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Reference: 005161.400

March 8, 2016

Frank Shaw Bacik, President
Town of Scotia Company, LLC.
PO Box 245
Scotia, CA 95565

Subject: Detailed Engineering Analysis, Scotia, California; Revision 4

Dear Frank:

As requested by the Scotia Community Services District (SCSD), SHN has revised Chapters 2 and 5 of our May 2009 Detailed Engineering Analysis; Revision 3. The attached revised chapters of the report presents existing treatment systems and proposed recommendations and associated costs to implement the recommended changes to both the water and wastewater treatment plants. SHN Consulting Engineers & Geologists, Inc. conducted this analysis on behalf of the Town of Scotia Company, LLC (TOS).

The revised chapters present a comprehensive study addressing avenues of service for water and wastewater treatment to be provided by the SCSD.

We look forward to continuing our work with you during the transition of services from TOS. to the SCSD. If you have any questions, please call either of us.

Best Regards,

SHN Engineers & Geologists

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12/31/17

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Enclosures: Chapter 2.0 Wastewater Treatment
Chapter 5.0 Water Treatment

2 Wastewater Treatment

2.0 Wastewater Treatment

2.1 Introduction

This section provides an overview of the existing treatment processes at the Scotia WWTF and assesses the condition, performance, and capacity of those processes. The assessment is based on analysis of wastewater operational data provided by TOS for the period from January 2010 through December 2014 and on-site inspections by SHN of the wastewater treatment facilities. Recommendations are included where deficiencies have been identified and system upgrades are required.

2.2 Description of Existing Treatment System

The TOS WWTF was constructed in 1954 and has not undergone any significant upgrades since start-up. The equipment has been well maintained and replaced or rebuilt as necessary, but much of the equipment and all of the main structural components are more than 50 years old. However, the existing WWTF has been operating in compliance with its existing NPDES permit conditions.

The treatment system, as illustrated in Figure 2-1, consists of the following processes:

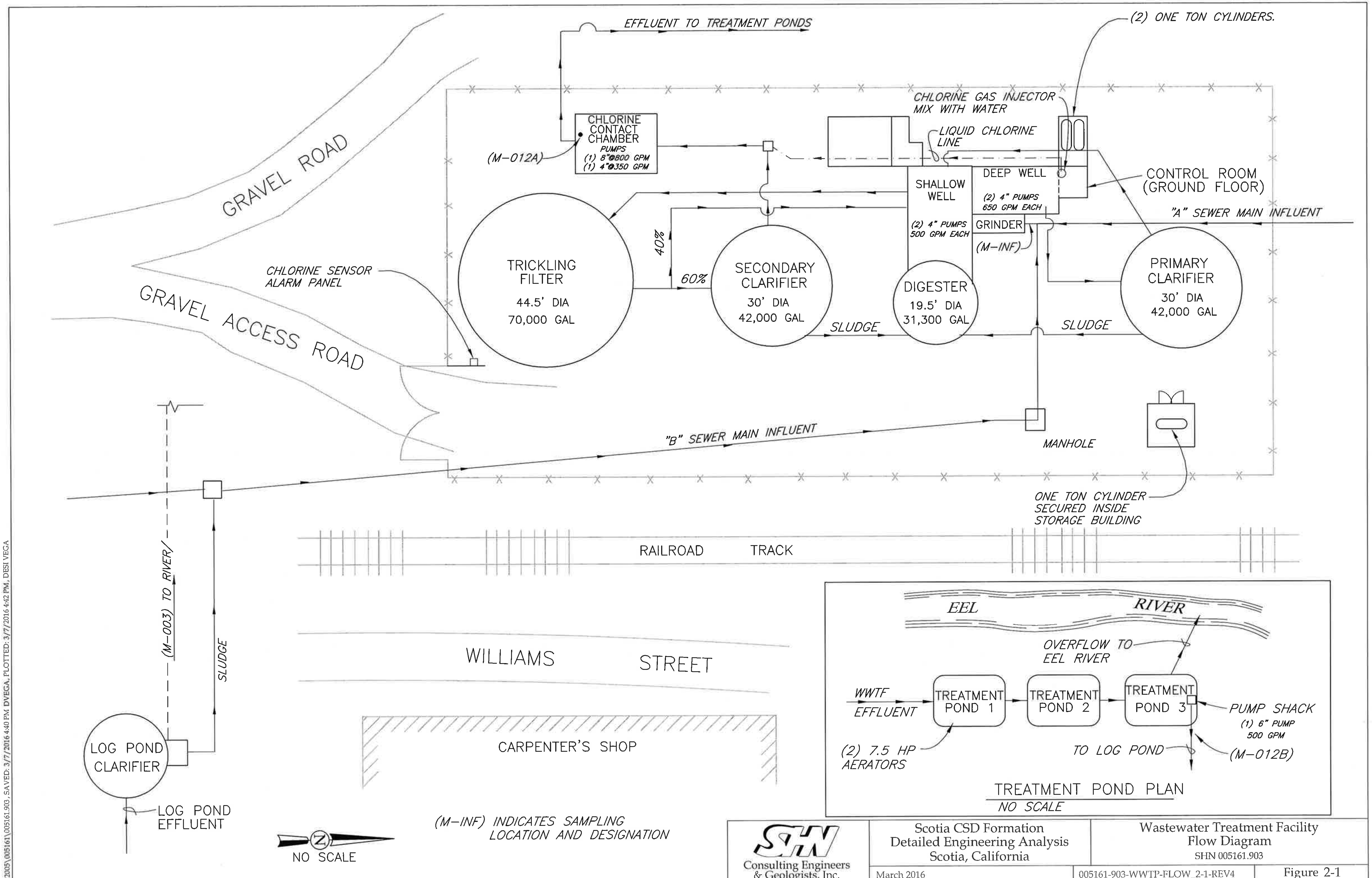
1. Pre-treatment: grit removal channel with grinder and bypass bar screen
2. Primary treatment: clarification
3. Secondary treatment: redwood trickling filter followed by clarification
4. Disinfection: gas chlorination
5. Advanced treatment: three treatment/polishing ponds following chlorine contact
6. Biosolids: storage in non-operational anaerobic digester and monthly disposal by a licensed contractor

Table 2-32 (at end of chapter) presents an inventory and listing of upgrades and maintenance activities of the various treatment equipment and structures.

Influent enters the WWTF through two gravity sewer mains that discharge into a headworks channel provided with a grinder and Parshall flume for flow metering. From the headworks, the sewage flows into a wet-well called the "deep well" where it is pumped to the primary clarifier. The effluent from the primary clarifier discharges to a second wet-well called the "shallow well" before being pumped to the trickling filter for secondary biological treatment.

The trickling filter effluent flows into a recirculation box where it is split into flow streams across two weirs. Operations staff has estimated that during normal operations, 60% of the trickling filter effluent flows to the secondary clarifier and the remaining 40% is diverted to the shallow well for re-circulation through the trickling filter.

From the secondary clarifier, secondary effluent is discharged to the chlorine contact chamber where chlorine solution is injected into the flow stream for disinfection. Disinfected effluent from the chlorine contact chamber is then pumped to a series of three treatment ponds. Treatment Pond 1 has two 7.5-hp aerators. From the treatment ponds, treated effluent is sampled for compliance before being pumped to the log pond for disposal. The effluent from the treatment ponds flows



Wastewater Treatment Facility
Flow Diagram
SHN 005161.903

Figure 2-1

through the log pond to the log pond clarifier, which discharges to the Eel River during wet weather (October 1 –May 14). During dry weather (May 15 – September 30), when discharge to the river is prohibited, treated effluent is stored in the log pond. Based on 24-hour composite samples of the influent wastewater (monitoring site M-INF) and effluent discharged from Treatment Pond 3 (monitoring site M-012B), the facility achieved average removal rates greater than 97% for both BOD and TSS.

A summary table listing the existing equipment and its age is presented at the end of this chapter.

2.2.1 Headworks: Pre-treatment/Flow Monitoring

Influent wastewater enters the WWTF through one of two gravity trunk mains. The Mill A line is a 15-inch VCP that conveys flows from the north end of the facility. Mill B Line is a 15-inch VCP line that conveys flows from the south end of the facility. The influent wastewater from the Mill A and B lines is combined at the headworks, before passing through a non-aerated grit channel and grinder. A bypass channel equipped with a bar rack is provided for flows diverted around the grinder. These flows are typically diverted to the bar rack for grinder maintenance or repair.

After the influent goes through the grinder, it is routed to the deep well through a Parshall flume. Water level is recorded using an ultra sonic level sensor that measures the water at the throat of flume. Depending upon the level of water ahead of the flume, the level sensor reading equates to a measurement of the flow into the WWTF. The flow meter is located in the chlorine control room and is equipped with a totalizer and recorder for 24-hour flows. The meter has a local readout of instantaneous flow rates in gallons per minute (gpm).

In September 2012, a new overflow pump system was installed at the WWTF headworks. The overflow pump system upgrade included installation of a new back-up pump designed to handle and route flows that are in excess of the facility’s hydraulic capacity. These excess flows would be routed to Treatment Pond 1.

2.2.1.1 Condition

TOS operators have noted that the grit chamber does not require frequent cleaning. It has also been noted that the collection system is in poor condition and it appears grit may settle out elsewhere in the collection system; or at high flows, the grit may wash through the channel and collect in other parts of the treatment train. Sand was noted in the bottom of the digester during cleaning.

Pre-treatment consists of a Muffin Monster grinder purchased in 1996. According to TOS operators, the Muffin Monster needs to be serviced. Much of the non-biodegradable material settles out in the primary clarifier or is scraped off with the floatables and delivered to the digester as primary sludge. The non-biodegradable material poses a maintenance concern contributing to wear and plugging of wastewater and biosolids pumps throughout the treatment process. The influent flow meter was installed in 2002 and is in good condition. During high flows, the grinder and sensor must be removed to avoid inundation and resulting damage.

2.2.1.2 Headworks Issues

- The system lacks automated notification of a bypass condition or metering of overflow from the headworks channel.
- The system lacks prescreening and removal of non-biodegradable material.

- The headworks is a confined space and requires a minimum of two operators for safe entry.
- The system lacks flow readings during major storm events.
- Muffin monster grinder needs to be serviced
- Some plastic and other non biodegradable material passes through the headworks

2.2.2 Primary Treatment

From the headworks, the sewage flows into the deep well, where it is pumped to the primary clarifier by the deep well submersible sewage pumps. Effluent from the primary clarifier, gravity feeds back to the shallow well through a 10-inch pipe.

2.2.2.1 Condition

The primary clarifier is a 30-foot-diameter buried concrete tank constructed in 1954. The distribution and collection system is a bridge-supported unit with a worm gear drive. The scrapers and collection arm were replaced in 1997. A new v-notch weir was installed and the clarifier was leveled in February 2012; the clarifier gear box and gears were replaced in April 2012. The top of the tank is covered by a square mesh screen, which is in need of replacement, supported by steel framework to deter vandalism and bird activity.

TOS operators have noted that the capacity of the discharge line to the shallow well is limited, and when both deep well pumps are on, the water level in the launders (primary effluent trough) increases to a point that it overflows and spills onto the ground on the low side of the clarifier. The 10-inch discharge line from the primary clarifier is cast iron and has an approximate slope of 1.2%. Assuming a Manning's coefficient (n) of 0.015 for rough, uncoated cast iron pipe, the full flow capacity is estimated to be 1.5 MGD.

The deep well pumps are two 20 horsepower (hp) submersibles with a design firm capacity (firm capacity assumes one pump is off-line) of 650 gpm (0.94 MGD). The pumps were replaced in November 2006. The new pumps were installed with a rail system so that they can be pulled for maintenance from the surface, eliminating the need for confined space entry. In August 2010, an auto-dialer was installed on the deep well high level alarm at the WWTF. The auto-dialer is set to call all operators and electricians when the alarm is engaged.

2.2.2.2 Primary Treatment Issues

- The second deep well pump cannot be brought on line for a significant period of time without overflowing the primary clarifier.

2.2.3 Secondary Treatment

Secondary wastewater treatment at the WWTF consists of a trickling filter with redwood slat filter media, followed by a secondary clarifier. Primary effluent is pumped to the trickling filter distribution arms by the shallow well pumps.

2.2.3.1 Condition

The shallow well pumps are line shaft turbines with an estimated firm capacity of 500 gpm. The pumps were rebuilt, one in 1994 and one in 1996. One shallow well pump was replaced in October 2015. The filter beds are dosed through a rotary/reaction distributor made up of two horizontal pipes supported by a center column.

The trickling filter is contained in an above-ground circular concrete tank that appears to be in good condition, with no visible cracks or leakage. The tank is approximately 6-8 feet deep and 44.5 feet in diameter. The redwood slats filter media are original and appear in good condition. The distributor arm was replaced in 2004. TOS personnel have noted that intermittent hydraulic loading allows the filter to dry out.

The secondary clarifier, identical in construction to the primary clarifier, is 30 feet in diameter and approximately 7 feet deep. The clarifier is shallower than typical depths recommended for secondary clarifiers following trickling filters (typically 11 feet). The shallow depth limits the treatment performance at high flow rates. The effects of the depth on the design surface overflow rate (SOR) and the resulting treatment capacity are discussed in Section 2.4.4.

In 2009, the secondary clarifier was rebuilt and an adjustable v-notch ring was placed on the secondary clarifier, as well as new screening.

2.2.3.2 Secondary Treatment System Issues

- Intermittent hydraulic loading allows filter media to dry out.

2.2.4 Disinfection

Chlorine gas contained in one-ton cylinders is injected into potable water by a chlorinator in the chlorine room to form chlorine solution for disinfection. Chlorine solution is piped to diffusers in the chlorine contact basin where it is mixed with secondary effluent. At the end of the chlorine contact basin (CCB), the disinfected effluent is pumped to the treatment ponds for additional treatment.

