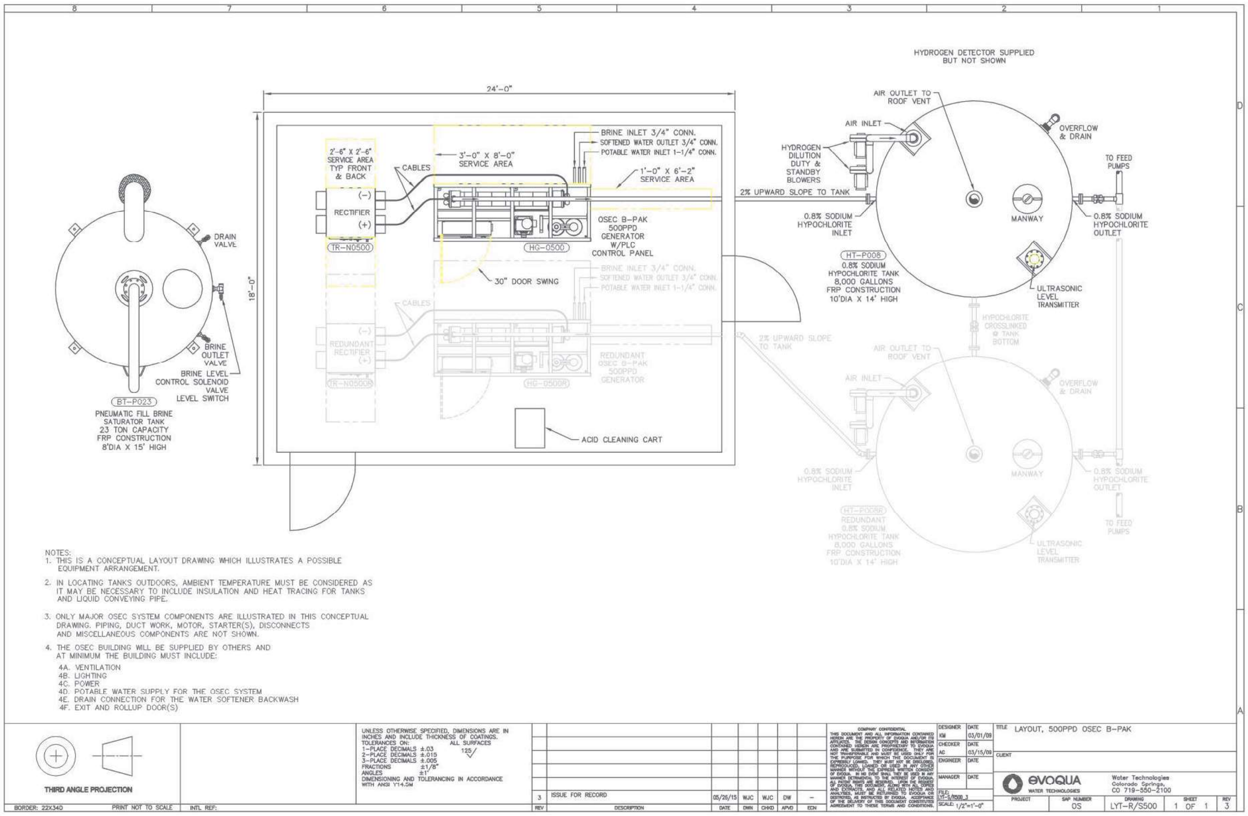

Figure 90: Evoqua Hypochlorite Generation System

6.2.8.2 Hypochlorite Generation System (12% Strength Solution)

The Japanese manufacturer, DeNora, is the only manufacturer in the world that can provide a system to produce the 12% strength hypochlorite. There are currently no installations in the United States, and the level of available operations and maintenance support is without reference at this time. This solution would need to be developed further as part of the BDR to provide a representative comparison to the 0.8% solution.

6.2.8.3 Hypochlorite Generation System Comparison

Table 91 provides a preliminary comparison of the two alternative strength hypochlorite generation systems to help assess if further evaluation of either type of system is warranted.

Table 91: Comparison of Hypochlorite Generation Systems

Item	Evoqua (0.8% Cl2)	DeNora (12% Cl2)
Footprint of Scope of Supply	24' x 18'	60' x 45'
Redundancy Provided within Footprint Noted	100% redundancy	0% redundancy
Salt Use at Max Production	750 lb/day	750 lb/day
Potable/Softened Water Use	3,750 gal/day	7,600 gal/day
Electricity Use	500 kw*hr/day	635 kw*hr/day
Salt Storage	30 days; Salt supply pneumatically blown into brine tank; No other on-site salt storage proposed	Could be the same; Brine tank by owner to figure out.
Use existing feed pumps	Replace with larger pumps	Yes
Means of loading salt into brine tank (dissolver)	Salt supplier to provide pneumatic loading equipment to connect to supplied brine tank	By others
Items Not in Manufacturer Scope of Supply		"Salt dissolver" (i.e., brine tank) Water softener NaOH storage tank NaOH Infilling Pumps (2) HCl Storage Tank HCl Infilling Pump Chiller Unit N2 Gas Cylinder Hypo storage tank(s)
US Municipal Installations	Many	None
Base Scope of Supply Cost	$344,000	$700,000

6.3 Sewer System Condition Assessment

During the preparation of the master plan, it was discovered that PACP scoring for sewer system assets does not exist. This project would include the following activities so that a comprehensive condition assessment could be completed, and all sewer system assets could be assigned an overall risk score.

