WATERWORKS
ENGINEERS

REDWAY COMMUNITY SERVICES DISTRICT

DRAFT WATER AND WASTEWATER SYSTEMS CAPACITY ANALYSIS

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Executive Summary

The Redway Community Services District (RCSD, or District) owns and operates water and wastewater infrastructure serving the community of Redway, which has a total service population of approximately 1,250 persons, 675 residential connections and approximately 15 commercial connections. RCSD solicited Water Works Engineers assistance in determining the capacity of its water and wastewater infrastructure, in order to assess whether or not additional significant connections can be made to the system.

The water and wastewater systems owned and operated by RCSD, and included in this analysis, were as follows:

Water Systems:

- Infiltration Gallery and Intake Pump Station
- Water Treatment Plant
- Water Storage
- Water Distribution System

Wastewater Systems:

- Collection System
- Wastewater Treatment Plant

Water demand and wastewater flow records were provided by RCSD to Water Works Engineers for determination of current demands on the water and wastewater systems. The capacity of the various water and wastewater systems has been determined based on treatment capacity, hydraulic capacity or other driving factors depending on the component, such as fire flows.

In general, the District does not have substantial, additional capacity at this time, for the following primary reasons:

- The Water Treatment Plant is essentially already operating at its full treatment capacity.
- **There are portions of the water distribution system which currently experience low pressures (< 20 psi) during fire flows, and the existing storage tank serving the Meadows Industrial Park may not be sufficient to provide fire flows for future businesses in the Park, depending on the type of business, size and designation of building, and the Fire Chief's assessment of required fire flow.**

- The existing wastewater treatment plant may not be capable of complying with what is believed to be a forthcoming effluent discharge limit for nitrate of 10 mg/L.

The analysis supporting these findings are detailed further herein.

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1. Introduction

The Redway Community Services District (District) owns and operates the Redway water and wastewater systems. The water system consists of an intake system on the South Fork of the Eel River, a surface water treatment facility, approximately 25 miles of distribution system pipeline, and 835,000 gallon of finished water storage tankage. The wastewater system consists of a gravity collection and partial pressurized forcemain system (four satellite lift stations and one terminal), a wastewater treatment facility and percolation ponds upland of the South Fork of the Eel River. As of 2014, there were 675 service connections to the systems, with a total population of nearly 1,250 persons. There are approximately 15 commercial establishments, and two summer camps in the area.

Recently, the District received inquiries regarding its ability to annex an approximate 50-unit residential development, a commercial microbrewery, and a commercial distillery. The purpose of this report is to access the District's water and sewer infrastructure to determine each system's capacity and ability to service these additional connections. The following infrastructure is included in this evaluation:

Water:

- Raw water infiltration gallery and pump station on the South Fork of the Eel River
- Surface Water Treatment Plant
- Water Storage System (for both fire and Maximum Day Demand)
- Water Distribution System (for both fire and Maximum Day Demand)

Wastewater:

- Wastewater Collection System
- Wastewater Treatment Plant
- Wastewater Effluent Disposal System

1.1. Project Location

The locations of the primary components of the District's water and wastewater system are shown in Figure 1.

Figure 1. Major Components of the Water and Wastewater Systems

2. Water System Evaluation

2.1. Existing Water Demand

Based on monthly flow data from 2011 to June 2014, the District's current average day demand (ADD) is 208,000 gallons or 0.208 mgd. With an approximate 675 connections on the system, this equates to an average of 308 gallons per connection per day. The current maximum day demand (MDD) is estimated at 493,000 gallons or 0.493 mgd (342 gpm). This equates to approximately 731 gallons per connection per day. At a population of 1,250 this equates to a per capita average day demand of 166 gallons per capita per day (gpcd), and a maximum day demand of 394 gpcd. The current peak hour demand (PHD) is estimated at 514 gpm. A peaking factor of 1.5 times the MDD was used. These demands are summarized in Table 1.

Table 1. Redway Community Service District's Water Demand

Period	Demand		Per Capita Demand
	mgd	gpm	gpcd
Average Day Demand (ADD)	0.208	145	167
Maximum Day Demand (MDD)	0.494	342	394
Peak Hourly Demand (PHD)	-	514	-

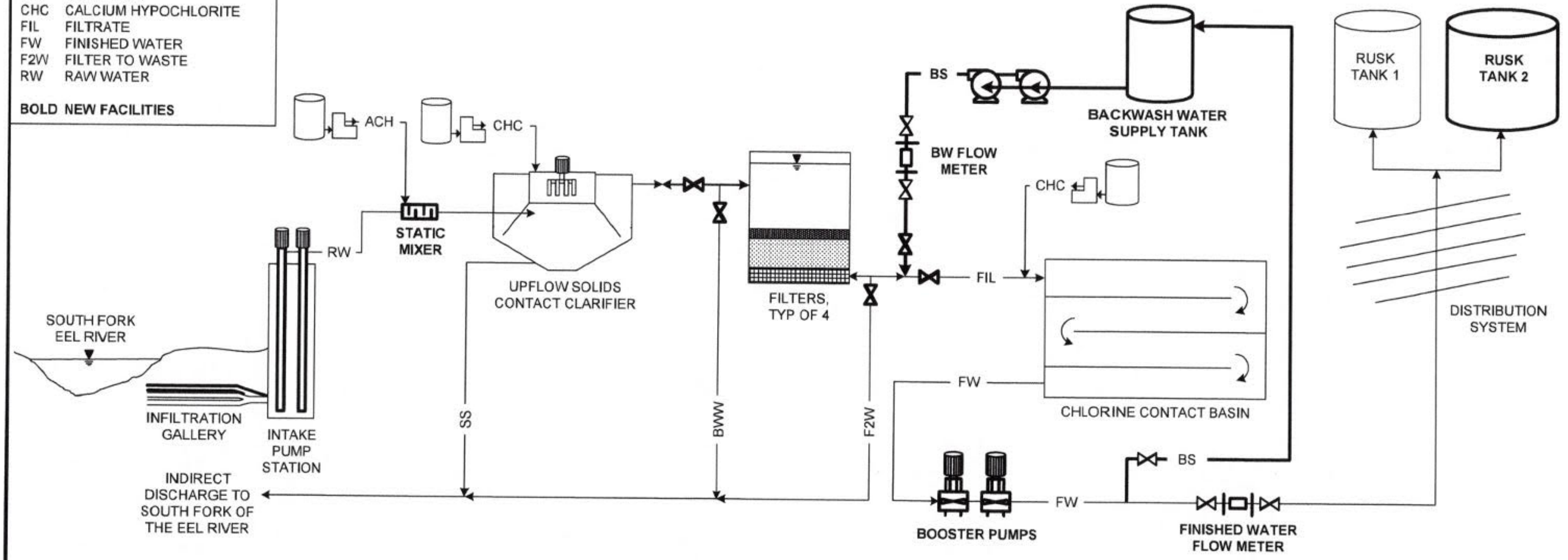
2.2. Water System

The WTP treats water from the South Fork Eel River and distributes it to the residents of Redway, California. The surface water treatment plant consists of an infiltration gallery, a raw water pump station, pretreatment chemical feed systems, a solids contact clarifier, four gravity filters, a backwash storage and supply system, two tablet chlorination systems, a chlorine contact tank, and a finished water pump station. The process flow diagram and a summary of design criteria for the treatment plant is provided in Figure 2 on the following page.

LEGEND:

ACH	ALUMINUM CHLOROHYDRATE
BS	BACKWASH SUPPLY
BW	BACKWASH
CHC	CALCIUM HYPOCHLORITE
FIL	FILTRATE
FW	FINISHED WATER
F2W	FILTER TO WASTE
RW	RAW WATER

BOLD NEW FACILITIES



DESIGN DATA SUMMARY

PEAK WINTER PLANT CAPACITY	365 GPM
PEAK SUMMER PLANT CAPACITY	450 GPM
INTAKE PUMPS	
NUMBER	2
TYPE	VERTICAL TURBINE
DESIGN CAPACITY	450 GPM
TOTAL DYNAMIC HEAD	85 FT
MOTOR SIZE	15 HP
DRIVE	VARIABLE SPEED
COAGULATION	
CHEMICAL	ACH
FEED PUMPS	DIAPHRAGM
DESIGN DOSE	15 MG/L
PRECHLORINATION	
CHEMICAL	CHC
FEED PUMPS	DIAPHRAGM
DESIGN DOSE	1 MG/L

RAPID MIXING	
TYPE	STATIC MIXER
SIZE (INCH)	8
FLOCCULATION AND SEDIMENTATION	
TYPE	SOLIDS CONTACT
TOTAL VOLUME	26,000 GAL
CLARIFICATION ZONE SURFACE AREA	365 FT^2
PEAK SUMMER LOADING RATE	1.5 GPM/FT^2
PEAK SUMMER TREATMENT CAPACITY	548 GPM
PEAK WINTER LOADING RATE	1.0 GPM/FT^2
PINK WINTER TREATMENT CAPACITY	365 GPM
FILTRATION	
NUMBER	4
TYPE	MIXED MEDIA
ANTHRACITE LAYER	3.5"
SAND LAYER	24"
UNDERDRAIN TYPE	GRAVEL
VESSEL DIAMETER	10.5 FT

FILTRATION (CONTINUED)	
FILTER SURFACE AREA (EACH)	86.6 FT
FILTER SURFACE AREA (TOTAL)	346.4 FT^2
DESIGN LOADING RATE	2 GPM/FT^2
MAX TREATMENT CAPACITY	693 GPM
MAX FIRM TREATMENT CAPACITY	520 GPM
BACKWASH SUPPLY	
TANK VOLUME	20,000 GAL
TANK HEIGHT	16 FT
TANK DIAMETER	16 FT
MATERIAL	BOLTED STEEL
BACKWASH PUMPS	
NUMBER	2
TYPE	END SUCTION
DESIGN CAPACITY	1500 GPM
TOTAL DYNAMIC HEAD	30 FT
MOTOR SIZE (HP)	20
DRIVE	VARIABLE SPEED

DISINFECTION	
CHEMICAL	CHC
FEED PUMPS	DIAPHRAGM
DESIGN DOSE	3.0 MG/L
CHLORINE CONTACT BASIN VOLUME	31,400 GAL
LENGTH TO WIDTH RATIO	40:1
HYDRAULIC EFFICIENCY (T_{10}/T RATIO)	0.53
DESIGN GIARDIA LOG REMOVAL	0.5
DESIGN CHLORINE RESIDUAL	1.0 MG/L
DRY WEATHER TREATMENT CAPACITY	694 GPM
WET WEATHER TREATMENT CAPACITY	507 GPM
BOOSTER PUMPS	
NUMBER	2
TYPE	INLINE TURBINE
DESIGN CAPACITY	420 GPM
TOTAL DYNAMIC HEAD	335 FT
MOTOR SIZE (HP)	60
DRIVE	VARIABLE SPEED

NO.	DATE	REVISION	BY	APVD

DESIGN | DRAWN | CHECKED | APPROVED

WATERWORKS ENGINEERS

REDWAY COMMUNITY SERVICES DISTRICT
WATER SYSTEM IMPROVEMENTS
PROCESS FLOW DIAGRAM AND
DESIGN DATA SUMMARY

DATE DEC 08
PROJECT NUMBER 08-037

FIGURE 2

Raw water flows from the South Fork into the Infiltration Gallery through piping located in a gravel bed under the South Fork of the Eel River. From the Infiltration Gallery, water flows into an exposed wetwell on the bank of the river, and it is pumped up to the treatment plant with submersible well raw water pumps. Raw water is dosed with coagulant (aluminum chlorohydrate (ACH)) and pre-oxidant (chlorine) upstream of the Solids Contact Clarifier. The coagulant particulates dissolved organics and suspended matter in the raw water and the clarifier provides flocculation of particls and settling. Any residual suspended materials are subsequently removed in the filters, which consist of anthracite over sand. After filtration, the water is dosed with chlorine and flows through the Chlorine Contact Tank for disinfection. Two finished water (booster) pumps convey the treated water to the distribution system which includes three water storage tanks.