2.2.4.1 Condition

The chlorinator, installed in 2003, is in good condition and is regularly serviced by the equipment suppliers. The chlorinator is flow-paced based on a signal from the influent flow meter, which is also located in the chlorine control room. Dosage is adjusted at the chlorinator control panel based on the pounds per day (lb/day) readout on a rotameter (a variable area flow metering device used for chemicals), which is located on the gas line prior to the injector. In July 2015, repairs to the chlorine injection system were made, including replacement of the ejector and rotometer.

Two pumps at the end of the CCB pump disinfected effluent to the treatment ponds. A 15-hp line-shaft turbine with a capacity of 800 gpm (1.15 MGD) was installed in October 2006 and operates as the lead pump. The lag pump is a 10-hp line shaft turbine pump with an estimated capacity of 350 gpm (0.50 MGD). There was an existing overflow pipe at the end of the CCB that allowed disinfected effluent to discharge to the Eel River; however, this outfall point has been removed. With both pumps running during high flow events, peak flows can be pumped to the treatment ponds without overtopping or diverting to the river.

The chlorine contact basin is a serpentine concrete basin constructed in 1954 and has a series of under-and-over baffles designed to prevent short-circuiting and maximize contact time in the basin. The weir wall that separates the effluent pumps from the CCB historically leaked but was repaired in February 2007.

2.2.4.2 Disinfection Issues

- System needs a second 15-hp pump in the contact basin to provide redundancy.

2.2.5 Treatment Ponds

The CCB discharges into the first of three aerobic treatment ponds. The ponds have been operated with highly variable levels, but generally function as aerobic low rate or "maturation ponds." Aerobic maturation ponds are lightly loaded, relatively shallow ponds 3 to 5 feet deep. Oxygen is provided in the ponds by surface re-aeration, photosynthesis by algae, and denitrification of nitrate (NO_3). A summary of the treatment ponds sizing and equipment is provided in Table 2-1.

Table 2-1
Wastewater Treatment Facility Size and Equipment Assessment—Treatment Ponds
TOS Detailed Engineering Analysis

Equipment	Description	Size			Installation	Major Repair
		Area (SF)[1]	Depth (feet)	Volume (MG)[2]		
Treatment Pond 1	Aerobic pond	28,000	4	0.84	1960	2005 Cleaning
Treatment Pond 2	Aerobic pond	45,000	4	1.35	1960	2005 Cleaning
Treatment Pond 3	Aerobic pond	40,000	4	1.20	1960	2005 Cleaning
		(inches)	(gpm[3])	(hp[4])		
Effluent Pump	Line shaft turbine	6	500	40	2004	
Aerators (2)				7.5	2009	

1. SF: square feet
2. MG: million gallons
3. gpm: gallons per minute
4. hp: horsepower

The treatment ponds were cleared of vegetation in 2007 and 2008, and two 7.5-hp pond aerators were installed in the first pond in 2009.

2.2.5.1 Effluent Pumps

Effluent from Treatment Pond 3 is pumped to the log pond by the line-shaft turbine pump located at the end of the pond. A single pump is activated by the level in the treatment pond. The pump is accessed by a catwalk that extends out into the pond. An emergency overflow is plumbed to the Eel River at the end of Pond 3.

A small pump house adjacent to the catwalk at Pond 3 contains the pump controls and a composite sampler. Samples collected from Pond 3 are analyzed for compliance with discharge requirements for BOD, TSS, and pH. In 2007, the pump house was cleaned out and additional security work was performed.

2.2.5.2 Condition

The ponds are full of biosolids. Although the ponds are reportedly more than 10 feet deep in some sections, depth of clear water above the sludge blanket is only approximately 4 feet during winter months and approximately 2 feet in the summer months. Vegetation continually encroaches on the edge of the ponds and at times, Pond 3 has been almost entirely covered with duckweed. In June 2006 and in 2007, much of the vegetation was removed from the treatment ponds. It is necessary to perform this maintenance on an annual basis, and this task will be part of the Operations and Maintenance Plan that will be developed in accordance with the NPDES permit requirements. A sludge inventory and removal plan is included as recommended improvements in Section 2.10.

2.2.5.3 Treatment Pond Issues

- Culverts between ponds do not allow a single pond to be taken off line.
- There is a lack of level control in the ponds.

2.2.6 Biosolids

In December 2011, changes to the sludge handling system were completed. The use of the unlined dewatering trench has been discontinued and the existing sludge digester is now used to store sludge generated at the facility temporarily. The digester was retrofitted with a new fitting so that the sludge can be removed periodically from the digester for transport to an appropriate biosolids handling facility. TOS has contracted with Steve's Septic of McKinleyville, California, for sludge removal. Steve's Septic has a permitted dewatering process pad and all dewatered biosolids are sent to a licensed facility for disposal.

Solids are pumped from the primary and secondary clarifiers to the anaerobic digester using one of two sludge pumps located in the pump room.

2.2.6.1 Condition

The sludge pumps are positive displacement, plunger pumps that were installed when the WWTF was constructed in 1954. According to the operator, the pumps were rebuilt in 2000. They are well maintained and in good condition.

2.3 Regulatory Criteria

This section summarizes the NPDES waste discharge requirements for the TOS Scotia WWTF. TOS currently discharges under Order No. R1-2012-0065 and NPDES Permit No. CA0006017. This permit was adopted by the RWQCB on April 26, 2012, by Order No. R1-2012-0065, and contains the waste discharge requirements for both the Scotia municipal WWTF and the Eel River Power cogeneration plant. The new permit went into effect on July 1, 2012, and expires on June 30, 2017.

2.3.1 Discharge Prohibitions

The Scotia WWTF is prohibited from discharging wastewater to the Eel River during the period May 15 through September 30 each year. During the period October 1 through May 14 of each year, discharges of treated wastewater to the Eel River shall not exceed 1% of the flow of the Eel River, based on the most recent daily flow measurement, as measured at the Scotia gauging station

(United States Geological Survey [USGS] Station 11477000). Additionally, the total volume of treated wastewater discharged to the Eel River in a calendar month shall not exceed 1% of the total volume of the Eel River in the same calendar month.

2.3.2 Effluent Limitations

The effluent limitations contained in the new permit are similar to the previous permit. Table 2-2 summarizes the monitoring locations for compliance with the effluent limitations. These locations are also shown in Figure 2-1.

Table 2-2
Wastewater Treatment Facility Monitoring Locations[1]
TOS Detailed Engineering Analysis

Monitoring Location Name	Monitoring Location Description
M-INF	Influent monitoring location—a point in the facility headworks preceding any treatment and receiving all waste from the collection system
M-012A	Chlorine contact basin effluent weir
M-012B	Point of discharge at the end of the sanitary waste treatment train prior to discharge into the log pond
M-003	Log pond effluent discharge

1. Reproduced from NPDES No. CA0006017, Attachment E: Monitoring and Reporting Program (MRP)

Table 2-3 summarizes the effluent limitations for the WWTF. Treated wastewater discharged to the Eel River from the log pond must not contain detectable levels of total chlorine, as measured at Monitoring Location M-003. In addition to these effluent limitations, the permit requires that the average monthly removal of BOD and TSS shall not be less than 85% as measured at Monitoring Location M-012B. The removal shall be determined from the monthly average influent concentrations and monthly average effluent concentrations for each constituent over the same period. A Toxicity Reduction Evaluations Workplan is also required by the permit. This has been completed, and was updated in May 2015.

Table 2-3
Wastewater Treatment Facility Effluent Limitations[1]
TOS Detailed Engineering Analysis

Parameter		Compliance Point	Monthly Average[2]	Weekly Average[3]	Daily Max.	Instantaneous		Sampling	
						Min.	Max.	Type	Frequency
BOD[4]	mg/L[5]	M-012B	30	45	60	--	--	24-hr. Composite	Weekly
	lb/day[6,7]		64	96	129	--	--		
TSS[8]	mg/L	M-012B	30	45	60	--	--	24-hr. Composite	Weekly
	lb/day		64	96	129	--	--		
pH	unitless	M-012B	--	--	--	6.5	8.5	Grab	Weekly
Total Coli-form	MPN/100 ml[9]	M-012A	23 (median)	--	230	--	--	Grab	Weekly

1. Reproduced from NPDES No. CA0006017
2. The arithmetic mean of all daily determinations made during a calendar month
3. The arithmetic mean of all daily determinations made during a calendar week
4. BOD: 5-day biochemical oxygen demand at 20ºC
5. mg/L: milligrams per liter
6. lb/day: pounds per day

Table 2-3 Wastewater Treatment Facility Effluent Limitations[1] TOS Detailed Engineering Analysis								
Parameter	Compliance Point	Monthly Average[2]	Weekly Average[3]	Daily Max.	Instantaneous		Sampling	
					Min.	Max.	Type	Frequency

7. Per the current NPDES permit, mass based effluent limitations are based on an average flow rate of 0.257 MGD. During wet weather periods, when the effluent flow rate exceeds 0.257 MGD mass limitations shall be calculated using the actual daily average effluent flow rate, but shall never be based on an effluent flow rate greater than 0.770 MGD.
8. TSS: total suspended solids
9. MPN/100 ml: most probable number per 100 milliliters

2.4 Wastewater Characterization

2.4.1 Influent Flow

Influent WWTF flow characteristics were evaluated based on influent flow provided by TOS for the period from January 2010 through December 2014. Precipitation data was obtained from the National Oceanic and Atmospheric Administration (NOAA), station number GHCND: USC00048045, located in Scotia, California.

2.4.1.1 Average Dry and Wet Weather Flows

Average Dry Weather Flow (ADWF) is the average influent flow during the months of May through October. For the purposes of this analysis, the dry season flow has been defined to correspond with the period of prohibited discharge to the Eel River, May 15 through September 30, except that all of May is included. Due to low regional rainfall averages in October, this month has also been included in the average dry weather flow analysis. Based on analysis of the dry weather season data for the period from 2006 through 2014, the ADWF is approximately 0.124 MGD.

The ADWF can be divided into two descriptive components: base sanitary flow and base infiltration. The portion of treatment plant flow that is entirely attributable to sanitary sewage is known as the base sanitary flow. Because the water usage for Scotia is unmetered, the base flow was estimated based on the minimum repeated flow occurring during the driest month of the year. The base flow for the Scotia WWTF is estimated to be 0.087 MGD.

The difference between the ADWF and the base sanitary flow is the base infiltration rate. Base infiltration rates depend upon such factors as the quality of material, workmanship, age, and condition in the sewers and building connections; maintenance efforts; and groundwater elevations compared with the elevation of the sewer pipes. A base infiltration rate of 20 to 40 gpd per equivalent dwelling unit (gpd/EDU) is considered unavoidable infiltration.

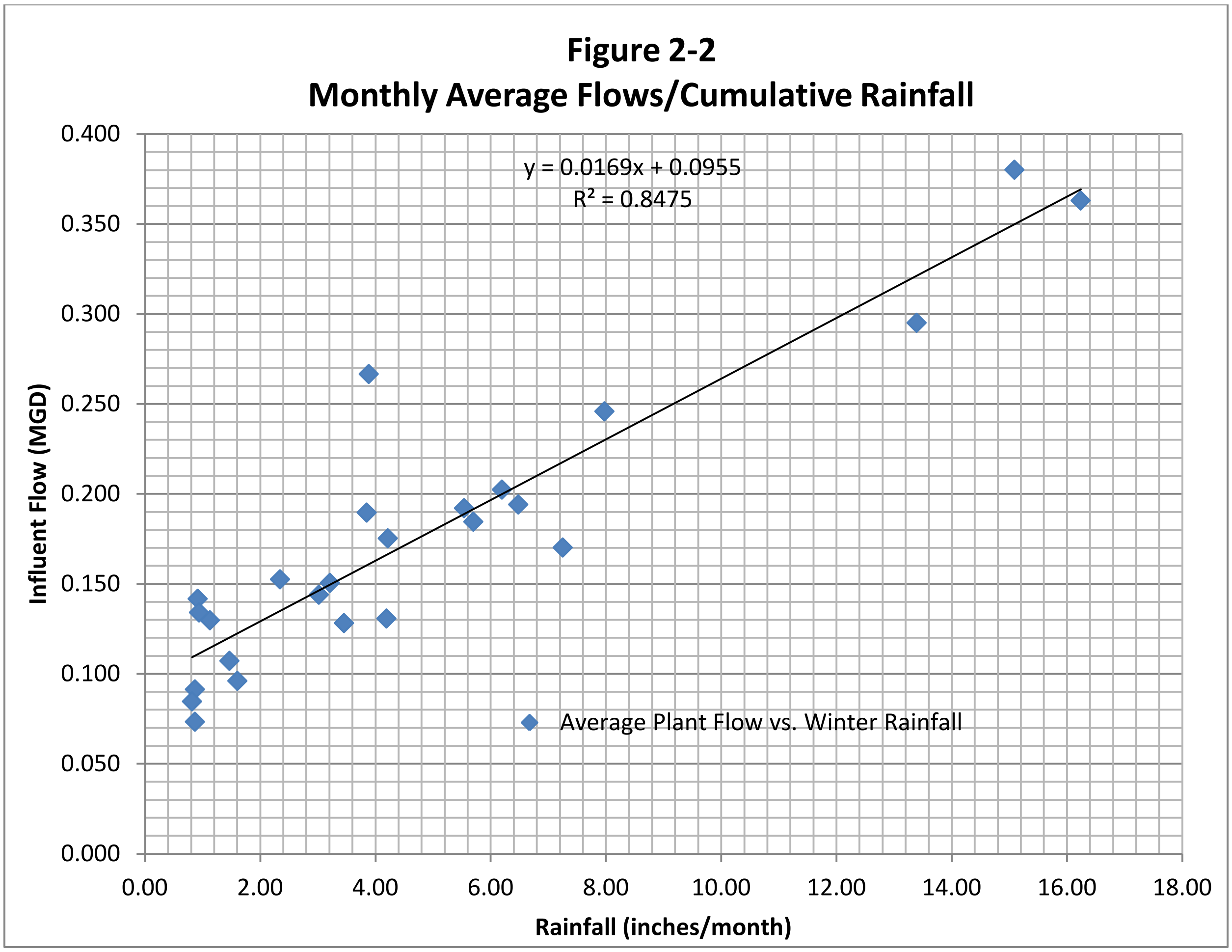
Based on an ADWF of 0.124 MGD, and a base sanitary flow of 0.087 MGD the base infiltration rate at the Scotia WWTF is estimated to be 0.037 MGD. It is estimated that there are 419 EDUs that contribute wastewater to the collection system. This equates to a base infiltration rate of 88 gpd/EDU.