6.3.1 Review of Existing Sewer System Video Gravity Sewer CCTV & Manhole Inspection Program

IVGID shall implement a program to inspect the remaining two thirds of the gravity sewer mains in the system. This inspection will be for mains6 inches in diameter or greater with a video camera device and have all assets be given a PACP score by a NASSCO certified technician. Additionally, all sewer manholes along the inspection route shall also be inspected and scored. This will occur over a period of six years, with a recommended budget of **$394,000** (in 2023 dollars) be allocated **Annually** from **FY 26** to **FY 31**.

It is also recommended that upon completion of the condition assessment, the District adopt inspection program in perpetuity, inspecting one tenth of the system every year. It is recommended that a budget of **$354,400** (in 2023 dollars) be allocated **Annually** beginning in **FY 35**.

6.3.2 Sewer System Condition Assessment

Once the entire system has been inspected and proper PACP scoring applied, it is recommended that the District use the acquired data to update the condition assessment that is a part of this Plan. It is recommended that a budget of **$105,000** be allocated in **FY 32** to complete the Sewer System Condition Assessment.

6.4 Sewer System SCADA Master Plan and Upgrades

Originally a part of the Water and Sewer Master Plan(s), the Supervisory Control and Data Acquisition (SCADA) system has been identified as an asset in need of evaluation. This project will provide IVGID with a comprehensive SCADA master plan including assessment findings, engineered recommendations, and budgetary estimates. The SCAD Master Plan will include an in-depth assessment of all facilities and automation assets including a summarization of the district's current and future operational objectives.

A budget of **$91,600** is recommended to be allocated in **FY 25** for the Sewer System SCADA Master Plan. A budgetary amount of **$100,000** (in 2023 dollars) has been added to the CIP for **FY 26**, **FY 27**, and **FY 28** for possible system upgrades. This annual amount should be considered preliminary, and the CIP be updated upon completion of the SCADA Master Plan

6.5 Sewer Pump Station Condition Assessment and BDR

Of the 19 sewer pump stations operating within the District, over half of them have never had a rehabilitation project, or the last rehabilitation was over 25 years ago. Based on the findings in Section 3.0, many of the pump stations also have capacity issues for one of their components. It is recommended that the District perform a condition assessment on all 19 of their sewer pump stations. This condition assessment will investigate and assess each lift stations electrical, structural, mechanical, and hydraulic components. The assessment will also verify the hydraulic capacity findings of this Plan. One specific item of investigation will be the gravity mains feeding into SPS-9 to determine if the wet well configuration is causing surcharging per the hydraulic model results. Each lift station will then be ranked based on criticality of failure and prioritized.

Once the condition assessment is completed, a BDR for each lift station will then be prepared that will fully scope the rehabilitation effort for each site, provide a 15% design, and prepare a Class 3 cost estimate that will be used to update the CIP.

It is recommended that the District budget **$163,800** in **FY 25** for the Sewer Pump Station Condition Assessment and BDR.

6.6 Sewer Pump Station Rehabilitation Program

Upon completion of the Sewer Pump Station Condition Assessment and BDR, it is recommended that the District act on the findings and begin a Sewer Pump Station Rehabilitation Program. It is recommended that the District tentatively plan to perform one rehabilitation per year unless the findings of the Sewer Pump Station Condition Assessment and BDR recommend otherwise.

It is recommended that a budget of **$250,000** (in 2023 dollars) be allocated **Annually** from **FY 26** to **FY 44** for the Sewer Pump Station Rehabilitation Program. It should be noted that this is a preliminary budget, and that the CIP should be updated once the Sewer Pump Station Condition Assessment and BDR is completed.

6.7 Gravity Sewer Main Investigation

The findings of the hydraulic model showed that three gravity mains in the District system had a capacity deficiency. One of the areas located just upstream of SPS-9 is recommended to be investigated during the Sewer Pump Station Condition Assessment and BDR project. The remaining two are located within the SPS-1 and SPS-8 sewersheds. These two locations show deficient capacity primarily due to extremely flat slope within the hydraulic model. It is recommended that a field survey and investigation take place to determine the actual slopes of the two pipes and the surrounding areas. The model should be updated with these new elevations and new results produced to see if the deficiency is still prevalent.

It is recommended that a budget of **$16,200** be allocated in **FY 25** for the Gravity Sewer Main Investigation.

6.8 SPS-16 Surge Protection BDR

As discussed in Section 5.1, the existing surge tank is in need of inspection and likely replacement. This project proposes to prepare a BDR for the installation of a new surge protection device at SPS-16. The BDR will include transient modeling, alternatives analysis, a 15% site design, a listing of permitting requirements, and a class 3 opinion of probable cost for the proposed improvements.