The design flows for each component of the water system were analyzed to determine the capacity of the water system. Each component and its design parameters are discussed in the following sections.

2.2.1. Water Intake System

The Infiltration Gallery is located in the South Fork of the Eel River and consists of two parallel perforated C-900 PVC laterals. In 2007 and 2008, low flow conditions in the South Fork of the Eel River resulted in insufficient raw water supply to the District's WTP. Therefore, as part of the 2009 WTP Improvements project, these laterals were extended further across the riverbed, and a subgrade claystone berm that was between the end of the laterals and the low-flow channel of the Eel was excavated and replaced with porous layers of gravel and cobble. The lateral extensions are 16" and connect to existing 14" C-900. These laterals are connected by a 14"x14"x12" wye to a 12" PVC pipe that connects the laterals to the Raw Water Pump Station. Since these improvements were made, the District has been able to convey water from the low-flow channel to the WTP in sufficient quantity to serve the system's maximum day demand.

From the Raw Water Pump Station, water is pumped by one of two submersible vertical turbine raw water pumps, through a flowmeter, through the pretreatment system to the upflow clarifier. The design capacity of the pumps is 450 gpm each. Both pumps are equipped with variable frequency drives (VFDs). Key design parameters for the raw water intake and pumps are listed in Table 2.

Table 2. Design Parameters for the Water Intake System

Item	Description
Intake Pump Station	
Wetwell Diameter	6 feet
Wetwell Depth	42 feet
Raw Water Pumps	
Pump Type	Submersible Vertical Turbine
Number of Pumps	2, 1 duty + 1 standby
Duty Point Capacity	450 gpm
Duty Point Total Dynamic Head	94
Pump Horsepower	15 hp
Drive Type	VFD
Wetwell Floor Elevation	261.2'
Minimum Water Surface Elevation	263.0'
Intake Piping Invert Elevation	263.2'

2.2.2. *Water Treatment System*

2.2.2.1. Pretreatment System

The Solids Contact Clarifier is a 26,000 gallon unit, Ondeo Degremont Accelerator® unit, installed when the WTP was first constructed in the 1970's. The unit provides mechanical mixing, coagulation, flocculation, and sedimentation within a single tank. The clarifier's treatment process is shown schematically in Figure 3, below.

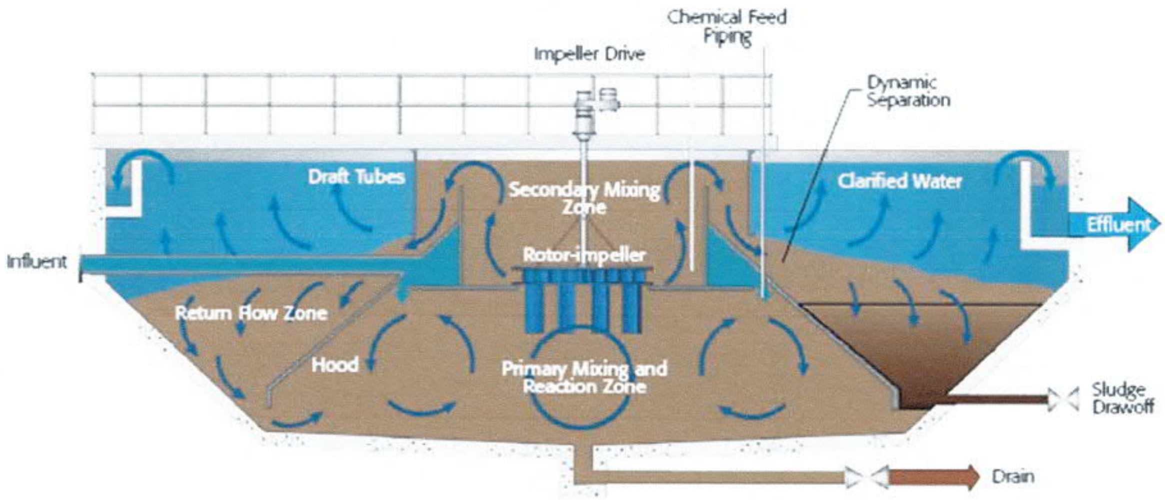


Figure 3. Solids Contact Clarifier Schematic

The primary purpose of the Solids Contact Clarifier is to reduce the amount of dissolved and particulate natural organic matter (color and turbidity) in the raw water prior to filtration. Coagulant and calcium hypochlorite are added into the static mixer upstream of the clarifier, which was installed as part of the WTP Improvements project in 2009. The mixer was designed for optimal operation across the typical operating flow range of 250 to 450 gpm.

High solids concentrations are maintained in the primary mixing and reaction zone in the clarifier to increase the number of collisions of particles and enhance flocculation. The water then flows through the more-gently-mixed secondary mixing zone and into the clarified water zone where the solids settle out. Settled sludge flows down the outside of the hopper shell and is periodically removed from the tank via a manual drain valve. Some of the solids are recirculated to the primary mixing and reaction zone. Clarified water rises to the surface outside the flocculation well where it is collected by submerged orifices in effluent troughs and flows into the filter feed piping.

Upflow Clarifier Treatment Capacity

The Solids Contact Clarifier was originally designed to provide one (1) hour of detention time at a flow of 440 gpm (0.63 mgd). This is appropriate for typical wet weather demand flows, which are typically between 225 and 330 gpm. The wet-weather season is when the Solids Contact Clarifier performance is most critical, as the turbidity in the South Fork of the Eel can spike during runoff events to 200 NTU or greater.

Summer flows (407 to 552 gpm) exceed the optimal capacity of the Solids Contact Clarifier, and the California Department of Public Health raised concerns about the upflow clarifier's treatment capacity prior to the 2009 WTP Improvements project. However, in the summer the Eel River's raw water quality is usually very good. Quiescent conditions in the river and the fact that algal blooms are rare in the upper South Fork of the Eel result in typical raw water turbidities of 0.5 NTU or less. The primary purpose of the Solids Contact Clarifier under these conditions is to provide reaction time for the ACH with dissolved natural organic matter (color) prior to chlorination, in order to minimize the formation of disinfection byproducts. The reaction time of ACH is on the order of seconds, but a minimum flocculation time of 15 minutes should be provided for contact of the floccs with dissolved organics. The clarifier provides a contact time of nearly three times this even at peak summer flow.

The general design guide for settling of aluminum-based coagulation of raw surface water is 1 gallon per minute per square foot (gpm/ft^2)of clarifier settling area per minute. However, higher areal hydraulic loading rates may be treatable, depending on raw water quality and downstream treatment facilities. The District's operators do not report any issues with the clarifier's higher summer throughput, and past performance supports a higher maximum loading limit of 1.5 gpm/ft^2. This information was conveyed to CDPH in 2007, and they approved the proposed seasonal approach treatment capacity for the upflow clarifier. Expansion of clarifier capacity at that time would have been expensive (on the order of $0.5 million) and unnecessary.

Design parameters for the Pretreatment System are listed in Table 3.

Table 3. Design Parameters for the Pretreatment System

Item	Description
Static Mixer	
Type	Stationary Element Static Mixer
Diameter	8 in
Length	54 in
Number of Elements	4
Headloss at 250 gpm	1.2 ft (0.5 psi)
Headloss at 450 gpm	4.6 ft (2.0 psi)
Chemical Injection Ports	2" NPT
Solids Contact Clarifier	
Total Volume	26,000 gallons
Clarification Zone Surface Area	365 ft^2
Peak Summer Loading Rate	1.5 gpm/ft^2
Peak Summer Treatment Capacity	548 gpm
Peak Winter Loading Rate	1 gpm/ft^2
Peak Winter Treatment Capacity	365 gpm

2.2.2.2. Filtration

The Redway WTP has four 86.5 ft^2 sand and anthracite filters, for a total filter surface area of 346 ft^2. The media within the filters consists of anthracite, sand, and stratified support gravels. The media within the filters consists of 3" of anthracite (effective size 0.85 to 0.90 mm, uniformity coefficient 1.72) over 24" of sand (effective size 0.4 to 0.55 mm, uniformity coefficient 1.75), over 15" of stratified support gravels. The filters are shown in Figure 4 on the following page.