Average Wet Weather Flow (AWWF) is the average influent flow during the months of November through April. Based on analysis of the wet weather season data for the period from 2006 through 2014, the AWWF is approximately 0.192 MGD.

2.4.1.2 Maximum Monthly Dry and Wet Weather Flows

Calculation of maximum monthly flows is based on identifying the monthly rainfall and the monthly average wastewater flows during the months when inflow and infiltration (I/I) impacts the collection system. The linear relationship between monthly rainfall and average wastewater flow is presented graphically and used to predict the flow that corresponds to the cumulative monthly precipitation defined by the required recurrence interval. The methodology employed identifies the seasonal maximum monthly average flow, which has the probability of recurrence once every 5 years during the winter and once every 10 years during the summer.

A graphical representation of flow as a function of cumulative rainfall for the Scotia WWTF is presented in Figure 2-2.



Maximum Month Dry Weather Flow-10 ($MMDWF_{10}$) is the maximum monthly average dry weather flow with a 10% probability of occurrence. This flow represents the wettest dry weather season monthly average flow, which is probabilistically occurring every 10 years. For the purposes of this analysis, the dry season flow has been defined to correspond with the period of prohibited discharge to the Eel River, May 15 – September 30; except that all of May and also October are included in the dry season.

Maximum Month Wet Weather Flow-5 ($MMWWF_5$) is the maximum monthly wet weather average flow with a 20% probability of occurrence. This flow represents the wettest wet season monthly average flow that is anticipated to have a five-year recurrence interval.

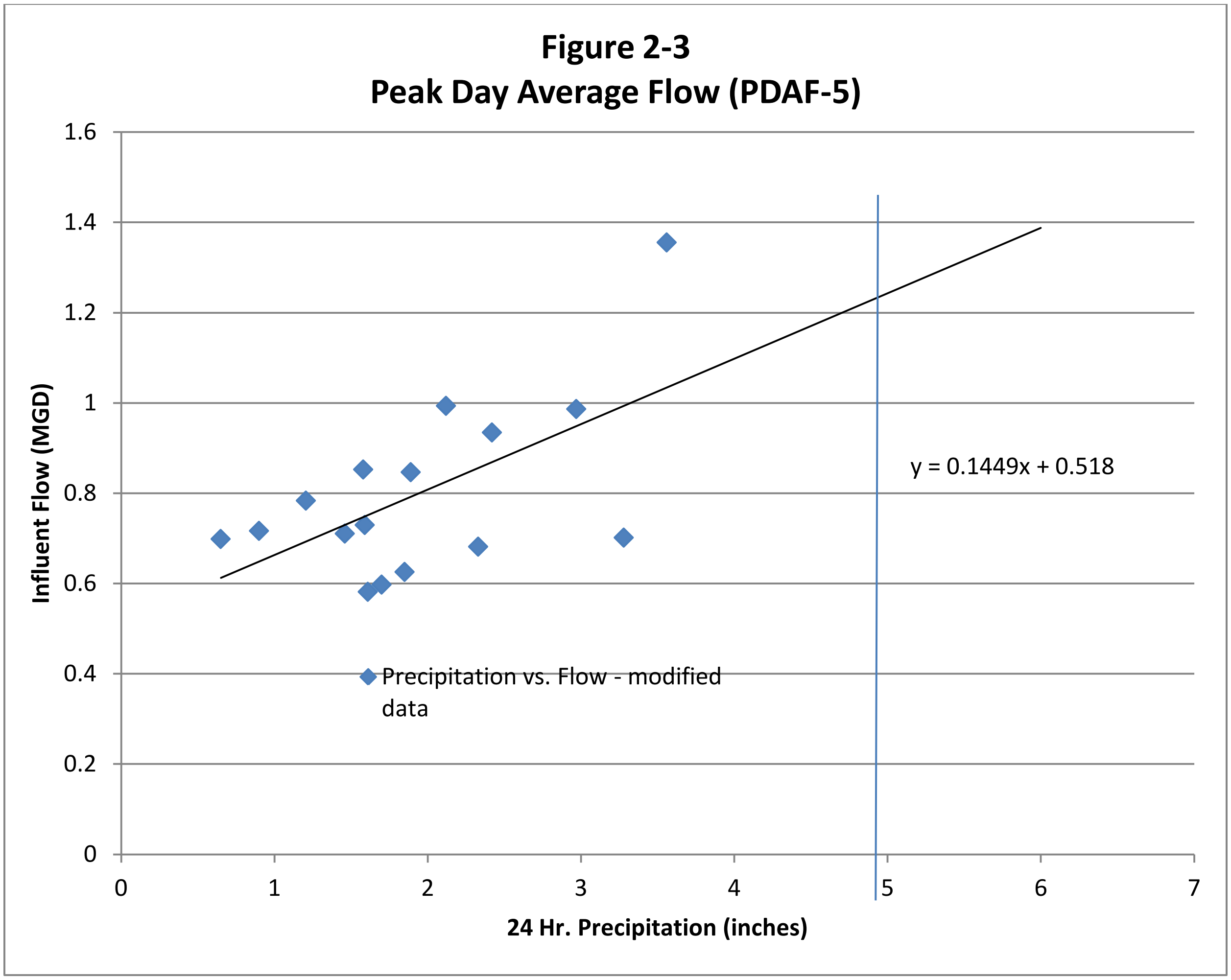
Based on monthly total precipitation data from the Scotia Rainfall Station, the rainfall with a 1-in-10 year recurrence interval in May is 3.65 inches. On Figure 2-2, this corresponds to a $MMDWF_{10}$ of 0.157 MGD. Based on the monthly total precipitation data, the rainfall with a one-in-five year recurrence interval in January is 12.86 inches. On Figure 2-2, this corresponds to a $MMWWF_5$ of 0.313 MGD.

2.4.1.3 Peak Day Average Flow

Peak Day Average Flow-5 ($PDAF_5$) is the largest daily flow associated with a 5-year, 24-hour precipitation event. The peak day average flow has a 0.27% probability of occurrence or 1 day in 365 days of any given year. Estimation of peak day flow is based on a regression analysis of daily plant flows during or immediately following wet season significant rainfall events.

Because the increased influent flow to the WWTF during wet weather is highly correlated with rainfall, evaluation of this regression can be used to define peak day flow associated with a specific rainfall event. The $PDAF_5$ event is determined from a plot of the recorded daily flow that occurred during, or 24 hours after, a significant rainfall event.

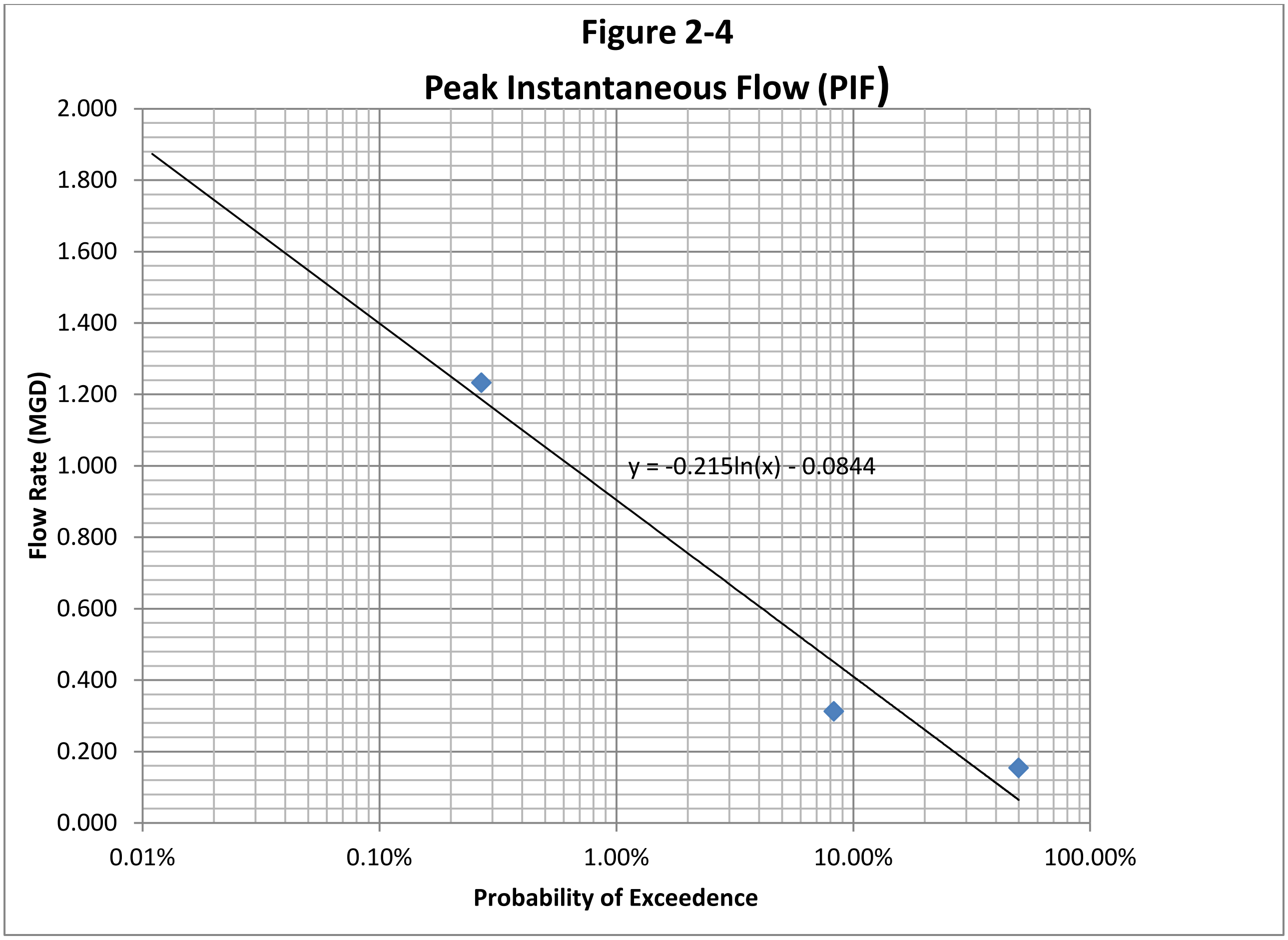
By performing a regression analysis of data, a linear relationship is established, as shown in Figure 2-3. The $PDAF_5$ is based on the intercept of this line with the 5-year, 24-hour precipitation event. To calculate the estimated $PDAF_5$, the 5-year, 24-hour precipitation event for the Scotia area was set equal to 4.93 inches (Period of Record Daily Climate Summary). Based on the regression analysis shown in Figure 2-3, the resulting $PDAF_5$ for a 4.93-inch event is equal to approximately 1.23 MGD.



2.4.1.4 Peak Instantaneous Flow

Peak Instantaneous Flow-5 (PIF_5) is the peak instantaneous flow, which is the highest sustained hourly flow rate during wet weather. The PIF_5 has 0.011% probability of occurrence (1 hour in 8,760 hours of the year). Hydraulic design of channels and pumps at a treatment facility is usually based on this flow.

Determination of the PIF_5 attained during a 5-year PDAF results from a probability projection of the average annual flow (AAF), $MMWWF_5$, peak weekly flow (PW), and $PDAF_5$ parameters. The projection plot shown in Figure 2-4 shows that the PIF_5 for the Scotia WWTF is estimated to be 1.875 MGD.



2.4.1.5 Influent Flow Analysis Summary

A summary of the wastewater flows characterized is included in Table 2-4.

Table 2-4
Wastewater Treatment Facility Influent Flow Summary
TOS Detailed Engineering Analysis

	MGD[1]	gpd/EDU[2]	gpcd[3]
Base Sanitary Flow	0.087	208	65
Base Inflow and Infiltration	0.037	88	28
Average Dry Weather Flow (ADWF)	0.124	296	92
Average Wet Weather Flow (AWWF)	0.192	458	143
Average Annual Flow (AAF)	0.155	370	116
Maximum Month Dry Weather Flow (MMDWF-10)	0.157	375	117
Maximum Month Wet Weather Flow (MMWWF-5)	0.313	747	233
Peak Weekly Flow (PW)	0.768	1,832	573
Peak Day Average Flow (PDAF-5)	1.233	2,943	920
Peak Instantaneous Flow (PIF-5)	1.875	4,476	1,399

1. MGD: million gallons per day
2. gpd/EDU: gallons per day per equivalent dwelling unit (EDU); 419 EDUs associated with sewer (SHN, 2015)
3. gpcd: gallons per capita per day (2.49 persons per household)

Table 2-5 summarizes the wastewater characteristics from each time period, 2003 to 2006, 2006 to 2009, and the current characteristics based on data from 2010-2014. A measurable decrease in both dry and wet weather flows can be seen over time, demonstrating that the 2007 repairs were effective in reducing I/I. The decrease occurring in the 2010-2014 data is likely attributable drought conditions.

Table 2-5
Wastewater Treatment Facility Flows Comparison
TOS Detailed Engineering Analysis

	2003-2006	2006-2009	2010-2014
Flows	**MGD[1]**	**MGD**	**MGD**
Base Sanitary Flows	0.100	0.080	0.087
	0.080	0.076	0.037
Average Dry Weather Flow (ADWF)	0.180	0.156	0.124
Average Wet Weather (AWWF)	0.288	0.242	0.192
Average Annual Flow (AAF)	0.240	0.200	0.155
Maximum Dry Weather Flow-10 (MMDWF-10)[2]	0.280	0.213	0.157
Maximum Month Wet Weather Flow-(MMWWF-5)	0.420	0.367	0.313
Peak Week (PW)	0.750	0.723	0.768
Peak Day Average Flow (PDAF-5)[8]	1.67	1.72	1.23
Peak Instantaneous Flow (PIF)	2.50	2.00	1.88

1. MGD: million gallons per day

2.4.2 Loading

Loadings in Table 2-6 are based on composite sampling conducted on the influent from January 2010 through December 2014.