A budget of **$70,100** is recommended to be allocated in **FY 25** for the SPS-16 Surge Protection BDR.

6.9 SPS-16 Surge Protection Improvements

This project will include the engineering and construction of the recommended alternative proposed in the surge protection BDR. Additionally, it is expected that the costs for inspection and/or removal of the existing surge tank will be included in this project as well.

Because the scope of the improvements is currently unknown, a budget of **$838,800** is recommended to be allocated in **FY 26** for the SPS-16 Surge Protection Improvements. However, this budget and project priority should be updated after the BDR is completed and the class 3 estimate is available.

6.10 Gravity System Replacement and Rehab Program

It is recommended that the District implement a replacement and rehabilitation program for the gravity portion of the sewer collection system. This will begin once the recommended condition assessment has been completed. The condition assessment will produce for a prioritized list of sewer assets that will require attention.

As the total scope of areas that will require replacement or rehab are currently unknown, a preliminary budget of **$2,000,000** (in 2023 dollars) is recommended to be allocated **Annually** starting in **FY 33**. This preliminary budget number should be replaced, and the CIP updated once the condition assessment is completed.

6.11 System Capital Improvement Program

The projects described above have been organized into a 10-year CIP and a Year 11-20 CIP. The projects and their projected future costs are shown in the appropriate fiscal year can be seen in Table 92 and Table 94.

Table 92: Year 1-5 Capital Improvement Program

Project	Type	FY25	FY26	FY27	FY28	FY29
New Effluent Storage Tank	WRRF	$7,172,900				
Aeration System Improvements	WRRF	$3,858,900				
Sewer System SCADA Master Plan	Study/Planning	$91,600				
Existing Sewer Video Scoring	Inspection	$424,500				
SPS-16 Surge Protection BDR	Study/Planning	$70,100				
Sewer Pump Station Condition Assessment and BDR	Study/Planning	$163,800				
Gravity Sewer Main Investigation	Study/Planning	$16,200				
Effluent Storage Tank Rehabilitation	WRRF		$1,217,000			
Headworks Improvements	WRRF		$49,000			
Headworks Second Screen BDR	Study/Planning		$33,600			
Gravity Sewer CCTV & Manhole Inspection Program	Inspection		$440,700			
Sewer Pump Station Rehabilitation Program	Repair/Replacement		$279,600			
Sewer System SCADA Upgrades	Repair/Replacement		$111,900			
SPS-16 Surge Protection Improvements	Repair/Replacement		$838,800			
Gravity Sewer CCTV & Manhole Inspection Program	Inspection			$457,400		
Sewer Pump Station Rehabilitation Program	Repair/Replacement			$290,300		
Sewer System SCADA Upgrades	Repair/Replacement			$116,100		
Headworks Second Screen Installation	WRRF			$694,300		
Aeration Basin Lining	WRRF			$2,579,000		
Gravity Sewer CCTV & Manhole Inspection Program	Inspection				$474,800	
Sewer Pump Station Rehabilitation Program	Repair/Replacement				$301,300	
Sewer System SCADA Upgrades	Repair/Replacement				$120,500	
Aerated Sludge Holding Tanks	WRRF				$347,700	
Gravity Sewer CCTV & Manhole Inspection Program	Inspection					$492,800
Sewer Pump Station Rehabilitation Program	Repair/Replacement					$312,700
Sludge Dewatering Improvements	WRRF					$1,649,200
Yearly Total		$11,798,000	$2,970,600	$4,137,100	$1,244,300	$2,454,700

Table 93: Year 6-10 Capital Improvement Program

Project	Type	FY30	FY31	FY32	FY33	FY34
Gravity Sewer CCTV & Manhole Inspection Program	Inspection	$511,600				
Sewer Pump Station Rehabilitation Program	Repair/Replacement	$324,600				
Secondary Clarifier Mechanism Replacement	WRRF	$2,447,400				
Onsite Hypochlorite System BDR	Study/Planning	$97,400				
Gravity Sewer CCTV & Manhole Inspection Program	Inspection		$531,000			
Sewer Pump Station Rehabilitation Program	Repair/Replacement		$337,000			
Onsite Hypochlorite System Installation	WRRF		$1,090,300			
Sewer Pump Station Rehabilitation Program	Repair/Replacement			$349,800		
Sewer System Condition Assessment	Study/Planning			$105,000		
Sewer Pump Station Rehabilitation Program	Repair/Replacement				$363,100	
Gravity System Replacement and Rehab Program	Repair/Replacement				$2,904,100	
Sewer Pump Station Rehabilitation Program	Repair/Replacement					$376,900
Sewer Master Plan Update	Study/Planning					$339,200
Gravity System Replacement and Rehab Program	Repair/Replacement					$3,014,500
Yearly Total		**$3,381,000**	**$1,958,300**	**$454,800**	**$3,267,200**	**$3,730,600**