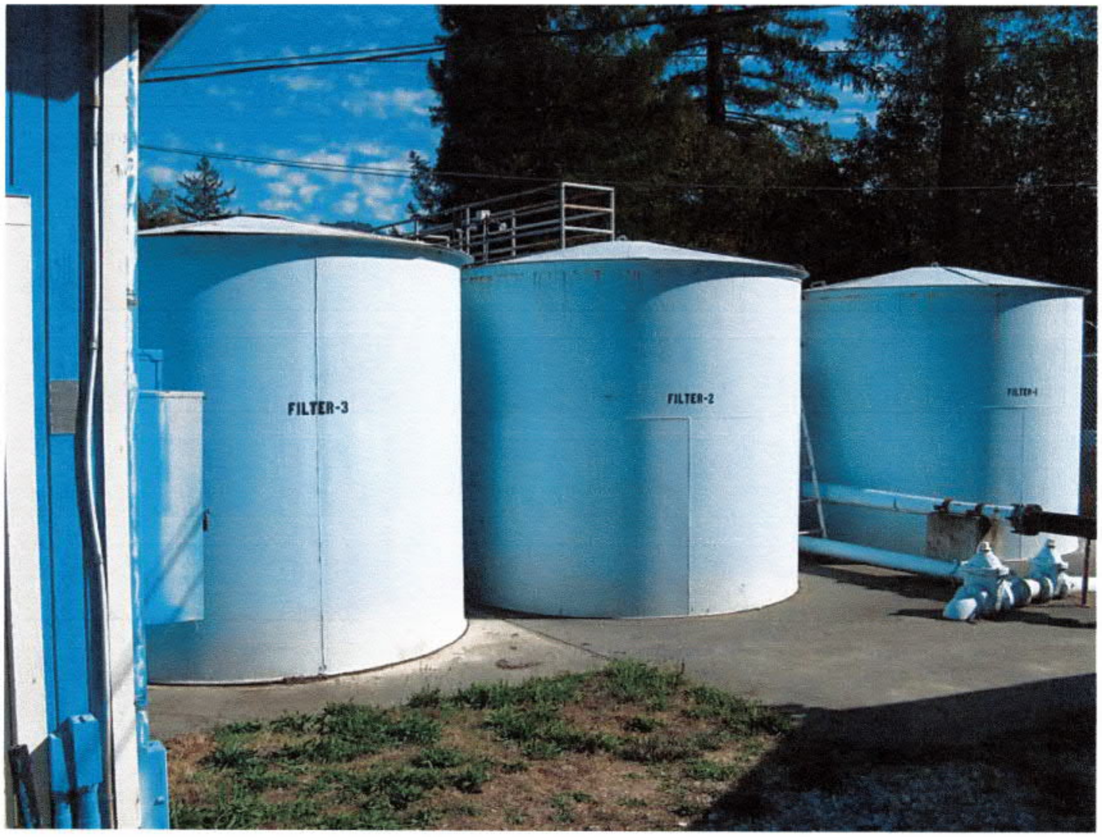


Figure 4. Rapid Sand Filters

The design surface loading rate for the filters is 2.0 gpm/ ft^2. At this loading rate the forward flow through the filters is 692 gpm with all filters in service, and 522 gpm with three filters in service. Allowing for 10% backwash waste, the firm capacity (capacity with one unit out of service) of the filters is 470 gpm (0.68 mgd). The design parameters for the filters are listed below in Table 4.

Table 4. Design Parameters for the Filtration System

Item	Description
Rapid Sand Filters	
Number	4
Vessel Diameter	10.5 ft
Filter Surface Area, Each	86.6 ft^2
Filter Surface Area, Total	346 ft^2
Design Loading Rate	2 gpm/ft^2
Media Configuration	
Anthracite	3 inches
Sand	24 inches
Support Gravels	15 inches
Maximum Treatment Capacity	693 gpm
Maximum Firm Treatment Capacity (3 filters in service and 10% backwash waste)	470 gpm

The filters were not altered in the 2009 WTP Improvements Project. The piping and valving around them was upgraded, but this was a modification made to improve operation and performance rather than capacity.

Backwash Supply Storage and Pump Station

A new backwash storage and supply system was constructed as part of the 2009 WTP Improvements, consisting of a 36,000 gallon supply tank and two backwash supply pumps.

The backwash supply tank is an 18-ft diameter bolted steel tank 22-ft in height. The backwash supply tank is fed off a 3" pipe from the finished water main downstream of the Booster Pump Station. The finished water piping is routed through the side of the tank and discharges above the maximum water level to provide an air gap between potable water the backwash supply system.

There are two backwash supply pumps, located directly west of the backwash supply tank. Design criteria for the backwash supply pumps are listed in Table 5.

Table 5: Backwash Supply Pump Criteria

Item	Description
Backwash Supply Pumps	
Pump Type	End Suction Centrifugal
Number of Pumps	2, 1 duty + 1 standby
Duty Point Capacity	1730 gpm
Duty Point Total Dynamic Head	24-feet
Pump Horsepower	20 hp

2.2.2.3. Disinfection

Calcium hypochlorite from a tablet feed system is used for disinfection at the entrance to the Chlorine Contact Tank. The Chlorine Contact Tank is a 31,000 gallon 4-pass serpentine concrete tank. The contact tank is 12 ft (width) x 8 ft (depth) x (40) ft length. The tank is baffled into 4 passes, each 3 feet wide. The total length to width ratio is 40:1. The total length to depth ratio is 15:1. Based on the California Department of Public Health's hydraulic modeling database of chlorine contact tanks, the basins hydraulic efficiency (T_{10}/T ratio) is conservatively projected at 0.53.

For surface water treatment plants using conventional filtration, the disinfection system must provide a minimum of a 0.5 log *Giardia* inactivation. The kinetics of *Giardia* inactivation by chlorine is dependent upon temperature and pH. In the winter, the coldest temperature of the Eel River water is 6.8 deg C and the highest pH is approximately 8.0. Assuming a minimum chlorine residual of 1.0 mg/L is maintained at the discharge end of the contact basin, these conditions require a minimum actual contact time "T_{10}"of 32 minutes. Adjusting for hydraulic inefficiency, the minimum theoretical contact time becomes 58 minutes (32 min divided by 0.53). The contact basin's volume will provide 58 minutes of contact time at flows up to 760,000 gpd (0.76 mgd).

In the high demand period of the summer the minimum water temperature is approximately 17 deg C, and the maximum pH is 8.3. Assuming a minimum chlorine residual of 1 mg/L at the contact basin discharge, the District's disinfection system must provide a minimum "T_{10}" contact time of 17 minutes. Adjusting for hydraulic inefficiency, the minimum theoretical contact time becomes 32 minutes (17 min divided by 0.53). The contact basin's volume will provide the necessary 32 minutes of contact time at flows up to 1.4 mgd.

The design parameters for the disinfection system are summarized in Table 6 on the following page.

Table 6. Disinfection System Design Criteria

Item	Description
Disinfection	
Chemical	Calcium Hypochlorite
Feed System	Tablet
Design Dose	3.0 mg/L
Chlorine Contact Basin Volume	31,400 gal
Length To Width Ratio	40:1
Hydraulic Efficiency (T_{10}/T Ratio)	0.53
Design Giardia Log Removal	0.5
Design Chlorine Residual	1.0 mg/L
Dry Weather Treatment Capacity	972 gpm
Wet Weather Treatment Capacity	527 gpm

2.2.2.4. Finished Water Pump Station

The treated water is conveyed to the distribution system via two (one duty, one standby) finished water (booster) pumps. Both submersible vertical turbine pumps are equipped with variable frequency drives. The design capacity for each pump is 450 gpm at 352 feet of head. Design Criteria for the finished water (booster) pumps are listed in Table 7.

Table 7. Design Criteria for the Finished Water Pump Station

Item	Description
Finished Water Pump Station	
Wetwell Diameter	1-foot
Wetwell Depth	11.25-feet
Finished Water (Booster) Pumps	
Pump Type	Submersible Vertical Turbine
Number of Pumps	2, 1 duty + 1 standby
Duty Point Capacity	450 gpm
Duty Point Total Dynamic Head	352-feet
Pump Horsepower	50 hp
Drive Type	VFD
Pump Manufacturer	Grundfos

2.2.3. Water Storage Tanks

The Redway Community Services District has three storage tanks; the Meadows Industrial Park tank and the two adjacent Rusk Avenue tanks. The Meadows Industrial Park tank is 100,000 gallons. The original Rusk Avenue Tank is 275,000 gallons. A second Rusk Avenue storage tank was included in the 2009 WTP Improvements project, and has a nominal capacity of 460,000 gallons, see Figure 5. The total storage capacity provided by the three tanks is 835,000 gallons.

Figure 5. The New 460,000 gallon and Original 275,000 gallon Rusk Avenue Tanks.

The required storage capacity for municipal distribution systems is the maximum day demand plus fire flow. The current maximum day demand is 494,000 gallons. Fire flow requirements vary, depending on the methodology utilized. The International Fire Code (IFC) and the National Fire Protection Agency (NFPA) both have common recommendations, but they are not requirements unless adopted by the Authority Having Jurisdiction (AHJ), which in the District's case would be the Redway Fire Protection District's Fire Chief. However, for commercial buildings, it is recommended that 1,500 gpm or greater be available at 20 psi or greater, for a minimum period of two hours, which equates to 180,000 gallons. Therefore, the minimum recommended storage capacity for the District as defined above is 674,000 gallons. By this metric there is adequate storage capacity in the system; however, due to elevation differences and lack of booster pumping not all of this capacity is available throughout the entire system. This will be discussed in more detail in Section 2.2.4.

The water for Meadows Industrial Park is supplied by the 100,000 gallon Meadows Industrial Park tank, which is located east of Highway 101. This tank is fed by the Evergreen Booster Station which is a 50 gpm booster station that lifts water from the primary pressure zone west of Highway 101, up to the Meadow Industrial Tank. The 100,000 gallon tank and 50 gpm pump station potentially does not provide enough storage, flow and pressure to meet the recommended fire flow demand for the Industrial Park (180,000 gallons, see discussion in preceding paragraph). In order to meet the minimum recommended fire flow requirements for the Industrial Park, approximately 80,000 gallons of additional storage would need to be constructed, or a fire pump would need to be installed in parallel with the existing Evergreen Booster Station. The fire pump would need to be capable of pumping a minimum of 1,500 gpm from the primary pressure zone up to the Industrial Park.

2.2.4. Distribution System

The District maintains approximately 25 miles of distribution system piping ranging in size from 1 ½ inches to 10 inches and consisting of ductile iron, asbestos cement, and PVC pipe. There are also two pressure reducing stations, the Oakridge Pressure Reducer and the Orchard Pressure Reducer stations. The location of the reducer stations, storage tanks, and booster stations are shown in Figure 6.

A basic distribution system model was compiled by Spencer Engineering in 2006 using EPANET 2, which is public domain software developed by the U.S. Environmental Protection Agency (US EPA). EPANET 2 is a simple hydraulic modeling program that tracks the flow and pressure of water within a piping network. The model compiled by Spencer Engineering consisted of the existing Rusk Ave. storage tank, 39 pipes, and 30 nodes. The model was an over-simplification of real-world conditions, as the base flow rate utilized (330 gpm), was

Figure 6. Schematic Map of RCSD Distribution and Storage Systems

distributed evenly across all the nodes in the system. In addition, the Meadows Industrial Park area was not included in the model.