The only significant industrial discharger to the WWTF is the Eel River Brewery that went online in 2007. The brewery is required to provide pre-treatment to minimize the impact of its discharge on the WWTF. Minimum pre-treatment currently consists of a septic tank, which is intended to prevent shock loading of the treatment facility due to inconsistent organic loading. The septic tank is expected to remove 50 to 75 percent of the total suspended solids (TSS) from the waste stream. Based on previous sampling conducted at the Eel River Brewery in Fortuna, effluent discharged from the septic tank is also expected to have an average biochemical oxygen demand (BOD) concentration of 2,000 milligrams per liter (mg/L).

Table 2-6 Wastewater Treatment Facility Estimated BOD and TSS Loadings TOS Detailed Engineering Analysis			
	EDUs[3]	**BOD[4] (ppd)[5]**	**TSS[6] (ppd)**
Residential	270	127	135
Commercial	44	21	22
Industrial	81	366	418
Institutional	24	11	12
Total EDUS	419	---	---
Average loading	---	525	587
Maximum Loading	---	6,212[7]	15,529[7]

1. Composite sampling conducted on the influent from October 2006 through October 2007
2. Composite sampling conducted on the influent from September 2007 through August 2008
3. EDUs: equivalent dwelling units
4. BOD: biological oxygen demand
5. ppd: pounds per day
6. TSS: total suspended solids
7. The maximum loadings for BOD and TSS occurred on August 16, 2012

2.4.3 Performance

The WWTF is currently meeting permit requirements for loading, concentration, and percent removal for both BOD and TSS. Shock loading from the brewery discharge historically caused the WWTF to have difficulty meeting concentration limits on effluent, but the pretreatment implemented at the brewery appears to have been effective in mitigating the high concentration loading.

Historically the WWTF has also had difficulty meeting the percent removal requirement during high flow events. Due to the dilute influent, 85% removal was difficult to achieve, although concentration limits were met. The addition of the high strength waste from the brewery has ameliorated this difficulty.

A summary of percent removal data from 2010 through 2014 is presented in Table 2-7.

Table 2-7
Wastewater Treatment Facility Removal Percentages for BOD1 and TSS2
TOS Detailed Engineering Analysis

Parameter		2010	2011	2012	2013	2014
BOD	Average	97.3	98.3	98.6	98.4	98.3
	Minimum	87.9	78.4	92.4	88.4	89.8
TSS	Average	97.6	98.0	99.1	98.7	98.9
	Minimum	81.6	60.8	93.1	89.3	95.4

1. BOD: biochemical oxygen demand
2. TSS: total suspended solids
3. Removal Percentage calculated using monthly average influent (M-INF) results and effluent results (M-012B)

2.4.4 Capacity

There are no design documents available that describe the biological design capacity of the WWTF; therefore, general design criteria for each of the treatment systems have been developed based upon published values.

The estimated hydraulic and biological treatment capacity of each treatment system component based on published design criteria is summarized in Table 2-8.

Table 2-8
Wastewater Treatment Facility Design Criteria
TOS Detailed Engineering Analysis

	Description	Design Criteria	Capacity
Preliminary Treatment			
Muffin Monster	3 Horsepower	-	
6-inch flume			Hydraulic capacity 3.6 MGD[1]
Primary Treatment			
Deep Well Pumps (2)	Submersible, 15 hp[2]	-	650 gpm[3] each (0.936 MGD)
Clarifier	Diameter 30 feet Depth 7.25 feet	SOR[4] @ ADWF[5] 800 gpd/SF[6] SOR @ PDAF[7] 900 gpd/SF	0.48 MGD 0.640 MGD
Secondary Treatment			
Shallow Well Pumps (2)	Vertical Turbine Wastewater Power 10 hp	-	Approximately 500 gpm (0.72 MGD)
Trickling Filter	Diameter 44.5 feet Depth 6 to 8 feet Volume 9,330 cf[8] Adjusted Volume: 4,350 cf	Low Rate 25 lbs BOD/d/1,000 cf[9] Intermediate Rate 30 lbs BOD/d/1,000 cf	Low rate : 107 ppd[10] Intermediate Rate 130 ppd
Secondary Clarifier	Diameter 30 feet Depth 7.25 feet	SOR @ ADWF 300 gpd/SF SOR @ PDAF 475 gpd/SF	0.20 MGD 0.40 MGD
Disinfection			
Chlorine Gas	Chlorinators One ton cylinders	-	-
Chlorine Contact Basin (CCB)	Volume 14,000 gallons	CT[11] @ ADWF 40 minutes CT @ PDAF 20 minutes	0.504 MGD 1.0 MGD
Chlorine Contact Basin Pumps (2)	Lead 15 hp Lag 10 hp	-	800 gpm (1.15 MGD) 350 gpm (0.50 MGD) 1,150 gpm (1.65 MGD)
Treatment Ponds			
Ponds	Total Area 2.6 Acres Volume @ 4 ft , 3.39 MG Volume @ 6 ft , 5.09 MG	Loading 15 lbs BOD/d/Acre DT[12] 5-20 Days	39 lbs BOD/day 0.678 MGD 1.0 MGD
Effluent Pump	Line shaft turbine Goulds 40 hp	-	500 gpm (0.72 MGD)
Aerators	Two 7.5 hp		

Table 2-8 Wastewater Treatment Facility Design Criteria TOS Detailed Engineering Analysis			
	Description	**Design Criteria**	**Capacity**
Biosolids			
Digester	Standard Rate Volume 33,500 gals; 4,470 cf	SRT[13] 30-60 days 40-100 lbs VSS[14]/1,000 cf 4-5 cf/capita	116 gpd 178 lbs VSS equivalent population: 1,118
Sludge Pumps (2)	Piston - 15 hp	-	800 gpm (1.15 MGD)

1. MGD: million gallons per day
2. hp: horsepower
3. gpm: gallons per minute
4. SOR: surface overflow rate as a function of depth.
5. ADWF: average dry weather flow
6. gpd/SF: gallons per day per Square Foot
7. PDAF: peak day average flow
8. cf: cubic feet
9. lbs BOD/d/1,000 cf: pounds of biological oxygen demand per day per 1,000 cubic feet; EPA Wastewater Technology Fact Sheet for Trickling Filters EPA 832-F-00-014. Loading based on intermediate filter corrected for specific area of redwood media
10. ppd: pounds per day
11. CT: chlorine concentration over time
12. DT: detention time
13. SRT: sludge retention time
14. VSS: volatile suspended solids

2.5 Basis of Design

2.5.1 Design Flow and Loading

Based on the influent flow analysis presented in Section 2.4, the collection system has excessive rates of I/I. More than 70% of the collection system is in the process of being replaced due to the condition of the pipes and/or location of the pipes within what would be considered private property once the subdivision is complete. It is anticipated that replacement will result in decrease in rates of I/I. Table 2-9 includes estimates of flows based on current and projected EDUs assuming 70% I/I removal. Build-out flows will be the used as the design basis for proposed improvements.

Table 2-9 Projected Design Flows Assuming 70% I/I[1] Removal TOS Detailed Engineering Analysis						
EDUs[2]	**419**					**449**
Equivalent Population	**1,341**					**1,437**
Design Flow	**Existing Conditions**			**70% I/I Removal**		**Build-out**
	MGD[3]	**gal/EDU/day[4]**	**gpcd[5]**	**gal/EDU/day**	**gpcd**	**MGD**
Base Sanitary	0.087	208	65	208	65	0.093
Base I/I	0.037	88	28	26	8	0.012
ADWF[6]	0.124	296	92	234	73	0.105
AWWF[7]	0.192	458	143	283	88	0.127
AAF[8]	0.155	370	116	256	80	0.115
MMDWF[9]	0.157	375	117	258	81	0.116
MMWWF[10]	0.313	747	233	369	115	0.166
Peak Week	0.768	1,832	573	695	217	0.312
PDAF[11]	1.233	2,943	920	1,028	321	0.462

Table 2-9 Projected Design Flows Assuming 70% I/I[1] Removal TOS Detailed Engineering Analysis						
EDUs[2]	**419**					**449**
Equivalent Population	**1,341**					**1,437**
Design Flow	**Existing Conditions**			**70% I/I Removal**		**Build-out**
	MGD[3]	**gal/EDU/day[4]**	**gpcd[5]**	**gal/EDU/day**	**gpcd**	**MGD**
PIF[12]	1.875	4,476	1,399	1,488	465	0.668

1. I/I: infiltration/inflow
2. EDUs: equivalent dwelling units
3. MGD: million gallons per day
4. gal/EDU/day: gallons per equivalent dwelling unit per day
5. gpcd: gallons per capita per day
6. ADWF: average dry weather flow
7. AWWF: average wet weather flow
8. AAF: average annual flow
9. MMDWF: max month dry weather flow
10. MMWWF: max month wet weather flow
11. PDAF: peak day average flow
12. PIF: peak instantaneous flow

Projected organic loadings at the WWTF for build-out conditions were estimated based on the following assumptions:

- full occupancy of residential units,
- an increase in room occupancy and restaurant use at the Scotia Inn, and
- increased brewery production, (6,000 gpd to 1,100 gpd, 83% increase in flow)

The discussion of brewery loadings in Section 2.4.2, and as summarized in Table 2-6, addressed reductions in influent BOD loading at the WWTF assuming varying level of pre-treatment at the Eel River Brewery. Projections of the average daily organic loading at the WWTF are summarized in Table 2-10 based upon implementing pre-treatment at the brewery at the following levels:

- Waste load reduction to a maximum concentration of approximately 5,000 mg/L
- Additional pre-treatment to reduce concentrations to approximately 2,500 mg/L
- Additional pre-treatment to reduce concentrations to approximately 500 mg/L

Table 2-10 Projected WWTF[1] Organic Loadings TOS Detailed Engineering Analysis		
Source/Reduction Level	**BOD[2] (ppd)[3]**	
	Existing	**Projected**
Domestic and Commercial Loading	148	169
Industrial (Brewery) Loading[4]	367	661
Total Community Loading	515	830
Brewery Loading with Waste Load Reduction[5]	256	461
Total with Community Loading	**404**	**656**
Brewery Loading with Waste Load Reduction and Pre-treatment A[6]	128	231
Total with Community Loading	**276**	**426**
Brewery Loading with Waste Load Reduction and Pre-treatment B[7]	26	46
Total with Community Loading	**174**	**241**

Table 2-10
Projected WWTF[1] Organic Loadings
TOS Detailed Engineering Analysis

1. WWTF: wastewater treatment facility
2. BOD: biochemical oxygen demand
3. ppd: pounds per day
4. Based on estimated discharge concentration of 7,320 milligrams per liter (mg/L) BOD, and an existing brewery flow of 6,000 gallons per day (gpd)
5. Assuming 30% waste load reduction (5,120 mg/L BOD), and a flow of 6,000 gpd
6. Assuming 30% waste load reduction and 50% pre-treatment reduction (2,560 mg/L BOD) , and a flow of 6,000 gpd
7. Assuming 30% waste load reduction and 90% pre-treatment reduction (510 mg/L BOD) , and a flow of 6,000 gpd

2.5.2 Seasonal Land Irrigation Requirements

Treated wastewater, along with process water stemming from industrial activities, is pumped to an 18-acre log pond for temporary storage. The log pond water overflows to a clarifier and clarifier effluent is discharged directly to the Eel River or during the prohibited discharge period retained in the log pond. Storage in the log pond is addressed by drawing down the log pond prior to the summer time prohibition. The additional free board is then used as summer time storage. The Town of Scotia has been storing the treated effluent in the log pond during the summer time discharge prohibition successfully since 2012.

In order to conservatively determine the required disposal capacity during the non-discharge period of the year (May 15 through September 30), the projected MMDWF was assumed for the month of May and the ADWF was assumed for the months of June, July, August, and September. Table 2-11 summarizes required disposal capacity for WWTF discharges during the non-discharge season.

Table 2-11
Wastewater Flows During the Non-Discharge Period
TOS Detailed Engineering Analysis

Month[1]	Days/Month	Existing Flow		Projected[2] Flow		Basis of Flow
		gpd[3]	MG[4]	gpd	MG	
May	17	157,000	2.67	115,733	1.97	MMDWF[5]
June	30	124,000	3.72	105,000	3.15	ADWF[6]
July	31	124,000	3.84	105,000	3.26	ADWF
August	31	124,000	3.84	105,000	3.26	ADWF
September	30	124,000	3.72	105,000	3.15	ADWF
Total	**139**	**---**	**17.8**	**---**	**14.8**	**---**

1. Non-discharge period from May 15 through September 30
2. Projected flow includes build-out conditions and 70% I/I reduction
3. gpd: gallons per day
4. MG: million gallons
5. MMDWF: maximum month dry weather flow
6. ADWF: average dry weather flow

2.5.3 Biosolids Production

The current treatment process includes removal of biosolids from the primary and secondary clarifiers and storing the sludge in the inactive anaerobic digester. Current biosolids production is approximately 2,000 gallons per month of liquid containing 3% solids.

2.6 Site Constraints

2.6.1 Floodplain

Any proposed WWTF improvements shall take into account the wastewater treatment facility's location within the 100-year floodplain.

2.6.2 Proximity to Eel River

The HRC tree farm has been proposed as a possible site for disposal of biosolids from the WWTF; however, the site is adjacent to the Eel River and the Eel River is designated as a wild and scenic river. Land application of biosolids under the General Waste Discharge Requirements (General Order) is prohibited within ¼ mile of a wild and scenic river. Although application of Class B biosolids on the HRC tree farm would not be allowed under the General Order, TOS could apply for individual WDRs that would take into account the site-specific conditions for land application of biosolids in that area.

2.7 Basis for Cost Estimates

The estimated construction costs included in the evaluation of alternatives, as presented in Section 2.8, are based on actual construction bidding results from similar work, published cost guides, and other construction cost experience. Reference was made to the available drawings of the existing facilities to determine construction quantities. Where required, estimates were based on preliminary layouts of the proposed improvements.