However, the model was still useful as a conceptual look at pressures in the system during fire flows, as the fire flow is input at a specific location in the model and the base flows become negligible. The fire flow model indicated that the lowest water pressures occur at closest to the Rusk Ave storage tank location, and the highest pressures occur down gradient at those nodes nearest the Eel River, which is in agreement with the District's observations. This problem is most pronounced for connections located along Rusk Avenue and Murrish Road, where the static pressure provided by the storage tank is the least.

The model showed that the pressure at some locations drops below 20 psi during fire flows (2500 gpm). This problem is most pronounced for those users located along Rusk Avenue and Murrish Road, where the static pressure provided by the storage tanks is least.

Initially the District considered upsizing the existing 10 and 12 inch transmission pipeline from the Rusk Ave Tank to Empire Ave. However, according to Spencer Engineering this approach would not result in significantly higher pressures during fire flows. It would also be difficult and costly to construct – trenchless construction would be required to install the portion of the pipeline that is underneath the commercial zone on Redwood Dr. A second approach was considered by the District entailing the installation of a second 10-inch transmission line from the storage tanks down to the first junction at Rusk Avenue and the installation of a 6-inch loop along Rusk Avenue that would connect the main 10-inch line leaving the storage tank with the existing 4-inch line on Murrish Road. Addition of this approximately 775-ft line would create a loop to the east of Redwood Drive that would decrease pressure losses throughout the system.

These improvements are shown in Figure 7, and were modeled under fire flow conditions. Adding both the 6-inch loop and the second 10 inch pipe from the tank has an additive effect on the system pressures during fire flow, with the greatest effect observed at the top of Murrish Road. Adding the second 10-inch line has a more significant effect on pressures throughout the distribution system, and adding the 6 inch line between Rusk Avenue and Murrish Road will provide an additional increase in pressures during fire flow in that portion of the distribution system.

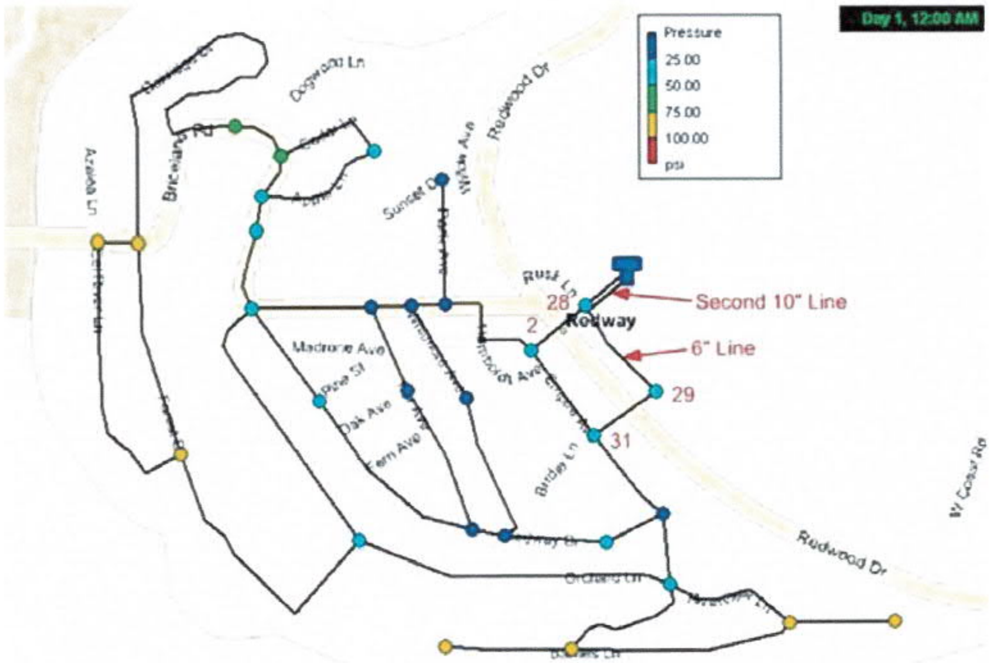


Figure 7. EPANET 2 Fire Flow Model of RCSD Distribution System (Spencer Engineering, 2007) with System Improvements

The impact of additional connections on the low pressures during fire flow will depend on the location of the connections and the additional demand placed on the system by the new connections. Quantifying the impact in terms of pressure would require a thorough distribution system modeling analysis that would map all components of the distribution system including pipes, valves, tanks, booster pumps, pressure reducing stations, etc. This would cost approximately $30,000 for a system in the District's size range.

However, it is clear that substantial additional demand should not be placed on the system until the low pressures during fire flows are addressed, as this additional demand would, in all likelihood, contribute to the problem.

2.3. Summary of Water System Capacity

The current Average Day Demand, Maximum Day Demand, water treatment unit capacities, storage and distribution system components are all summarized in Table 8.

The water treatment plant is limited by the design flows of the water intake pumps and booster pumps, and the pretreatment system to a capacity of 450 gpm. At the current Maximum Day Demand of 494,000 gallons, the the plant has to run 18.3 hours per day at its full output of 450 gpm in order to provide this volume of water. This should be viewed as essentially at capacity, because the plant is not staffed 24 hours per day, and while the bulk of the treatment process is automated, it is not designed for complete un-manned operation. Depending on the demand of significant new connections to the water system, the WTP capacity would need to be increased.

Table 8. Water System Capacity

Parameter	Value
Current Average Day Demand	0.208 mgd / 145 gpm
Current Maximum Day Demand	0.494 mgd / 342 gpm
Water Intake and Pump Station Capacity	450 gpm
Pretreatment Capacity	365 gpm Winter / 548 gpm Summer
Filtration	470 gpm
Disinfection	527 gpm
Booster Pumps	450 gpm
Water Storage	735,000 gallons in residential area 100,000 gallons at Meadows Industrial Park
Distribution System	Inadequate pressure in portions of the system during fire flows

The 100,000 gallon Meadows Industrial Park tank does not provide enough storage to meet the fire flow demands for the Meadows Industrial Park, and the 45 gpm Evergreen booster pumps are insufficient to provide fire flow to up to the tank. In order to meet the fire flow requirements for the Industrial Park, 80,000 gallons of additional storage would have to be added, or a fire pump would need to be installed down off Redwood Drive. The fire pump would have to be capable of pumping 1,500 gpm from the primary pressure zone up to the Industrial Park.

The distribution system does not provide sufficient pressure during fire flows, particularly for users located along Rusk Avenue and Murrish Road, where the static pressure provided by the Rusk Avenue storage tanks is minimized, and the Meadows Industrial Park, where there is insufficient fire flow capacity. It is recommended that a thorough hydraulic capacity analysis which maps all the components of the distribution system including all pipes, valves, tanks, booster pumps, pressure reducing stations, etc. be conducted to pinpoint the exact locations of the deficiencies in the distribution system, and to develop the most economical approaches to addressing the issue.

It is the opinion of WWE that water treatment system is currently at capacity, and the water storage and distribution system is already struggling to meet the demands of existing connections. These issues should be addressed before substantial additional service connections are made.

3. Wastewater System Evaluation

3.1. Existing Wastewater Flows and Loads

Historical flows from January 1, 2013 to present were obtained from the District records. The flows were analyzed to develop the current average flows, peak flows, and resulting peaking factors. The results are shown in Table 9 on the following page.

Table 9. Current WWTF Flows and Peaking Factors

Flow Period	Current Flow	Peaking Factor
Average Dry Weather Flow (ADWF)	0.104 mgd	NA
Annual Average Flow (AAF)	0.130 mgd	1.3
Peak Month Wet Weather Flow (PMWWF)	0.293 mgd	2.8
Peak Day Wet Weather Flow (PDWWF)	0.578 mgd	5.6
Peak Hourly Wet Weather Flow (PHWWF)	600 gpm[1]	8.3

[1]Estimate based on the 2015 design PHWWF provided by SHN Consulting and by Operator Input. Currently there is no hourly flow monitoring.

Wastewater influent five-day biochemical oxygen demand (BOD) and total suspended solids (TSS) are routinely measured at the WWTF. The approximate average BOD_5 and TSS are shown in Table 10.

Table 10. Typical BOD and TSS Ranges

Wastewater Quality	Average (mg/L)	Typical Range (mg/L)
Biochemical Oxygen Demand (BOD_5)	219	190 to 250
Total Suspended Solids (TSS)	206	175 to 235

Average BOD_5 and TSS values were used to estimate the loading that currently occurs at average and peak flows for the WWTF. The results are shown in
Table 11e 11.

Table 11. Current WWTF BOD and TSS Loads

Flow Period	BOD Loading (lb/day)	TSS Loading (lb/day)
Average Dry Weather Flow (ADWF)	191	175
Annual Average Flow (AAF)	238	217
Peak Month Wet Weather Flow (PMWWF)	537	488
Peak Day Wet Weather Flow (PDWWF)	1,060	965
Peak Hourly Wet Weather Flow (PHWWF)	1,717	1,561

3.2. Permitting

The Regional Board adopted a permit for the Redway WWTF, effective September 1, 2011 through August 31, 2016 (NPDES No. CA0022781). The permit details Waste Discharge Requirements (WDRs) for both direct discharge to the Eel River and to the Evaporation/Percolation Ponds. Table 12 lists the limits that are imposed on the WWTF when discharging to the Eel River. There are additional limits when discharging to the Eel River than when discharging to the Evaporation/Percolation Ponds, so the River discharge limitations were used for gauging the capacity of the WWTF.

The WWTP does not currently have total nitrogen limit, but it is anticipated that the Regional Board will impose a total nitrogen limit of 10 mg-N/L in the near future. Therefore, a total nitrogen limit of 10 mg-N/L was used to analyze the future treatment capacity of the plant. Trace compounds such as carbon tetrachloride, chlorodibromoethane, and dichlorobromomethane are not related to the facilities capacity, therefore they were not considered in this report. The WWTF has been able to meet the limits for trace compounds to date.

Table 12. NPDES Permit No. CA0022781 Limits

Parameter	Units	Effluent Limitations				
		Average Monthly	Average Weekly	Maximum Daily	Instantaneous Minimum	Instantaneous Maximum
Biochemical Oxygen Demand (5-day @ 20°C)	mg/L	30	45	60	-	-
	lbs/day	48	71	95	-	-
Total Suspended Solids	mg/L	30	45	30	-	-
	lbs/day	48	71	95	-	-
pH	std units	-	-	-	6.5	8.5
Carbon Tetrachloride	µg/L	0.25	-	0.50	-	-
Chlorodibromomethane	µg/L	0.40	-	0.80	-	-
Dichlorobromomethane	µg/L	0.56	-	1.1	-	-
Chlorine, Total Residual	mg/L	0.01	-	0.02	-	-
Settleable Solids	mL/L	0.1	-	0.2	-	-
Total Coliform Organisms	MPN/100 mL	23	-	230	-	-

3.3. Redway Wastewater System Capacity Analysis

The Redway wastewater system consists of a collection system and the wastewater treatment facility. The hydraulic and treatment capacities of the system were estimated using a combination of spreadsheets and Biowin, a whole-plant and activated sludge modeling software from Envirosim.

3.3.1. Collection System

The District's collection system incorporates both gravity sewer collection and five lift stations, two pipelines underneath the South Fork of the Eel River, and one aerial crossing to connect the Eel River Conservation Camp to the wastewater treatment plant (WWTP). Figure 8 was taken from the 1996 WWTP and Dogwood Lift Station As-Built drawings and shows four of the lift stations. The Evergreen Lift Station, located at the intersection of Redwood Ave and Evergreen Rd, is not shown.

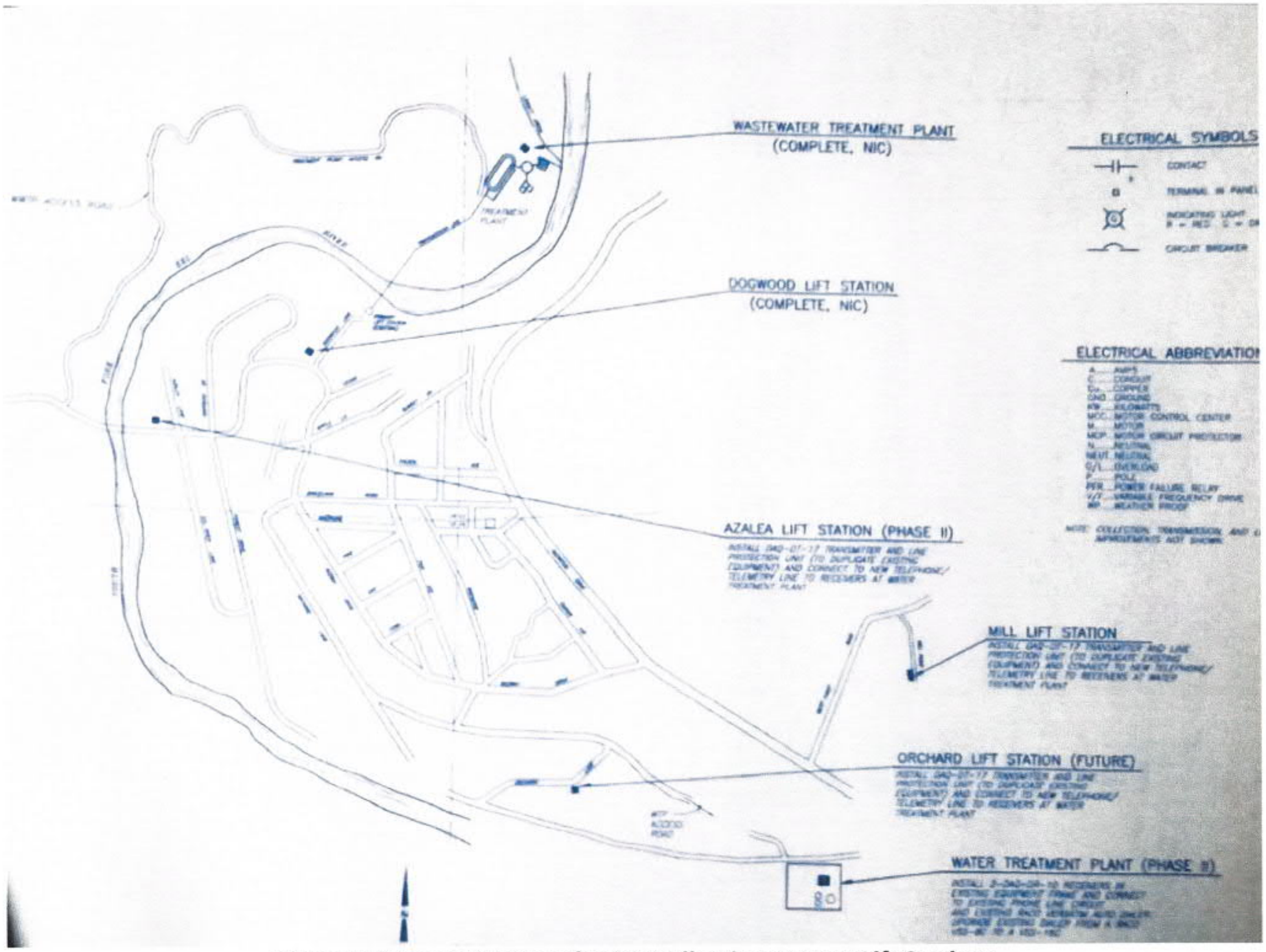


Figure 8. Schematic Map of RCSD Collection System Lift Stations

All sewage generated within the District ultimately reaches the Dogwood primary lift station located at the intersection of the Briceland-Thorne Road and Dogwood Lane. The Mill Lift Station pumps sewage directly to the Dogwood Lift Station. The Evergreen, Orchard and Azalea Lift Stations pump sewage sequentially to the Dogwood Lift Station.

Each intermediate lift station consists of a wet well with duplex, submersible grinder pumps, motors, controls, and a propane fueled generator for backup power. Typically, a single pump can handle the average dry and wet weather sewage flows, but both pumps may be required during peak wet weather periods. The Dogwood Lift Station has four pumps, two low flow 275 gpm pumps and two high flow 600 gpm pumps. Typically, the low flow pumps can handle the average dry and wet weather sewage flows, but during peak wet weather events one of the high flow pumps will often be required.

A schematic flow diagram taken from the 1996 wastewater system improvements project (SHN, 1996) is provided in Figure 9. The diagram shows the Orchard, Azalea and Dogwood Lift Stations, along with an overflow that connects the Dogwood Lift Station wetwell at high level to an older terminal lift station, called the "Existing Primary Lift Station". The diagram also shows the 8" and 6" forcemains beneath the South Fork of the Eel riverbed.

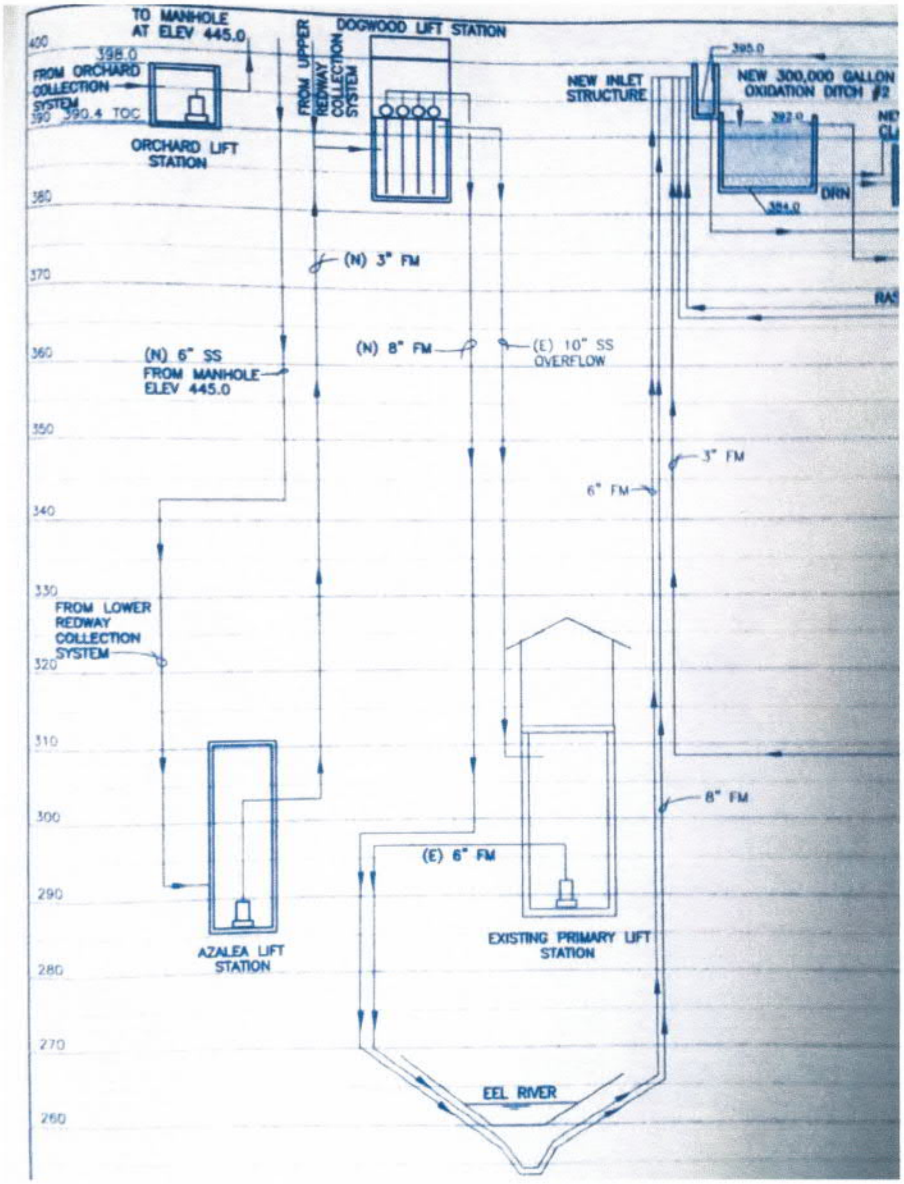


Figure 9. Flow Schematic of RCSD's Primary Lift Stations

The current design capacities for each station are listed in Table 13. The total firm capacity of the terminal Dogwood Lift Station is 600 gpm, approximately equal to the estimated current peak instantaneous flow (600 gpm, See Table 9. However, this appears to be due to the control scheme in use at the pump station – at peak inflow only one of the larger pumps is allowed to operate. The forcemain between the Dogwood Lift Station and the WWTP is 8" diameter. This diameter forcemain could reasonably convey up to 1,200 gpm (velocity would be 7.7 ft/sec). If the older Primary Lift Station and its 6" forcemain are still serviceable, the two stations combined could likely convey as much as 1,800 gpm with new pumps.

Table 13. Lift Station Design Parameters

Parameter	Value
Dogwood Lift Station	
Number of Pumps	4- 2 High Flow, 2 Low Flow
Capacity-High Flow	600 gpm each
Capacity- Low Flow	275 gpm each
Evergreen Lift Station	
Number of Pumps	2
Capacity	50 gpm each
Azalea Lift Station	
Number of Pumps	2
Capacity	80 gpm each
Mill Road Lift Station	
Number of Pumps	2
Capacity	40 gpm each
Azalea Lift Station	
Number of Pumps	2
Capacity	75 gpm each

Further evaluation would be necessary to assess the capacity of the existing satellite stations to receive substantial additional sewage from multiple new residential or commercial connections. It is recommended that the District obtain a map and hydraulic model of the entire collection system, in order to develop specific capacities at specific points of future connections to the collection system. For a collection system of the District's size, this would cost approximately $30,000.