2.7.1 Contingencies

A contingency factor equal to 20% of the estimated construction cost has been added. Recognizing the cost estimates are based on concept design, allowances must be made for variations in final quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigations, and other difficulties that cannot be foreseen at this time, but may increase final costs.

2.7.2 Engineering

The cost of engineering services for major projects typically include special investigations, a pre-design report, surveying, foundation exploration, preparation of contract drawings and specifications, bidding services, construction management, inspection, construction staking, start-up services, and the preparation of operation and maintenance manuals. Depending on the size and type of project, engineering costs may range from 15 to 25% of the contract cost when all of the above services are provided. The lower percentage applies to large projects without complicated mechanical systems. The higher percentage applies to small, complicated projects. The engineering costs for design and construction of the proposed project will average about 18% of the construction cost.

2.7.3 Legal and Administrative

An allowance of 4% of construction cost has been added for legal and administrative services. This allowance is intended to include internal project planning and budgeting, project administration, liaison, interest on interim financing, legal services, review fees, legal advertising, and other related expenses associated with the project.

2.8 Development and Evaluation of Alternatives

2.8.1 Secondary Treatment Alternatives

The Scotia WWTF is more than 50 years old, and has undergone no significant upgrades. In this section, alternatives for providing reliable secondary treatment during a 20-year planning period are evaluated. All alternatives considered provide the additional treatment capacity to meet required secondary effluent limits, and minimize the risk of the WWTF location in the floodplain. Nutrient removal capabilities and the potential impact of nutrient loading on disposal options are also considered.

2.8.1.1 Tricking Filter Combined Processes

Generally, intermediate rate filters can be loaded up to a maximum of 30 pounds BOD per day per 1,000 cubic feet (lbs BOD/d/1,000 cf). At higher loading rates, filters are considered high-rate filters and secondary quality treatment may not be possible without a second-stage process (EPA, 2000). Prior to the brewery coming online in 2007, loadings on the WWTF trickling filter averaged 33 lbs BOD/d/1,000 cf and maximum day loadings were estimated to be 72 lbs BOD/d/1,000 cf. Current average loadings on the trickling filter are approximately 106 lbs BOD/d/1,000 cf, more than double the recommended loading for secondary treatment.

To treat the projected loadings, the facility could be upgraded to a combined suspended growth fixed/film process in which a suspended growth secondary treatment process follows the fixed film trickling filter to increase BOD removal. Table 2-12 summarizes the loading criteria for suspended growth processes.

Table 2-12
WWTF[1] Organic Loading for Combined Processes
TOS Detailed Engineering Analysis

Process	Acronym	lbs BOD/d/1,000 cf [2]
Trickling Filter	TF	15-40
Biofilter	BF	10-75
Trickling Filter/Solids Contact	TF/SC	20–75
Biofilter/Activated Sludge	BF/AS	75-200

1. WWTF: wastewater treatment facility
2. lbs BOD/d/1,000 cf: pounds BOD per day per 1,000 cubic feet

In the trickling filter/solids contact (TF/SC) process, trickling filter effluent is aerated in a small contact chamber prior to clarification. Solids from the secondary clarifier either are wasted as waste activated sludge (WAS), or returned to this basin as return activated sludge (RAS) as they would be in a conventional activated sludge process.

To create an activated biofilter (ABF), RAS is mixed with primary effluent and recycled over the redwood media to improve performance and sludge settleability. When an ABF is used in combination with an activated sludge basin, the process is called biofilter/activated sludge (BF/AS). The suspended growth portion of the process is an activated sludge basin with a hydraulic residence time of approximately 2 hours. The activated sludge basin required for the BF/AS process is larger than the TF/SC solids contact basin and is designed to provide secondary treatment at high hydraulic and organic loading rates.

Based on the projected loadings, and assuming that average loadings from the brewery are not significantly reduced, the BF/AS would be the recommended combined process. The redwood filter is particularly suited to the use as an activated biofilter. Activated biofilters increase solids settleability and when used in conjunction with an activated sludge basin, yield high quality secondary effluent.

2.8.1.2 Shallow Well Pump Upgrade

The shallow well pumps are responsible for distribution of primary effluent across the trickling filter media. Wetting rates for the combined process are similar to those for trickling filters; and for preliminary design a wetting requirement of 0.75 gallons per minute per square foot (gpm/sf) is assumed (WEF MOP 8, 1998). Based on a trickling filter area of 1,555 square feet (sf) this translates to a required pumping rate of approximately 1,200 gpm.

Variable speed drives installed on the shallow well pumps are recommended to allow for a more continuous filter application rate. These drives will also provide operational control, with the ability to promote controlled sloughing by turning the application rate up to maximum on a weekly basis, but keeping the average rate at minimum to provide maximum wetting efficiency.

2.8.1.3 Secondary Clarifier

Due to the shallow depth (7.25 feet), the existing secondary clarifier is hydraulically overloaded during high flow events. With installation of a new collection system, the projected PDAF of 0.312 MGD, the SOR falls within the recommended rate of 475 gpd/sf for a clarifier of this depth (Metcalf and Eddy, 2003).

2.8.1.4 Estimated Costs

Estimated construction costs for upgrading the existing trickling filter process to a BF/AS system are presented in Table 12-13. These costs include proposed upgrades to the primary treatment system, disinfection system, electrical system, and elevated control room that address deficiencies in the existing system and are required as part of any upgrade. These improvements will be discussed in more detail in the description of the recommended project included in Section 2.12.

Table 2-13
Biofilter Activated Sludge (BF/AS) Process Estimated Costs
TOS Detailed Engineering Analysis

Item (Unit Type)	Unit(s)	Unit Cost	Quantity	Total Cost
Mobilization 12%				$227,600
Equipment				
Headworks Grinder/Ventilation	EA	$35,000	1	$35,000
Primary Clarifier Drive	EA[1]	$85,497	1	$85,497
Shallow Well Pumps	EA	$20,863	1	$20,863
Secondary Clarifier Drive	EA	$85,497	1	$85,497
Blower (7.5 hp)[2]	EA	$11,400	2	$22,799
Diffusers	LS[3]	$34,199	All	$34,199
CCB[4] Pump	LS	$11,400	1	$11,400
Onsite Chlorine Generation	LS	$70,000	100%	$70,000
Pump VFDs[5]	EA	$17,386	4	$69,544
RAS[6] Pumps	EA	$20,519	2	$41,039
WAS[7] Pumps	EA	$11,400	2	$22,799
Equipment Installation	LS	$11,400	All	$11,400
Electrical I/C				
Electrical	LS	$370,489	All	$370,489
Construction				
Railings	lf[9]	$150	100	$15,000
RAS Pump Station	sf[10]	$1,150	250	$287,500
Suspended Growth Reactor	cy	$1,391	60	$83,452
Second Floor Control Room	sf	$398,988	All	$398,988
Stairs	LS	$18,545	All	$18,545
Recirculation from Log Pond Clarifier to WWTP	LF	$100	450	$45,000
Sludge Storage Tank repair (old digester)	LS	$75,000	All	$75,000
Modifications to CCB	LS	$23,181	All	$23,181
Earthwork				
Yard Piping	LS	$57,953	All	$57,953
Excavation/Grading	cy	$29	400	$11,591
Wastewater Treatment Facility Upgrade Estimated Construction Cost Subtotal				**$2,124,336**
Engineering[11] (18%)				$382,380
Contingency (20%)				$424,867
Administration (4%)				$84,973
Total Wastewater Treatment Facility Upgrade Cost				**$3,016,556**

1. EA: each
2. hp: horsepower
3. LS: lump sum
4. CCB: chlorine contact basin
5. VFDs: variable frequency drives
6. RAS: return activated sludge
7. WAS: waste activated sludge
8. cy: cubic yard
9. lf: linear foot
10. sf: square foot
11. Engineering includes design, permitting, and construction management.

2.8.2 New Secondary Treatment System

Because of the extent of the improvements required to upgrade the existing trickling filter system to handle projected loadings, the cost of installing a new treatment system was considered.

Sequencing batch reactors (SBRs) were considered as a cost-effective option for construction of a new secondary treatment system at the existing site. The reactors would have sidewalls approximately 20 feet above the existing ground level, providing protection against flooding that could not be provided in a cost-effective manner for the existing trickling filter system. In addition the SBRs would provide redundancy for major systems, and increase the secondary treatment capacity as required.

SBRs are a variation of the activated sludge process in which the aeration and clarification steps take place in a single clarifier. Installation of an SBR system and associated control room is described below. Improvements to the primary treatment system, disinfection system, and digester would be as described for the BF/AS system.

2.8.2.1 Control Room Upgrades

It is recommended that a second-story addition be constructed above the pump room. The elevated addition would include a control room and blower room. The control room would contain the variable frequency drives (VFDs), electrical control panel, and an enunciator panel.

The blower room would also be included on this upper level. The blowers would provide aeration to the SBR reactors. A preliminary design proposal provided by the manufacturer indicates that three 10-hp blowers would be required, one for each of the reactors plus a back-up.

2.8.2.2 Sequencing Batch Reactors

The installation of SBRs to replace the trickling filter treatment process provides the opportunity to produce a reliable, high-quality secondary effluent. The secondary treatment facilities would include two SBR tanks, and associated blower and pumping facilities.

During peak flow events, the SBR is operated to produce a continuous effluent stream by varying the timing of each cycle in each tank. One tank operates in the fill and react mode, while the other tank is in the settle and decant mode. Automated control valves are used to adjust the cycle time based on the effluent quality desired by the process. Two tanks provide continuous treatment; one tank is filling while the other goes through the treatment cycle.

Preliminary design of the SBR system was based on the construction of two rectangular reactors adjacent to the existing pump room and proposed second story control and blower rooms. The following assumptions were made in developing preliminary sizing. Note that the current BOD loading on the secondary processes exceed the projections used for these preliminary estimates.

- Two rectangular SBR basins would be constructed adjacent to the existing control room.
- Aeration requirements are based on an average organic loading of 261 pounds per day (ppd) BOD.
- Detention time (DT) at the projected MMWWF of 0.18 MGD is 17 hours (DT at 0.25 MGD is 24 hours).

Based on the preceding assumptions and assuming a side wall depth of 18 feet, the reactors would each be 22 feet wide by 44 feet long. Construction of the reactors next to the existing pump room building would require that the existing secondary clarifier be demolished during construction. During construction, it would be necessary to operate the trickling filter as a roughing filter preceding the treatment ponds. A temporary disinfection system would also need to be provided at the end of the treatment ponds.

2.8.2.3 Estimated Costs

Estimated construction costs for installation of a new SBR secondary treatment system are presented in Table 12-14. These costs include proposed upgrades to the primary treatment system, disinfection system, electrical system, and elevated control room that address deficiencies in the existing system and are required as part of any upgrade. These improvements will be discussed in more detail in the description of the recommended project included in later sections.

Table 12-14
Sequencing Batch Reactor (SBR) Estimated Costs
TOS Detailed Engineering Analysis

Item (Unit Type)	Unit(s)	Unit Cost	Quantity	Total Cost
Mobilization 12%				$285,620
Equipment				
Headworks Grinder/Ventilation	EA	$35,000	1	$35,000
Primary Clarifier Drive	EA[1]	$85,497	1	$85,497
Shallow Well Pumps	EA	$20,863	1	$20,863
VFDs[2]	EA	$17,386	2	$34,772
SBR Reactor	LS[3]	$463,621	All	$463,621
Installation	LS	$231,811	All	$231,811
CCB[4] Pump	LS	$11,400	1	$11,400
Onsite Chlorine Generation	LS	$70,000	100%	$70,000
Electrical I/C				
Electrical	LS	$405,669	All	$405,669
Construction				
Tanks	cy[5]	$1,391	250	$347,716
Control and Blower Building	sf[6]	$405,669	All	$405,669
Stairs	LS	$18,545	All	$18,545
Grating	sf	$46	525	$24,340
Railing	lf[7]	$150	200	$30,000
Decant Clear Well	cy	$1,391	60	$83,452
Digester Repair	LS	$75,000	All	$75,000
Modifications to Chlorine Contact Basin	LS	$23,181	All	$23,181
Earthwork				
Yard Piping	LS	$57,953	All	$57,953
Site Work (Demo)	LS	$46,362	All	$46,362
Estimated Construction Cost Subtotal				**$2,756,471**

Table 12-14 Sequencing Batch Reactor (SBR) Estimated Costs TOS Detailed Engineering Analysis				
Item (Unit Type)	**Unit(s)**	**Unit Cost**	**Quantity**	**Total Cost**
Engineering[8] (18%)				$496,164
Contingency (20%)				$551,294
Administration (4%)				$110,259
Total Wastewater Treatment Facility Upgrade Cost:				**$3,914,188**

1. EA: each
2. VFDs: variable frequency drives
3. LS: lump sum
4. CCB: chlorine contact basin
5. cy: cubic yard
6. sf: square foot
7. lf: linear foot
8. Engineering includes design, permitting, and construction management.

2.8.3 Roughing Filter/Facultative Ponds

In the past, the treatment ponds have functioned as a failsafe mechanism for the Scotia WWTF in removing solids not captured by the trickling filter solids contact process. One alternative that needed to be evaluated was to determine if the treatment ponds could provide adequate additional secondary treatment without modification of the trickling filter treatment process.

The trickling filter is operating as a high rate or roughing filter, with BOD loading rates of greater than 100 ppd/1,000 cf, and can be expected to achieve BOD removal rates of only 40% reliably. Currently, the average influent BOD concentration at the WWTF is 392 mg/L (510 ppd @ ADWF). Assuming the primary clarifier removes 25% of influent BOD, and the trickling filter subsequently removes 40% of the remaining load, the average BOD loading rate on the treatment ponds would be 176 mg/L (229 ppd @ ADWF).

The treatment ponds are currently full of biosolids, and although the treatment ponds are reportedly more than 10 feet deep in some sections, depth of clear water above the sludge blanket is only 2 to 4 feet. If the sludge were to be removed, the treatment ponds would function as facultative treatment ponds, with a design loading of 35 lbs BOD/d/acre. Loading on the ponds at approximately 73 lbs BOD/d/acre would exceed the optimum loading for un-aerated facultative ponds. With supplemental aeration (existing in Pond #1) the loading would be within the 50 to 100 lbs BOD/d/acre recommended range.