3.3.2. Wastewater Treatment Plant Process Capacity

3.3.2.1. Preliminary Treatment

The only treatment process used at Redway WWTF prior to secondary treatment is grinding of the raw wastewater. This is not done on the WWTF grounds, but at the Dogwood Pump Station using a JWCE Muffin Monster® pictured at right. In addition, there is an overflow from this facility to the former Primary Pump Station.

3.3.2.2. Secondary Treatment Process

The secondary treatment process at the Redway WWTF consists of a 300,000 gallon (nominal capacity) single-loop oxidation ditch, a 36-ft diameter secondary clarifier, and a RAS/WAS pump station. This system is operated to maintain appropriate environments to grow and support heterotrophic and autotrophic bacteria for the oxidation of biochemical oxyen demand and ammonia, respectively. Air is introduced into the mixed liquor by a 25 hp mechanical brush aerator, to maintain appropriate dissolved oxygen levels so that the bacterial population can metabolize the BOD and oxidize ammonia to nitrite and nitrate.

Figure 10. 300,000 Gallon Single Loop Oxidation Ditch

The bacteria suspended in the oxidation ditch (mixed liquor suspended solids (MLSS)) are settled out by gravity in a single, 36-ft diameter secondary clarifier. The biomass that settles to the bottom of the clarifier is collected by sludge scrapers and routed to a sludge sump in the bottom of the clarifier. To ensure enough bacteria are available to consume the BOD and ammonia in the oxidation ditch, a portion of the sludge settled in the secondary clarifier must be returned to the oxidation ditch from the secondary clarifiers (returned activated sludge or RAS). Excess biomass is slowly removed from the system and is termed waste activated sludge (WAS). The WAS is wasted to a circular sludge thickener for thickening and then drained to sludge drying beds for subsequent dewatering and land disposal on RCSD property to the west of the WWTP.

An effluent weir on the oxidation ditch outlet can be raised or lowered to increase or decrease the depth of the brush aerator's submergence, which will correspondingly increase or decrease the amount of oxygen transferred to the mixed liquor. At the middle of the weir's adjustment range the sidewall depth of the oxidation ditch is 8 ft and the volume of the ditch is 300,000 gallons. For the purposes of estimating peak treatment capacity of the oxidation ditch, it was assumed that the weir is at its highest level and therefore the aerator is at its maximum submergence. However, grit build-up in oxidation ditches of this type are typically significant, so the total reactor volume was held at 300,000 gallons to account for both a raised weir level and some accumulation of grit in the ditch. Grit should be periodically removed from the oxidation ditch or it will decrease the viable volume of the ditch.

Aeration is provided by a single 25 horsepower (hp) horizontal-rotor surface brush aerator. The aerator is the Magna Brush Rotor from Lakeside Equipment Corporation. Lakeside Equipment Company claims that the standard aeration efficiency for the brush aerator unit is 3.3 lb O_2/hp/hr. This was derated to 2.4 lb O_2/hp/hr to account for the differences between the vendor's test conditions and the Redway WWTF oxidation ditch, and for efficiency losses in the unit itself due to wear and tear. 2.4 lb O_2/hp/hr at 25 hp equates to a maximum standard oxygen transfer rate (aeration capacity) of 60 lb O_2/hr. Determination of the exact oxygen transfer capacity of the aerator would require extensive on-site testing.

The 36-ft diameter center-feed, inboard launder secondary clarifier has a sidewall depth of 12 ft. The activated sludge from the secondary clarifier can be returned to the oxidation ditch using RAS pumps. There are two centrifugal RAS pumps, each with a maximum capacity of 275 gpm (0.4 mgd). The pumps can be run in parallel with a total capacity of approximately 450 gpm (0.65 mgd). There is a single progressing cavity WAS transfer pump with a maximum capacity of 50 gpm, which pumps waste sludge from the secondary clarifier underflow to the WAS Thickener. A single TWAS pump pumps thickened sludge from the thickener underflow to the sludge drying beds. This pump is also a progressing cavity type with a maximum capacity of 50 gpm. All four (2 RAS, 1 WAS Transfer and 1 Thickened WAS) pumps have variable speed drives.

Figure 11. 36 Foot Diameter Secondary Clarifier

The Redway WWTF oxidation ditch was designed as an extended-aeration system. Extended air systems utilize a long (> 15 days) solids retention time (SRT)and relatively high mixed liquor suspended solids (MLSS) concentration. This design enables stable operation even in the face of the relatively broad fluctuations in raw wastewater quantity and quality common in smaller communities such as Redway.

Oxidation Ditches also inherently employ a very high internal recycle flow rate, brought about by the closed loop configuration. An internal recycle flow of approximately 85 mgd was used to analyze the Redway oxidation ditch. This was estimated using the cross-sectional area of the ditch (132 ft^2) and the typical minimum design velocity for oxidation ditch systems of 1.0 ft/sec, which is required to maintain the mixed liquor suspended solids in suspension.

Return Activated Sludge (RAS) is returned to the head of the oxidation ditch at a rate set and periodically adjusted as necessary by the operations staff. WAS pumping is monitored to maintain a target MLSS concentration and SRT in the oxidation ditch. Table 14 compiles the most critical characteristics of the Redway WWTF oxidation ditch for the purposes of determining its capacity.

Table 14. Redway WWTF Secondary Process Description

Parameter	Value	Notes
Volume	300,000 gal	Nominal, depends on effluent weir level and amount of grit buildup
Sidewater Depth	8-ft	
Channel Width	16.5-ft	16.5-ft
Solids Retention Time	>20 days	Operators currently utilize a very long SRT (> 40 days) and very high MLSS (> 7,000 mg/L) 2,000 to 6,000 mg/L
Mixed Liquor Suspended Solids	2,000 to 6,000 mg/L	
Internal Recycle Rate	85 mgd	Estimate
Brush Aerator Horsepower	25 hp	25 hp is motor size, actual input horsepower depends on submergence of brushes
Maximum Aeration Capacity	60 lb/day	60 lb/day
Secondary Clarifier Surface Area	1,018 ft^2	N/A
RAS Pumps	(2) 275 gpm VFD	N/A
WAS Pump	(1) 50 gpm VFD	N/A
TWAS Pump	50 gpm VFD	N/A

3.3.2.3. Existing WWTP Capacity Analysis

Secondary Treatment Capacity

The treatment capacity of secondary treatment systems should be evaluated against maximum month flows, since NPDES discharge permits limit publicly owned treatment works on average monthly concentrations for BOD_5, TSS and ammonia. Based on the expectation that the North Coast Regional Water Quality Control Board will enforce a 10 mg/L total nitrate limit in the next NPDES permit, the capacity of the Redway WWTP has been evaluated against achievement of this effluent quality.

It appears that the original designers sized the ditch based on volumetric organic loading rate (8 lb/1,000 ft3). This is a very simple metric used in the past to size oxidation ditches where nitrogen removal was not a significant concern. 8 lbs BOD_5/1,000 ft^3 is a very conservative value. Some oxidation ditches are designed for loading rates as high as 20 lbs BOD_5/1,000 ft^3. At the current approximate maximum month BOD_5 loading rate of 537 lb/day (see Table 11) and assuming an oxidation ditch volume of 300,000 gallons, the volumetric loading rate is currently 13.4 lbs BOD_5/1,000 ft^3. However, these simple volumetric loading rate, rules-of-thumb design guidelines have limited utility when examining the capacity of an existing oxidation ditch, or when designing a new oxidation ditch for nitrogen removal. For this reason the existing WWTP was modeled using more sophisticated spreadsheet based models and BioWin modeling software.

First, a general spreadsheet activated sludge model was utilized (provided in Appendix B). This model indicates that 290,000 gallons of oxidation ditch capacity is needed just for complete nitrification of the current maximum month BOD_5 and nitrogen load (ammonia plus organic nitrogen present in the raw wastewater influent). This does not include any assumption of anoxic volume for denitrification. Typically an anoxic volume is required that is at least 20% of the aerated volume, indicating that the existing 300,000 gallon oxidation ditch cannot be reasonably expected to provide nitrification and denitrification at current maximum month loading conditions.

The Biowin model consists of wastewater influent characterization, the 300,000 gallon oxidation ditch with 25 hp brush aerator, the 36-ft diameter clarifier, and RAS and WAS streams. A screen shot of the model is shown in Figure 12. A series of four bioreactor elements were used to model the oxidation ditch, effectively dividing the ditch into four quarters, in order to simulate the changing dissolved oxygen and waste concentrations across the reactor. The first quarter, or 75,000 gallon volume was modeled as un-aerated, similar to the District's oxidation ditch. A bioreactor with a 25-hp brush aerator was used in the second 75,000 gallon quarter to model the location of the brush aerator in the ditch. Two un-aerated quarters were used to model the transfer of dissolved oxygen along the length of the ditch as well as other constituent concentration profiles.

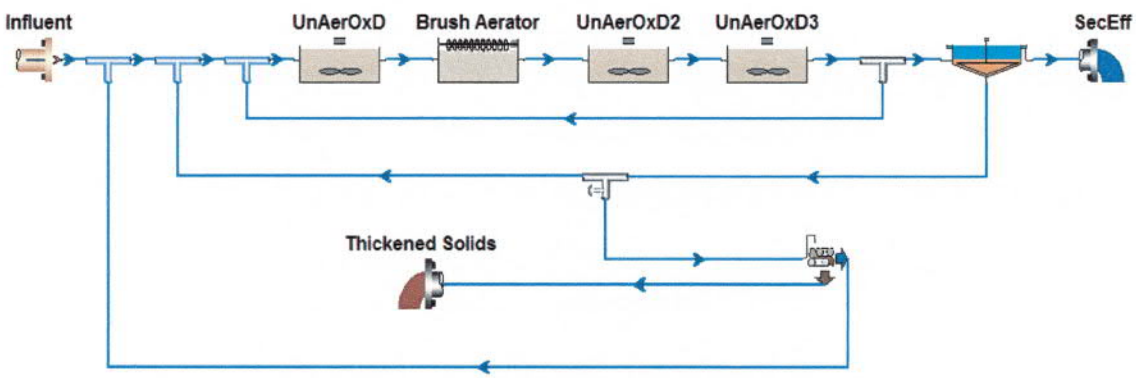


Figure 12. BioWin Secondary Treatment Process Flow Model of Existing WWTP

The default alpha coefficient (0.5) was changed to 0.85 to better reflect oxygen transfer in a brush aerator system. The Aerobic/Anoxic DO half saturation switch was adjusted from the default 0.05 to 0.25, which allows more anoxic activity to occur in the presence of oxygen, resulting in some degree of simultaneous nitrification and denitrification that typically occurs in moderately-loaded oxidation ditches.

An initial model run was executed at the current peak month wet weather flow of 0.293 mgd. An internal recycle rate of 85 mgd and a RAS flowrate of 0.15 mgd (~ 0.5Q) was utilized. The first run was executed with an SRT of 15 days to simulate a lower design SRT. The second run was executed with an SRT of 45 days, which more closely simulates current operating conditions at the plant, as operations staff usually run the oxidation ditch with a very high MLSS (> 5,000 mg/L) concentrations. The results of these runs are summarized in Table 15.

Table 15. BioWin Model Results

Parameter	Units	Influent	SRT 15 Days	SRT 45 Days
Flow (Q)	mgd	0.293	0.293	
RAS Rate	Portion of Flow	N/A	0.15 (0.5Q)	
Mixed Liquor Suspended Solids	mg/L	-	2,100	5,000
Biochemical Oxygen Demand	mg/L	219	2.5	2.7
Total Suspended Solids	mg/L	206	6.2	14.9
Ammonia	mg-N/L	26.4	0.35	0.23
Nitrite	mg-N/L	0	0.09	0.05
Nitrate	mg-N/L	0	13.4	12.9
Total Kjeldhal Nitrogen	mg-N/L	30	2.1	1.95
Total Nitrogen	mg-N/L	40	16	15.7

All model runs indicated that the existing treatment plant would be able to meet the current permitted waste discharge requirements in regards to BOD and TSS at peak month wet weather flows (30 mg/L average monthly limit, for each). However, if the Regional Water Quality Control Board imposes a nitrate-nitrogen limit of 10 mg-

N/L in the near future, the model indicates that the existing WWTP would not be able to comply. However, additional model runs were executed, which indicated that the WWTP would be able to comply with this limit at flows up to the existing maximum month demand, but only by reducing the water surface elevation in the oxidation ditch to a point where the 20 hp brush aerator would input approximately 15 hp of aeration energy. It is recommended that the District begin experimenting with this approach, and also reducing the SRT and MLSS concentration utilized at the WWTP, in anticipation of the new nitrate limit, to field-verify if this approach would enable the District to comply with the new limit without costly new activated sludge infrastructure.

The spreadsheet and Biowin models reveal that the existing WWTP is at or very near its treatment capacity in regards to meet a 10 mg/L nitrate limit.

Secondary Clarifier Treatment Capacity

The capacity of the secondary clarifier must ultimately be evaluated in conjunction with the oxidation ditch as they are components of an activated sludge system. However, the clarifier can also be evaluated using hydraulic loading rate (gpm/ft^2/day) and/or solids loading rate (SLR) in lb/d/ft^2, representing the mass of solids applied per unit area per unit time. It is calculated as follows:

$SLR = (8.34\ lb/gal*(Q + Q_{ras})*X) / A$

Where:

Q = Influent flow, mgd

Q_{ras} = RAS flow, mgd

X = MLSS concentration, mg/L

A = Clarifier surface area, ft^2

The equation above was rearranged to calculate the influent flow capacity. The equation and parameters used are listed below. In general, the maximum solids loading rate for secondary clarifiers coupled with oxidation ditches is 25 lb/d/ft^2. A value of 30 lb/d/ft2 was used to evaluate the Secondary Clarifier capacity. A MLSS concentration of 4,000 mg/L was used as well. Although operations staff currently operates at an MLSS much greater than this, 4,000 mg/L is a typical upper limit design value, and using the much higher MLSS concentration currently utilized at the WWTP would result in a much lower clarifier capacity.

$Q= (SLR*A)/(8.34*X) - Q_{ras}$

SLR = 25 lb/day/ft^2

A = 1,018 ft^2

$Q_{ras} = 0.5Q$

X = 4,000 mg/L MLSS

According to this, the clarifier is capable of treating an influent flow capacity of 0.5 mgd. This is substantially greater than the current maximum month wet weather flow of 0.293 mgd. It should be noted that if an MLSS value of 7,000 mg/L is utilized, reflective of current WWTP operation, the resulting flow limit would be 0.29 mgd, which is the current maximum month flow at the WWTP.

3.3.2.4. Disinfection

The Redway WWTF utilizes calcium hypochlorite and a Chlorine Contact Basin for disinfection purposes. The basin is a 38 ft x 18 ft x 6 ft depth masonry block structure with three baffle walls, resulting in a four-pass serpentine channel configuration, see Figure 13. The minimum operating water surface elevation in the basin is 4 ft. The minimum operating volume of the basin is approximately 18,000 gallons.

Figure 13. RCSD WWTP Chlorine Contact Basin

Redway's waste discharge permit requires that they achieve a monthly median effluent total coliform count of 23 MPN/100 mL, and a maximum daily count of 230 MPN/100 mL. The WWTP must also maintain a minimum chlorine residual of 1.5 mg/L at the end of the contact basin. This residual must then be quenched using sulfur dioxide.

The contact basin volume will provide a contact time of 30 minutes at the WWTP design PWWF of 600 gpm or less. The recommended chlorine dosage to achieve an effluent total coliform count of 230 MPN/100 mL at a contact time of 30 minutes for a nitrified and clarified activated sludge effluent is 6 to 16 mg/L (Metcalf & Eddy). The broad chlorine dose range recommended by Metcalf & Eddy reflects the high variability in site-specific initial chlorine demand. Without actual site testing, it is not possible to more accurately estimate the required chlorine dose.

Because current peak flows appear to have exceeded the original design flow for the Contact Basin of 600 gpm, it is necessary to re-calculate the residual that should be maintained at the outlet of the basin when flows are in excess of 600 gpm. To do this, White's (1999) equation can be utilized:

$$\frac{N}{N_O} = \left[\frac{(C_R t)}{b}\right]^{-n}$$

Where:

N	= Required Winter Total Coliform Count (MPN/100 mL)
N_o	= Nitrified Effluent Total Coliform Count (MPN/100 mL)
CR	= Residual Chlorine Concentration at Time (t, mg/L)
B	= Coefficient for Secondary Effluent Total Coliform Inactivation (unitless)
N	= Slope of Total Coliform Inactivation Curve (unitless)
T	= Required Contact Time (minutes)

To solve for the required chlorine contact residual at a given contact time, the equation is rearranged to:

$$t = \left(\frac{N}{N_O}\right)^{-1/n} b\left(\frac{1}{C_R}\right)$$

The values for the parameters in the equation are shown in Table 12. N_O is an estimate of the total coliform count for a nitrified effluent such as Redway's. Metcalf & Eddy gives a range for total coliform in nitrified effluents of 1.0E+04 to 1.0E+06. 1.0E+06 was selected as a conservative value. b and n are coefficients established experimentally and supported by empirical data from chlorine disinfection systems. The Regional Board requires Redway to maintain a minimum chlorine residual of 1.5 mg/L at the end of the contact basin. A higher residual should be maintained at flows in excess of 600 gpm. As shown in Table 16, if a residual of 3.2 mg/L is maintained, then the maximum flow rate through the basin is over 1 mgd.

Table 16. Estimation of Disinfection Capacity Based on Chlorine Residual

Parameter	Symbol	Units	Low Residual	High Residual
Required Total Coliform Count	N	MPN/100 mL	230	230
Nitrified Effluent Total Coliform Count	N_o	MPN/100 mL	1.0E+06	1.0E+06
Residual Chlorine Concentration at Time (t)	C_R	mg/L	1.5	3.2
Coefficient for Secondary Effluent - Total Coliform	b	NA	4	4
Slope of Inactivation Curve	n	NA	2.8	2.8
Required Contact Time	t	mins	53.1	25.0
Corresponding Maximum Flow Rate		gpm	340	720
		mgd	0.49	1.04

Redway WWTF operations staff experience will be the best reference for the initial chlorine dose required to maintain a given residual at the Chlorine Contact Basin outlet. Typically the initial chlorine demand for secondary effluent is approximately 3 mg/L, and over 30 minutes a maximum decay of 4 mg/L can be expected. These conditions would require an initial dose of 10.2 mg/L to maintain a residual of 3.2 mg/L.

Chlorinator capacity is 100 lb/day per chlorinator. One chlorinator could provide an initial dose of 12 mg/L at a flow of 1.0 mgd. Obviously there is plenty of chlorinator capacity for the WWTF. There is more than enough sulfur dioxide capacity for the WWTF since only a 1:1 ratio of sulfur dioxide to chlorine residual is required. Assuming a maximum residual of 5 mg/L, the treatment capacity of the sulfonators is over 2.0 mgd.

The ultimate capacity of the chlorination system should be viewed as the flow that would reduce the contact time in the basin to 15 minutes. This is a commonly used minimum design standard. For Redway, this occurs once the flow reaches 1,200 gpm. The residual requirement at this flow would be approximately 5.3 mg/L, which would require an approximate initial dose of 12.3 mg/L.

3.3.2.5. Effluent Disposal Capacity

From October 1 through May 14, treated wastewater is discharged to the South Fork Eel River. From May 15 through September 30, treated wastewater is discharged to percolation ponds located in an upland area away from the river channel and floodplain. The percolation ponds are located on property adjacent to the WWTF across a deep ravine. Treated wastewater is conveyed approximately 1200 feet to the percolation ponds via a 4-inch suspended transmission line. Flow through this line is limited to 243 gpm (0.35 mgd). Although the percolation ponds appear to perform sufficiently to accept wintertime flows, the transmission line limits the amount of effluent that can be disposed in this manner. Consequently, due to wintertime infiltration and inflow (I/I), when flows exceed 0.350 mgd, effluent is transmitted to the South Fork Eel River for disposal.

A new Effluent Pump Station (EPS) and on-site Percolation Pond are currently being designed for RCSD by WWE. The new EPS is needed to replace the existing station, which is at the end of its usable life. The existing station will be retained as an emergency backup station. The new percolation pond will be located to the west of the existing oxidation ditch, and will be able to accept approximately 70,000 gallons per day. This will bring the total percolation capacity to approximately 0.42 mgd.