Although the BOD loading rate on the treatment ponds would be feasible for an aerated facultative pond system, it would create an unacceptable BOD demand on the disinfection system. Without the aerated BF/AS addition to the trickling filter system, suspended solids would also be high. To achieve compliance, a new the disinfection system would be required and the point of compliance would have to be moved to the end of the treatment ponds.

2.8.4 Evaluation of Secondary Treatment Alternatives

Two mechanical alternatives were evaluated for upgrading the existing WWTF to meet required hydraulic and organic loadings. A third alternative employing the existing filter as a roughing filter and adding aeration to the treatment ponds was also evaluated.

Not improving the trickling filters but relying on aerators to achieve required BOD removal is not a satisfactory alternative because reduced secondary treatment would negatively impact disinfection. Even if a filtration system were to be installed following the secondary clarifier to remove TSS, the high BOD demand would make achieving required disinfection difficult and would create high levels of toxic chlorinated organic compounds called trihalomethanes.

The preferred option for upgrading the secondary treatment system is to modify the existing trickling filter system to operate as a BF/AS process. When compared to the SBR alternative, the advantages of this system include the following:

- Significantly lower capital costs ($880,000)
- Less operational complexity
- Lower operational costs

2.8.5 Treatment Pond Improvements/Tertiary Treatment Alternatives

The treatment ponds provide enhanced BOD and nutrient removal following the secondary treatment system, and also provide some redundancy for secondary treatment system components. Two alternatives for improving treatment in the ponds were evaluated:

1. Sludge removal, possible aeration, and use for storage
2. Converting the ponds to a series of wetland treatment cells

2.8.5.1 Treatment Pond Improvements

Sludge removal from the treatment ponds, while not eliminating the need for secondary treatment improvements at the mechanical plant, will improve the treatment pond effectiveness and is necessary to eliminate washout of suspended solids. However, because of long DTs in summer months, suspended algae growth can be a problem. Additional treatment pond improvement recommendations include:

- Supplemental aeration to promote mixing and limit algae growth in the surface layer
- Relocating the intake of the effluent pumps in Pond 3 to 4 feet below the surface

There is the potential to use the treatment ponds for storage during the non-discharge season, eliminating the need to discharge to the log pond. This would require modifying the discharge of Ponds 1 and 2 so that the water level in both ponds could be drawn down.

The facultative ponds will be effective in removing BOD, but only partially effective in removing nitrogen and phosphorous. In a pond system, nitrogen is removed through the processes of nitrification and volatilization of ammonia with little opportunity for denitrification. Assuming long DTs are experienced during the summer months, expected nitrogen removal in the ponds would be 30 to 40%.

2.8.5.2 Wetland Treatment Cells

The other alternative is to convert the treatment ponds into a series of wetland treatment cells. Wetland treatment cells are designed to remove nutrients. Free water surface wetlands have alternating deep and shallow cells for nitrification of ammonia and denitrification of nitrates. The submerged vegetation and pondweed in the deep cells is rooted in the bottom with some floating leaves. This vegetation provides air in the water column and a submerged surface to which nitrifiers to attach. The shallow cells are planted with bulrush and are very effective at denitrification.

Preliminary estimates indicate that there is sufficient area to remove nitrogen to minimum levels, although there will be some release during periods of active decomposition during autumn. Phosphorous removal is facilitated in the wetlands through deposition and plant uptake.

2.8.5.3 Estimated Costs

Table 2-15 summarizes estimated project costs for the three treatment pond improvement options. For Options 1 and 2, it is assumed that if secondary effluent quality is achieved in the mechanical plant system, the treatment ponds will not need to be lined.

Table 2-15
Treatment Pond Improvement Options
TOS Detailed Engineering Analysis

Option	Description	Total Cost[1]
1	Sludge Removal Only	$1,128,106
2	Sludge Removal, Aeration, Storage	$1,175,280
3	Conversion to Wetland Treatment Cells	$317,883

1. costs are presented in 2015 dollars

The cost of sludge removal was based on an estimated volume of 5,835,000 gallons of biosolids (SHN, 2006) at a concentration of 4% or 973 dry tons. Based on similar projects in the area, the estimated cost for dewatering and disposal of biosolids is $1,000 per dry ton.

The storage option (Option 2) includes sludge removal plus the cost of modifying the culverts connecting the three ponds.

The wetland treatment cells option (Option 3) assumes that wetlands will be constructed in the 2 to 4 feet of free water surface that currently exists in the treatment ponds, using the existing pond footprint. The existing biosolids will be dried in place and capped.

2.9 Effluent Disposal

The Scotia WWTF is currently permitted to discharge to the Eel River, through the log pond, from October 1 through May 14 (the discharge period), provided that the treated wastewater discharge does not exceed 1% of the Eel River flow on either a daily or monthly basis. From May 15 through September 30 (the non-discharge or discharge prohibition period), treated wastewater effluent is stored in the log pond.

SHN has evaluated a series of alternatives for summer disposal, including the following:

- Log pond storage
- Municipal and industrial reuse

2.9.1 Log Pond Storage

For several years, the TOS has stored water in the log pond during the summer months with either no discharge or very limited discharge to the percolation pond. In this section, a water balance is developed to determine the required storage volume for zero discharge during the summer months.

2.9.1.1 Log Pond Water Balance

A water budget can be developed for any hydrologic system to account for flow pathways and storage components. The water budget accounts for conservation of mass during a defined period of time:

$$I = Q + \Delta S$$

where:

I	=	Inflow
Q	=	Outflow
ΔS	=	Change in storage

The project-specific water balance equation is described as:

$$PPT_{in} + Q_{WW} + Q_{IND} = E_s + Q_{ROADS} + \Delta S$$

where:

PPT_{in}	=	Precipitation inflow to the log pond storage reservoir
Q_{WW}	=	Treated wastewater effluent
Q_{IND}	=	Industrial wastewater from cogeneration plant
E_s	=	Evaporation from the log pond storage reservoir and ponds
Q_{ROADS}	=	Withdrawals for dust suppression from the log pond storage reservoir
ΔS	=	Accumulated summer storage to discharge to the Eel River during the discharge period

2.9.1.2 Treated Wastewater Flow into Storage, Q_{WW}

As detailed in Section 2.5.2, the total wastewater flow during the non-discharge period (Q_{ww}) is estimated to be 17.8 MG. It is estimated that the wastewater flow for this period may be reduced to 14.8 MG after proposed improvements to the collection system have been implemented.

2.9.1.3 Industrial Process Water Flow into Storage, Q_{IND}

The Scotia cogeneration plant also discharges process water to the storage pond during the summer months at an estimated rate of 90,000 gpd. The total industrial process water discharge to the log pond during the summer discharge prohibition period is estimated to be 12.5 MG.

2.9.1.4 Precipitation into Storage, PPT_{in}

Precipitation catchment areas were measured from the Scotia northwest digital orthophoto quarter quadrangle (NAIP, 2005) using ArcGIS. The WWTF precipitation catchment areas include the log pond, three treatment ponds, and approximately 5 additional acres that drain to the ponds. Table 2-16 summarizes these catchment areas.

Table 2-16 Storage Rainfall Catchment Areas TOS Detailed Engineering Analysis	
Rainfall Catchment Component[1]	**Surface Area (sf)[2]**
Log Pond	773,190
Treatment Pond 1	22,500
Treatment Pond 2	40,500
Treatment Pond 3	37,500
Additional Catchment	217,800
Total	**1,091,490**

1. Includes areas where precipitation contributes to storage requirements
2. sf: square feet

For the purposes of the water balance analysis, the log pond is considered the storage reservoir. The amount of water entering the storage reservoir is dependent upon the total catchment area, which is approximately 1.1 million square feet. Table 2-17 summarizes the flow rates into the log pond due to precipitation. Average precipitation rates are assumed except for the shoulder month[1] of May when discharge is not allowed.

Table 2-17 Precipitation into Storage TOS Detailed Engineering Analysis		
Month	**Average Precipitation (in/month)[1]**	**Precipitation Rate into Log Pond (gpd)[2]**
May	3.99[3]	87,569
June	0.63	14,288
July	0.07	1,536
August	0.22	4,828
September	0.59	13,380

1. in/month: inches per month
2. gpd: gallons per day
3. Precipitation for May based on 10-year return interval

2.9.1.5 Evaporation from Storage, E_s

The WRCC Ferndale Substation (Ferndale 2NW) is the closest proximity weather station to Scotia, with average monthly pan evaporation measured from 1963 to 1973. The California Irrigation Management Information System (CIMIS) locates Ferndale within reference evapotranspiration (ET_0) Zone 1, described to be coastal plains, heavy fog belt (CIMIS, 1999). It appears that the CIMIS Zone 1 ET_0 estimates the measured pan evaporation in Ferndale accurately, indicating that the measured pan evaporation and reference evapotranspiration are roughly equivalent (Grismer et al., 2002).

1 "Shoulder Month" refers to the part of May, just outside the discharge prohibition period.

The CIMIS ET_0 Zones Map locates Scotia at the western edge of ET_0 Zone 3, which is described as coastal valleys and plains and north coastal mountains (CIMIS, 1999). Scotia is located near the border of the CIMIS ET_0 Zones 1 and 3, evaporation rates in Scotia were estimated by averaging the ET_0 rates for these two zones. These estimates were used to calculate evaporation from the storage reservoir and treatment ponds (E_s); results are shown in Table 2-18:

$$E_s = ET_0 \times A$$

where:

E_s	=	Log pond and wetland evaporation rate (inches per month [in/mo])
ET_o	=	Reference evapotranspiration rate (in/mo)
A	=	Area of storage surface comprised of log pond and wetlands (sf)

Table 2-18
Evaporation Rate from Storage Reservoir
TOS Detailed Engineering Analysis

Month	ET_0[1] CIMIS[2] Zone 1 (in/mo)[3]	ET_0 CIMIS Zone 3 (in/mo)	ET_0 Scotia (in/mo)	Area, A (sf)[4]	Evaporation, E_s (gpd)[5]
May	4.03	5.27	4.65	873,690	81,690
June	4.50	5.70	5.10	873,690	92,582
July	4.65	5.58	5.12	873,690	89,859
August	4.03	5.27	4.65	873,690	81,690
September	3.30	4.20	3.75	873,690	68,075

1. ET_0: evapotranspiration rate
2. Source: CIMIS ET_0 Map (1999)
3. in/mo: inches per month = 0.03 in/day
4. sf: square feet
5. gpd: gallons per day

2.9.1.6 Dust Suppression, Q_{ROADS}

Under existing conditions, it has been estimated that the withdrawals for dust suppression from the log pond storage reservoir average approximately 60,000 gpd. Under projected conditions, this estimate has been reduced to 25,000 gpd to account for proposed road improvements in town that would no longer require water application for dust suppression.

2.9.1.7 Storage Requirements, ΔS

Storage requirements were calculated for the non-discharge period (May 15 through September 30). The required monthly storage space was determined by dividing the monthly accumulated precipitation and discharge volume by the surface area of the log pond, 773,190 sf (17.75 acres). Table 2-19 summarizes the findings of the storage requirements for the summer non-discharge period.

Table 2-19 Storage Requirements During Non-Discharge Period TOS Detailed Engineering Analysis								
Month	**Q_{WW}[1]**	**Q_{IND}[2]**	**PPT_{in}[3]**	**E_S[4]**	**Q_{ROADS}[5]**	**ΔS[6]**	**Storage**	**Elev. Change**
	gpd[7]	**gpd**	**gpd**	**gpd**	**gpd**	**gpd**	**gallons**	**feet**
May	213,000	90,000	87,569	81,690	25,000	283,879	4,825,943	0.83
June	156,000	90,000	14,288	92,582	25,000	142,706	4,281,180	0.74
July	156,000	90,000	1,536	89,859	25,000	132,677	4,112,987	0.71
August	156,000	90,000	4,828	81,690	25,000	144,138	4,468,278	0.77
September	156,000	90,000	13,380	68,075	25,000	166,305	4,989,150	0.86
							Total	**3.91**

1. Q_{WW}: wastewater discharge
2. Q_{IND}: Industrial (cogeneration plant) discharge
3. PPT_{in}: precipitation into storage
4. E_s: evaporation from storage
5. Q_{ROADS}: withdrawals for dust suppression
6. ΔS: change in storage
7. gpd: gallons per day

The water balance indicates a required storage of 22.7 MG, which corresponds to approximately 4.0 feet of water in the log pond. Bob Vogt, Director of Environmental Compliance for the owner at that time (PALCO), indicated to SHN in a conversation on March 16, 2007, that it is possible to draw down the log pond to provide the required storage.

2.9.1.9 Log Pond Water Quality

In November 2007 and again in 2008, the pH of the log pond effluent exceeded 8.5, the maximum permitted for discharge to the Eel River. These instances followed extended periods of no discharge, when effluent was stored in the log-pond. The high pH may be due to the presence of high concentrations of algae in the log pond. Algae uses up CO_2 during the day, which can cause diurnal cycling of pH.

Nutrients in the WWTF effluent may be promoting algae growth in the lagoons. The elevated temperature of water discharged from the cogeneration plant may also be a factor; long streamers of algae have been observed in the channel discharging into the log pond. Recommendations that may improve water quality in the log-pond include:

- Enhancing the wetlands around the discharge from the tertiary ponds
- Providing shading of algae using floating rafts or pondweed
- Providing sprinklers to cool discharge from cogeneration plant

2.9.2 Alternatives Evaluation

The feasibility of disposal for the discharge from the WWTF during the period from May 16 through September 30 (when discharge to the Eel River is prohibited) has been evaluated for extended storage in the log pond.

Storage of effluent in the log pond during this period is feasible and has been practiced for several years. However, following extended periods of no discharge, the pH of discharge from the log pond effluent has exceeded 8.5, the maximum permitted for discharge to the Eel River. It is assumed that high pH may be due to the presence of high concentrations of algae, which uses up CO_2 during the day and can cause diurnal cycling of pH.