3.3.2.6. Solids Dewatering Capacity

The Redway WWTF dewatering facility consists of four (4) 24 ft x 100 ft sludge drying beds, shown in Figure 14. The beds dewatering capacity had proven insufficient, so a bag filter dewatering unit, was purchased and installed to supplement the drying beds. However, the bag filter dewatering unit is batch operated and very labor intensive. In 2005, a sludge thickener was installed. This improved the capacity of the drying beds because a thicker sludge was being applied. Also, drainage tiles were installed in the drying beds which also improved the drying beds' capacity. This improved capacity eliminated the need to run the bag filter dewatering unit. The bag filter is still onsite, but is currently not being used.

Figure 14. Sludge Drying Beds and Drainage Tiles

The dewatering capacity of the sludge drying beds is very dependent on weather. Redway's climate is remarkable for its long wet-weather season and short dry-weather season. The dry-weather season brings warm temperatures, often above 90 degrees Fahrenheit, but the humidity and occurrence of cloud cover is higher than that found in California's Central Valley. There are about five months of reliable dewatering time, typically from early June to sometime in October.

In the summer, the drying beds are sufficient, but in the winter, it is not possible to dewater the solids. Currently, the District hires the services of a dewatering contractor on an as-needed basis. This is a relatively expensive solution over the long-term. In addition, a new treatment process designed around a nitrogen limit of 10 mg-N/L would produce more solids. A lower solids retention time would be used, and more solids would need to be wasted to keep the solids concentration down.

3.3.3. Treatment Plant Hydraulic Capacity

The WWTP hydraulic capacity was estimated using a calculation based spreadsheet tool developed by Water Works Engineers (Appendix C). The plant was divided into four (4) hydraulic units, separated by weirs or air gaps:

- Chlorine Contact Basin back to Secondary Clarifier Launder
- Secondary Clarifier back to Oxidation Ditch Outlet Box

- Oxidation Ditch
- Inlet Structure

The analysis was based on the following:

- Only the hydraulics within the plant were calculated. The analysis did not include the pipeline between the WWTF and the Dogwood or Primary Lift Stations, or the Treatment Plant Effluent pipeline to the percolation ponds or Eel River. The point of the analysis was to determine the peak hydraulic capacity of the WWTF. This event will occur during the wet-weather system, when effluent is discharged to the Eel River, nearly 100 ft below the plant elevation.
- Only gravity flow hydraulics were calculated in the spreadsheet format. The capacity for all pumped systems has been interpreted as the pumps design capacity. Design capacities for pumped systems have been taken from design criteria tables in the WWTF O&M manual.
- Piping existing prior to the 1996 upgrade is modeled as mortar lined ductile iron. Piping called out as C900 PVC in the 1996 upgrade was modeled as PVC.
- In order to estimate the water surface elevation through the inlet structure, oxidation ditch, and secondary clarifier influent piping, the RAS rate utilized at the time of the peak flow must be added to the flow. A conservative RAS flow during the peak wet weather periods will be approximately 0.432 (300 gpm). Therefore, the current PHWWF (600 gpm) would result in a total flow through the oxidation ditch and secondary clarifier piping of 900 gpm.

Results of the hydraulic analysis at a PHWWF of 600 gpm are shown in Table 17 at major structures and other key points in the WWTF (i.e., open channels where overflow or weir submergence is of concern).

Table 17. Results of Hydraulics Analysis at 600 gpm with 300 gpm RAS

Location	Critical Structure and Elevation (ft MSL)	Water Surface Elevation (ft MSL)
Inlet Structure	Top of Wall, 399.5	394.95
Oxidation Ditch	Top of Wall, 393.5	391.89
Oxidation Ditch Effluent Box	Top of Wall, 393.5 Top of Effluent Weir[1] 390.67 (min) 391.5 (mid) Unknown max	390.59
Secondary Clarifier	Weir Elevation, 387.92	388.28
Secondary Clarifier Launder	Weir Elevation, 387.92	387.67
Chlorine Contact Basin	Top of Wall, 388.0	386.0

[1]Oxidation Ditch Effluent Weir is adjustable.

The only location in the WWTF where water surface elevations are encroaching upon a critical structure is at the overflow weir of the secondary clarifier. This is a v-notch weir, and the elevation shown (387.92) is the bottom of the notch. The weir will be in a submerged condition (upstream and downstream water surface elevations equal) at approximately 665 gpm raw wastewater flow. Submergence of this weir is not critical because it is not used for flow measurement. However, v-notch weirs are used on secondary clarifiers to balance flow around the perimeter of the clarifier to prevent preferential flow over only a portion of the total weir length. Submergence is therefore not desirable, and if peak wet-weather flows increase over time then the piping downstream of the clarifier could be upsized (from 8" to 10") to lower the water surface elevation in the secondary clarifier launder.

All other structures in the plant will pass a flow well in excess of 800 gpm. Flows higher than this value were not analyzed because there is no real potential for peaks to exceed this amount in the near term.

3.4. *Wastewater System Capacity Analysis Summary*

Table 18. Wastewater System Capacity

Process	Current Treatment Capacity[1]	Current Hydraulic Capacity
Dogwood Lift Station	NA	600 gpm
Inlet Structure	NA	> 700 gpm
Oxidation Ditch	0.29 mgd	> 700 gpm
Secondary Clarifier	0.5 mgd	665 gpm
Disinfection	1,200 gpm	> 700 gpm
Sludge Dewatering	At Capacity	NA

[1]Capacity of Existing Treatment Plant to meet Current NPDES Permit Requirements.

[2] Capacity of Existing Treatment Plant to meet a nitrate limit of 10 mg-N/l.

The most limiting facility at the treatment plant currently is the sludge dewatering process. Dewatering capacity is currently sufficient during the dry weather season, but not during the wet-weather season.

The next item of concern is the peak hour wet weather flow hydraulic capacity of the piping downstream of the secondary clarifier overflow weir. Instantaneous flow above approximately 665 gpm will submerge the weir. This can be alleviated by upsizing the piping downstream of the clarifier from 8-inch to 10-inch, but this is not critical for short-term operation.

The last item of concern is the existing oxidation ditches apparent inability to denitrify at a level that will be necessary to achieve an anticipated future 10 mg/L nitrate limit. The existing WWTP may be able to meet this limit by closely monitoring immersion of the existing brush aerator and other process parameters, but it appears to be already close to its treatment capacity even if denitrification performance is optimized.

Several options exists for improving the denitrification capacity of the plant, including 1) supplementing the existing oxidation ditch with a pre-anoxic basin and internal mixed liquor recycle pumping provisions, 2) installation of a new oxidation ditch in parallel to the existing oxidation ditch, or 3) installation of a new, stand-alone oxidation ditch and re-tasking of the existing oxidation ditch to an ancillary facility such as emergency overflow, aerobic digester, etc. However, all of the options would be costly, and would require procurement of funding assistance in order to implement.

4. Water and Wastewater System Capacity Summary

Table 19 provides a summary table of the District's water and wastewater system components, and either their estimated quantifiable capacity, or a summary of current capacity issues; the current demand (for water systems) or flow (for wastewater systems); and notes identifying the key issues concerning the component.

Table 19. Summary of RCSD Water and Wastewater Infrastructure Component Capacities and Current Demand/Flows

Inftrastructure Group / Unit Process	Current Capacity	Current Demand / Flow	Notes
Water System			
Water Treatment Plant	450 gpm (Instantaneous) 0.494 mgd Maximum Day (**At Capacity**)	Average Day: 0.21 mgd Maximum Day: 0.494 mgd Peak Hour Demand: 514 gpm	At Current Maximum Day Demand of 494,000 gallons, the the plant has to run 18.3 hours per day at its full output of 450 gpm. Peak Hour Demand is partially supplied by storage.
Infiltration Gallery / Intake Pump Station	450 gpm		The infiltration gallery improvements improved raw water inflow capacity but ultimate capacity of the infiltration gallery is not known.
Pretreatment	365 gpm Wet-Weather 548 gpm Dry-Weather		Treatment capacity improves with water quality. Generally excellent raw water quality during dry weather season enables higher throughput.
Filtration	470 gpm		470 gpm Firm Treatment Capacity (1 filter out of service and 10% backwash waste)
Disinfection	527 gpm Wet-Weather 972 gpm Dry-Weather		Summer disinfection capacity is much higher than 527 gpm due to improved chlorine efficacy at higher temperatures
Water Storage			
Rusk Avenue Tanks	735,000 gallons	674,000 gallons	Current maximum day demand is 494,000 gallons, plus minimum recommended fire flow of 1,500 gpm for a minimum period of two hours, which equates to 180,000 gallons. Sum of these is 674,000 gallons.
Rusk Avenue Tank 1	275,000 gallons		
Rusk Avenue Tank 2	460,000 gallons		
Meadows Industrial Tank	100,000 gallons		
Water Distribution System	**At Capacity**	20 psi during Fire Flows	District's Fire Flow report and previous work by Spencer Engineering indicates less than 20 PSI of water pressure in various portions of the distribution system during fire flows. Accurate identification of zones that are under capacity would require a detailed water distribution system hydraulic model.
Wastewater System			
Wastewater Collection System			
Dogwood Lift Station	600 gpm	Peak Instantaneous Flow: 600 gpm	Capacity of 6" and 8" forcemains from Dogwood LS to WWTP is significantly greater than 600 gpm, but pump control configuration currently limits output to 600 gpm.
Satellite Lift Stations	Varies	Varies	Accurate capacity of satellite portions of the collection system cannot be verified without a detailed collection system hydraulic model.
Wastewater Treatment Plant			
Inlet Structure	700 gpm	Average Dry Weather: 0.104 mgd Annual Average: 0.130 mgd Peak Month Wet Weather: 0.293 mgd Peak Day Wet Weather: 0.578 mgd	
Oxidation Ditch	0.293 mgd (**At Capacity**)		WWTP currently does not comply with anticipated future nitrate limit of 10 mg/L. Basic spreadsheet model indicates that the existing maximum month treatment capacity for Nitrate is less than the current peak month wet weather flow of 0.293 mgd. Biowin modeling indicates that the existing WWTP may be able to meet a 10 mg/L nitrate limit if operational modifications are made. However, this should be carefully evaluated by District staff.
Secondary Clarifier	0.5 mgd		Based on a maximum MLSS concentration of 5,000 mg/L. At current MLSS concentrations > 7,000 mg/L, clarifier is **At Capacity**
Disinfection	700 gpm		This is the hydraulic capacity of the contact chamber, the disinfection capacity is much higher, depending on the amount of chlorine injected into the effluent.
Effluent Disposal	350,000 gpd (Percolation Ponds) Excess to South Fork of the Eel River		Does not include estimated capacity of new percolation pond (70,000 gpd).
Sludge Dewatering	Sufficient Capacity	N/A	