Increased algae growth in the pond has been observed to be the result of elevated temperatures from the non-contact cooling water use at the cogeneration plant. However, the nutrients in the WWTF discharge may also be contributing to algae growth. Any strategy relying on seasonal storage has the potential to have a negative impact on the water quality of the log pond, so mitigating these impacts must be part of a long-term strategy.

The water balance on the log pond indicated that there was sufficient storage in the log pond to store the treated effluent if the pond was reduced by 4 feet prior to the no discharge period. No modifications to the pond would be necessary to provide the required storage. Additional improvements could mitigate negative impacts to water quality and are capital improvements that could feasibly be implemented right away, while the process of obtaining funding for upgrades to the WWTF is underway. These improvements include:

- Enhancing the wetlands around the discharge from the treatment ponds
- Providing shading of algae through using floating rafts or pond-weed
- Providing sprinklers to cool discharge from cogeneration plant
- Aeration of log pond
- Recirculate water from log pond back through the Wastewater Treatment Plant

2.10 Biosolids Management

This section includes an analysis of alternatives for the handling and disposal of biosolids from the Scotia WWTF. The feasibility of the following methods of disposal was considered:

- Beneficial Reuse
- Class A biosolids to be sold or given away (requires dewatering)
- Landfill Disposal (requires dewatering prior to hauling)
- Contract disposal

2.10.1 EPA Compliance

The biosolids must comply with CFR 40 Part 503 requirements for Class B biosolids prior to land application. Biosolids are considered a Class B biosolid if they are treated with a process to significantly reduce pathogens (PSRP), such as anaerobic digestion. The anaerobic digester at the Scotia WWTF is considered a PSRP if operated to provide the following detention times (DT) and operating temperatures:

- Mean Cell Residence Time 15 days at 35 degrees to 55 degree C (131 degrees F).
- 60 days at 20 degrees C (68 degrees F).

The standard rate digester is operated with continuous feed and periodic sludge removal. The operating volume is maintained by adjustable level overflow, which returns supernatant back to the plant influent. Because the digester has a floating cover, the available volume in the digester is also variable. Based on a projected sludge volume of 1,200 gpd, the digester will provide an SRT of approximately 15 days, not including thickening provided by decanting supernatant. Although with thickening and recycling the DT can be increased, the digester must be heated to meet the time and temperature requirement cited above for a PSRP.

The current anaerobic digester is not functional and will require a reconstruction effort to put the unit back into service.

The biosolids will also need to meet CFR 40 Part 503 requirements for reducing vector attraction before they can be land applied. Vector attraction reduction options are listed in Section 2.10.3.1. Properly operated, the anaerobic digester should be able to meet the 38% reduction in volatile solids required for Option 1. If vector attraction reduction requirements are not met by volatile solids reduction in the digester, then additional treatment may be required, for example employing lime stabilization.

2.10.2 Sludge Dewatering Options

Sludge dewatering reduces the volume of biosolids that must be stored and transported. Except in special cases where the land application site is near the treatment facility, it is more cost effective to dewater biosolids than pay for the additional cost of handling liquid volumes. This section evaluates the construction of drying beds for dewatering the biosolids at the Scotia WWTF.

2.10.2.1 Drying Beds

Dewatering biosolids through the use of drying beds is a feasible dewatering alternative as land is readily available and operation and maintenance of drying beds is relatively low. The drying beds would have to be covered because of the high precipitation rates in Humboldt County. The sides of the drying beds would remain open to allow for free air flow. Table 2-20 summarizes the estimated costs for construction of drying beds at the Scotia WWTF.

Table 2-20
Estimated Costs for Construction of Drying Beds
TOS Detailed Engineering Analysis

Item	Unit(s)	Quantity	Unit Cost	Total Cost
Mobilization (12%)				$19,000
Asphalt Lining	sf[1]	6300	$6	$36,510
Gravel	cy[2]	121	$37	$4,488
Sand	cy	121	$37	$4,488
Sludge Vault	LS[3]	4	$8,693	$34,772
Underdrain System	LS	1	$13,909	$13,909
Cover	sf	6300	$10	$63,528
Construction Subtotal				**$176,695**
Contingency (20%)				$35,339
Engineering (18%)				$31,805
Administration (4%)				$7,068
			Total Estimated Cost of Drying Beds	**$250,907**

1. sf: square foot
2. cy: cubic yard
3. LS: lump sum

2.10.3 Disposal of Dewatered Biosolids

Dewatered biosolids can be further treated to comply with Class A requirements making it suitable for application to residential public and commercial landscaping and nurseries. Alternatively, dried Class B biosolids could be land applied at the HRC tree farm or hauled to a licensed landfill. Agricultural sites in the north coast region that are suitable for the application of Class B biosolids are increasingly difficult to find and permit.

2.10.3.1Class A Biosolids

Under 40 CFR Part 503, the creation of Class A biosolids requires that a process to further reduce pathogens (PSRPs) is implemented and that vector attraction reduction measures are carried out concurrently. PFRPs include composting, heat drying, and lime treatment. There are seven recommended PSRPs listed in the summary of Part 503 regulations. Alternative 2: "Biosolids Treated in a High pH-High Temperature Process," can be implemented with the addition of lime to raise the pH and temperature. The elevated pH will also provide compliance with vector attraction reduction requirements.

Lime addition for compliance with 40 CFR Part 503 regulations is recommended over composting for its reliability in achieving compliance with Class A requirements. Lime addition is also less operationally complex and less expensive than heat drying technologies.

The use of quick lime is recommended because it is less expensive and easier to handle than hydrated lime. Also, the heat produced during hydration, when the slaked lime is added to the dewatered cake, can enhance pathogen destruction. Class A biosolids are suitable for application on nurseries or residential landscaping. It is assumed that if the product is made available, public demand will exceed production. Table 2-21 summarizes the estimated costs for installation of a chemical mixing system for lime addition.

Table 2-21
Cost of Chemical Mixing System for Lime to Produce Class A Biosolids
TOS Detailed Engineering Analysis

Item	Unit(s)	Quantity	Unit Cost	Total Cost
Mobilization (12%)				$24,000
Pug mill	EA[1]	1	$17,386	$17,386
Screw conveyor	EA	1	$4,636	$4,636
Electrical	LS[2]	All		$0
Solids Building	sf[3]	300	$371	$111,269
Cover	sf	2100	$30	$63,771
Construction Subtotal				**$221,062**
Contingency (20%)				$44,212
Engineering (18%)				$39,791
Administration (4%)				$8,842
		Total Estimated Cost of Lime Mixing System		**$313,907**

1. EA: each 2. LS: lump sum 3. sf: square foot

2.10.3.2 Hauling to Landfill

The nearest landfill that accepts Class B biosolids is located in Redding, California. Estimated hauling and dumping cost for biosolids is $400/dry ton. Based on an estimated biosolids yield of 65.7 tons, the annual cost is estimated to be $26,280/yr. This equates to a 20-year present value of $391,000 based on an annual interest rate of 3%.

2.10.3.3 Contract Disposal

The Town of Scotia, LLC has been contracting with a local licensed septic service to remove and dispose of the sludge stored in the old inoperative anaerobic digester. The digester was retrofitted with a new fitting so that the sludge can be removed periodically from the digester for transport to an appropriate biosolids handling facility. The septic service has a permitted dewatering process pad and all dewatered biosolids are sent to a licensed facility for disposal.

2.10.4 Biosolids Disposal Cost Comparison

In the cost comparison shown in Table 2-22, the Class A sludge option includes costs to construct drying beds and the lime mixing system. The landfill option includes costs to construct drying beds and the estimated landfill disposal costs.

Table 2-22
Summary of Estimated Cost for Biosolids Disposal Options
TOS Detailed Engineering Analysis

Option	Description	Total Cost
1	Class A Sludge [1]	$564,814
2	Transport to Landfill[2]	$628,400
3	Contract Disposal[3]	$218,000

1. Drying beds plus lime addition
2. $237,400 cost for new drying beds plus 20–year present value of annual hauling and disposal
3. Continue to Contract for monthly removal and disposal to authorized land fill (20-year present value of sludge disposal ($143,000) along with $75,000 repairs to old Digester)

2.11 Development and Evaluation of Complete Alternatives

2.11.1 Common Parameters

All of the complete alternatives must address deficiencies in the existing facility as described in Section 2.2. Based on current loadings, the preferred method of improving secondary treatment is the BF/AS system.

2.11.2 Development of Combined Alternatives

This section combines treatment and disposal alternatives. In all the options, the BF/AS is assumed to be the preferred secondary treatment option. The four alternatives are shown in Table 2-23. The estimated costs for the four alternatives are summarized in Table 2-23. The lowest cost alternatives are those that use the log pond for seasonal, dry-weather storage without additional irrigation.

Table 2-23 Development of Complete Alternatives TOS Detailed Engineering Analysis			
Alternative	**Secondary Treatment**	**Tertiary Treatment**	**Disposal Storage**
1	BF/AS[1]	Tertiary Ponds With Sludge Removal	Seasonal Log Pond Storage
2	BF/AS	Tertiary Ponds With Sludge Removal	Seasonal Tertiary Pond Storage
3	BF/AS	Conversion to Wetlands (No Sludge Removal)	Seasonal Log Pond Storage
4	BF/AS	Tertiary Ponds With Sludge Removal	Seasonal Log Pond Storage
1. BF/AS: biofilter/activated sludge			

Table 2-24 Summary of Complete Alternatives and Estimated Costs TOS Detailed Engineering Analysis						
Alternative	**Secondary Treatment**	**Tertiary Treatment**			**Disposal**	
		Facultative	**Storage Pond**	**Wetlands**	**Storage/Contract Disposal**	**Total Est. Cost**
1	$3,016,557	$1,128,106	---	---	$143,000	**$4,287,663**
2	$3,016,557	---	$1,175,280	---	$143,000	**$4,334,837**
3	$3,016,557	---	---	$317,883	$143,000	**$3,477,440**
4	$3,016,557	$1,128,106	---	---	$143,000	**$4,287,663**

1.11.3 Matrix Evaluation

An evaluation of the preferred disposal alternative is summarized in Table 2-25. In this alternative matrix, the advantages and disadvantages of the alternatives were quantified by ranking them from 1-4, with the highest score being the most favorable.

Table 2-25 Alternatives Evaluation Matrix TOS Detailed Engineering Analysis Plan						
Alternative	**Cost**		**Reliability**	**Regulatory Issues**	**Operational Complexity**	**Total Score[1]**
1	$4,287,663	1	4	2	2	9
2	$4,334,837	1	2	2	2	7
3	**$3,477,440**	**4**	**3**	**4**	**4**	**15**
4	$4,287,663	3	3	4	4	14
1. Highest score represents most preferable alternative, with a maximum possible score of 16.						

Alternative 1 scored higher for system reliability because of the redundancy supplied by providing for irrigation and disposal. However, Alternative 3 is the highest ranking alternative with the lowest cost, operational complexity, and lack of regulatory issues and is considered the preferred alternative. Alternative 3 also provides for the conversion of the treatment ponds to wetlands treatment. The wetlands would provide enhanced BOD and nutrient removal and ensure that the wastewater discharged to storage is not providing any nutrient enrichment or contributing to algae growth in the log pond.

2.12 Recommended Plan

This section presents the plan to correct deficiencies in the TOS wastewater treatment, and disposal systems and defines the upgrades required to enable the WWTF to meet secondary permit limits for the 20-year planning period.

2.12.1 Pre-treatment and Primary Treatment

Influent enters the WWTF through two gravity sewer mains that discharge into a headworks channel provided with a grinder and Parshall flume for flow metering. From the headworks, the sewage flows into a wet-well (called the deep well), where it is pumped to the primary clarifier. The effluent from the primary clarifier discharges to a second wet-well (called the shallow well) before being pumped to the trickling filter for secondary biological treatment.

Recommended upgrades to the primary treatment system include:

- Clarifier drive replacement
- Installation of VFDs on deep well pumps

2.12.2 Control Room

To minimize the impact of the facility's location in the floodplain, it is recommended that an elevated control room be constructed over (or partially over and adjacent to) the existing pump house structure. The elevated room would be used for new equipment including VFDs and a new electrical control panel. Structural modifications required for the elevated control room include an external support structure for the second story. .

2.12.3 Secondary Treatment System

Recommended improvements to the secondary treatment system address system deficiencies, provide increased treatment capacity, and minimize the risk of the facilities location in the floodplain.

Project components include the following:

- Replacement of shallow well pumps with submersible pumps not impacted by flooding
- Installation of VFDs on the shallow well pumps
- Construction of an activated sludge suspended growth aeration basin or solids contact basin following the trickling filter
- Installation of RAS pumps and a pump station to transfer solids from secondary clarifiers to the Shallow Well (as part of the BF/AS process) or to the solids contact basin (in the case of TF/SC process)
- Installation of blowers for the solids contact process with controls installed in a new control room
- Install "onsite" chlorine generation system to replace gaseous system

2.12.3.1 Treatment Capacity

At current BOD loadings, the preferred method of improving secondary treatment is the BF/AS process. A BF/AS system would increase the loading capacity on the trickling filter to 75 to 200 lbs BOD/d/1,000 cf; current loadings on the filter average 107 ppd with a typical loading rate of 25 to 30 lbs BOD/d/1,000 cf.

Projected loadings are based on the design influent BOD loadings as summarized in Section 2.4.2. If it is assumed that waste discharges from the brewery will have reduced concentrations (due to the implementation of waste load reductions), but that production will increase, loadings on the filter will be approximately 110 lbs BOD/d/1,000 cf.

If the brewery installs a UASB or other sophisticated pre-treatment system to reduce concentrations of brewery effluent to less than 500 mg/L on a consistent basis and there are no significant increases in other commercial or industrial dischargers, it is possible that projected loadings will be reduced to 300 ppd. This would translate into an average loading on the trickling filter of 60 ppd/sf (assuming an average 14% reduction in the primary clarifier), and a trickling filter solids contact process would be appropriate.

2.12.3.2 Secondary Design Criteria

As discussed in Section 2.8, both the TF/SC and BF/AS processes are combined suspended growth fixed/film processes in which a suspended growth secondary treatment process follows the fixed film trickling filter to increase BOD removal. The aerated basin for the TF/SC process is smaller and the RAS is not recycled over the filter as in the biofilter process. Design criteria for both combined processes are summarized in Table 2-26.

Table 2-26
Design Criteria for Combined Processes[1]
TOS Detailed Engineering Analysis

	Unit	BF/AS[2]	TF/SC[3]
Media type		High Rate	High Rate
BOD[4] loading	lbs/1,000 cf[5]	75 – 200	20 – 75
Hydraulic loading	gpm/sf[6]	0.8 – 5.0	0.1 – 2.0
Channel MLSS[7]	mg/L[8]	1,500 – 4,000	1,500 – 3,000
Hydraulic residence time	hours	2.0 – 4.0	0.5 – 2.0
Mean cell residence time	days	2.0 – 6.0	0.5 – 1.5
Return Activated Sludge	mg/L	6,000 – 12,000	6,000 – 12,000
Food/Microorganism	F:M	0.5 – 1.2	NA[9]
Diffused air		High Rate	High Rate
O_2 for BOD removal[10]	ppd[11]	300	NA
O_2 Mixing[12]	scfm[13]	130	50

1. Source: WEF Manual of Practice 8, 1998
2. BF/AS: biofilter/activated sludge
3. TF/SC: trickling filter/solids contact
4. BOD: biochemical oxygen demand
5. lbs/1,000 cf: pounds per 1,000 cubic feet
6. gpm/sf: gallons per minute per square foot
7. MLSS: mixed liquor suspended solids
8. mg/L: milligram per liter
9. NA: not applicable
10. O_2 supplied per lb BOD removed = 0.6
11. ppd: pounds per day
12. 210-15 standard cubic feet per minute (scfm) per 1,000 cubic feet
13. scfm: standard cubic feet per minute

2.12.3.4 Disinfection System

Due to the safety risks associated with the existing chlorine gas system being used, we recommend a dry chlorine type system be installed. A calcium hypochlorite feed system is recommended.

The estimated cost of required modifications to the disinfection system is $70,000.

2.12.3.5 Solids Storage (re-use of non-functional anaerobic digester)

It is recommended that the cracked gunite coating on the outside of the digester be removed and the condition of the tank be assessed. The digester will be cleaned and inspected on the interior, and coated inside and out. An estimated cost of $75,000 has been included in the summary of WWTF upgrades presented in Table 2-27 (see next page).

2.12.3.6 Mechanical Plant Improvement Summary

Estimated costs for improvements to the mechanical plant are summarized in Table 2-39.

Table 2-27
Biofilter Activated Sludge (BF/AS) Process Estimated Costs
TOS Detailed Engineering Analysis

Item (Unit Type)	Unit(s)	Unit Cost	Quantity	Total Cost
Mobilization 12%				$227,600
Equipment				
Headworks Grinder/Ventilation	EA	$35,000	1	$35,000
Primary Clarifier Drive	EA[1]	$85,497	1	$85,497
Shallow Well Pumps	EA	$20,863	1	$20,863
Secondary Clarifier Drive	EA	$85,497	1	$85,497
Blower (7.5 hp)[2]	EA	$11,400	2	$22,799
Diffusers	LS[3]	$34,199	All	$34,199
CCB[4] Pump	LS	$11,400	1	$11,400
Onsite Chlorine Generation	LS	$70,000	100%	$70,000
Pump VFDs[5]	EA	$17,386	4	$69,544
RAS[6] Pumps	EA	$20,519	2	$41,039
WAS[7] Pumps	EA	$11,400	2	$22,799
Equipment Installation	LS	$11,400	All	$11,400
Electrical I/C				
Electrical	LS	$370,489	All	$370,489
Construction				
Railings	lf[9]	$150	100	$15,000
RAS Pump Station	sf[10]	$1,150	250	$287,500
Suspended Growth Reactor	cy	$1,391	60	$83,452
Second Floor Control Room	sf	$398,988	All	$398,988
Stairs	LS	$18,545	All	$18,545
Recirculation from Log Pond Clarifier to WWTP	LF	$100	450	$45,000

Table 2-27 Biofilter Activated Sludge (BF/AS) Process Estimated Costs TOS Detailed Engineering Analysis				
Item (Unit Type)	**Unit(s)**	**Unit Cost**	**Quantity**	**Total Cost**
Sludge Storage Tank repair (anaerobic digester)	LS	$75,000	All	$75,000
Modifications to CCB	LS	$23,181	All	$23,181
Earthwork				
Yard Piping	LS	$57,953	All	$57,953
Excavation/Grading	cy	$29	400	$11,591
Wastewater Treatment Facility Upgrade Estimated Construction Cost Subtotal				**$2,124,336**
Engineering[11] (18%)				$382,380
Contingency (20%)				$424,867
Administration (4%)				$84,973
Total Wastewater Treatment Facility Upgrade Cost				**$3,016,556**

1. EA: each
2. hp: horsepower
3. LS: lump sum
4. CCB: chlorine contact basin
5. VFDs: variable frequency drives
6. RAS: return activated sludge
7. WAS: waste activated sludge
8. cy: cubic yard
9. lf: linear foot
10. sf: square foot
11. Engineering includes design, permitting, and construction management.

2.12.3.7 Treatment Ponds

Converting the treatment ponds to wetland treatment cells is part of the preferred disposal option discussed in the previous section. The wetlands will provide enhanced BOD and nutrient removal and ensure that the wastewater discharge is not providing any nutrient enrichment or contributing to algae growth in the log-pond. The final design process will determine if all of the ponds will be required to be converted to wetland cells. The cost for converting the treatment pond to wetlands will include dewatering the existing treatment ponds and putting a soil cap over the existing solids in the basin. Estimated costs for the conversion of the treatment ponds to wetlands are summarized in Table 2-28.

Table 2-28 Conversion of Treatment Ponds to Wetlands TOS Detailed Engineering Analysis				
Item (Unit Type)	**Unit(s)**	**Unit Cost**	**Quantity**	**Total Cost**
Mobilization (12%)				$44,000
Dewatering				$8,693
Fill	cy[1]	$13.5	8,000	$108,000
Plantings	EA[2]	$0.58	109,000	$63,168
Subtotal Construction				**$223,861**
Contingency (20%)				$44,772
Engineering (18%)				$40,295
Administration (4%)				$8,954
Total Cost for Wetlands				**$317,882**

1. cy: cubic yards
2. EA: each

2.12.4 Disposal Systems

2.12.4.1 Effluent Disposal during the Summer Discharge Prohibition Period

If the log pond is drawn down several feet prior to the period of prohibited discharge to the Eel River, there should be sufficient volume in the log pond to store the seasonal discharge. A summary water balance was presented in Section 2.9, showing the estimated draw down required to provide the necessary storage. No modifications to the log pond are required to provide sufficient storage in the log pond, and converting the treatment ponds to wetland treatment cells should eliminate any potential impact that the wastewater discharge might have on the water quality of the log pond. Recommendations to mitigate the effect of other discharges to the log pond (such as, the cogeneration plant discharges) were discussed previously and include aeration, sprinklers, and shading of the bloom area. The purpose of these improvements is to mitigate the impact of the cogeneration plant discharges on the log pond water quality, so costs to implement these recommendations are not included as part of the proposed WWTF system improvements.

2.12.4.2 Biosolids Handling and Disposal

Three alternatives for biosolids disposal were evaluated in Section 2.10:

1. Creation of a Class A sludge for nurseries and landscaping
2. Dewatering and hauling to a landfill
3. Contact disposal

Contract disposal represents the lowest estimated life-cycle cost alternative for biosolids disposal from the WWTF. Initial costs for Contract disposal involves some cleaning and minor repairs to the old non-functioning anaerobic digester for continued use as a storage tank. Initial storage tank renovations are estimated at $75,000. Ultimate disposal is performed by Steve's Septic of McKinleyville, California, for sludge removal. Steve's Septic has a permitted dewatering process pad and all dewatered biosolids are sent to a licensed facility for disposal. It is recommended that this service be continued as long as contract prices are reasonable and fall within acceptable parameters.

2.13 Project Cost Summary

2.13.1 Project Cost Summary

Table 2-29 presents a project cost summary for the proposed wastewater treatment system improvements.

Table 2-29 Wastewater System Improvements Project Cost Summary TOS Detailed Engineering Analysis	
Wastewater Treatment Facility Improvements	$2,984,749
Conversion of Treatment Ponds to Wetlands	$317,882
Wastewater Treatment Total Cost	$3,302,631

Funding for the recommended improvements may not be available to perform all of the work through a single project. Therefore, all of the improvements represented by the above cost summary have been separated into identifiable individual projects which can be performed as funding becomes available. Table 2-30 presents the proposed individual projects along with a recommended priority for phasing. The priorities are based upon the following criteria:

Priority 1 - Improvements that need *immediate* attention to mitigate current critical operations or maintenance issues:

a. These projects or improvements *either* must be:
   i. addressed based on existing regulatory compliance mandates (that is, which have been identified by the regulatory agencies); *or*
   ii. they are required to reduce the risk of regulatory non-compliance based on analysis of equipment conditions and/or extraordinary maintenance costs currently known to TOS.
b. The time horizon for Priority 1 varies between immediate and 2-4 years. This kind of time frame could allow TOS or a future utility operator time to design and permit the specific improvement as well as other more extensive infrastructure improvements noted in Priority 2 below.

Priority 2 – Necessary infrastructure improvements with a projected 20+ year life that may also be critical to future utility system operations:

a. to meet current industry standards for similar facilities and
b. for existing or projected regulatory requirements.

Priority 3 – Desirable equipment or operational improvements that may not be required under current regulations, but which provide better facility controls, equipment automation and monitoring/reporting equipment for better management and communications with maintenance personnel. These may be recommended, but are subject to an acquirer's discretion.

Table 2-30
Wastewater System Improvements Project Prioritization
TOS Detailed Engineering Analysis

Project	Estimated Project Cost	Recommended Project Priority
Headworks Grinder/Ventilation	$56,936	1
Clarifier Drive Replacements	$278,166	2
Log Pond Clarifier recirculation to WWTP	$71,568	1
Well Pump Replacements and VFDs	$147,070	1
Suspended Growth Reactor	$869,481	2
Chlorine Contact Improvements	$169,284	2
Sludge Storage Tank Improvements	$135,870	3
Second Floor Control Room	$1,288,181	2
Total WWTP Improvements	**$3,016,556**	
Pond to Wetland Improvements	$317,882	3

Note: The individual projects and costs listed above were derived from groupings of line item costs presented in tables 2-13/27 that are combined with associated engineering, contingency, administration, and mobilization costs. Earthwork (yard piping & excavation/grading) and equipment installation line items were also distributed to specific projects. Table 2-31 presents the project cost breakdown.

All process analyses and projected projects area based upon estimating wastewater characteristics (flows and contaminant loading) considering future flow reduction anticipated from installation of a new and relatively "water-tight" collection system and service laterals, along with establishing limits for contaminant discharge with the Brewery. The need and extent of development related to some of the projects listed above and project prioritization will be more clearly defined as actual flow and loading data are realized over the next two to five years.

Table 2-31 Wastewater System Improvements Project Cost Breakdown TOS Detailed Engineering Analysis						
Specific Project Related Line Item	**Construction Cost**	**Equipment Installation Costs Including Associated Mobilization**	**Earthwork Costs Including Associated Mobilization**	**Engineering, Contingency, Administration Fees, and Construction Costs Associated Mobilization**	**Total Line Item Cost**	**Total Project Cost**
Headworks Grinder/Ventilation	$35,000	$1,273		$20,664	$56,936	**$56,936**
Primary Clarifier Drive	$85,497	$3,109		$50,477	$139,083	**$278,166**
Secondary Clarifier Drive	$85,497	$3,109		$50,477	$139,083	
Recirculation from Log Pond Clarifier to WWTP	$45,000			$26,568	$71,568	**$71,568**
Shallow Well Pumps	$20,863	$759		$12,317	$33,939	**$147,070**
Pump VFDs	$69,544	$2,529		$41,058	$113,131	
RAS Pumps	$41,039	$1,492		$24,229	$66,760	**$869,481**
WAS Pumps	$22,799	$829		$13,461	$37,089	
Blower (7.5 hp)	$22,799	$829		$13,461	$37,089	
Diffusers	$34,199	$1,243		$20,191	$55,633	
RAS Pump Station	$287,500			$169,738	$457,238	
Suspended Growth Reactor	$83,452		$82,951	$49,269	$215,672	
CCB Pump	$11,400	$414		$6,730	$18,544	**$169,284**
Onsite Chlorine Generation	$70,000	$2,545		$41,328	$113,873	
Modifications to CCB	$23,181			$13,686	$36,867	
Sludge Storage Tank repair (old anaerobic digester)	$75,000		$16,590	$44,280	$135,870	**$135,870**
Electrical	$370,489			$218,734	$589,223	**1,288,181**
Railings	$15,000			$8,856	$23,856	
Second Floor Control Room	$398,988		$11,060	$235,560	$645,608	
Stairs	$18,545			$10,949	$29,494	

Table 2-32
Wastewater Treatment Plant Structures and Equipment Inventory

Item Description	Unit Operation	Size	Units	Year Installed	Material of Const.
Treatment Headworks 18, 20, 21			1	1954	
Grinder-Muffin Monster 13				1996	
Primary Clarifier (1997) 1, 9, 14, 16 ,17	30-foot di	42,000 gal	1	1954	Concrete
Trickling Filter (2004) 2	44.5-foot di	70,000 gal	1	1954	Redwood Slat
Secondary Clarifier 10	30-foot di	42,000 gal	1	1954	Concrete
Sludge Digester (2004) Cleaned Out 3, 12	19.5-foot di	31,300 gal	1	1954	Concrete
Chlorine Contact Basin 4, 23		500 GPM	1	1954	Concrete
Treatment Ponds 5			3	1960's	Dirt
Log Pond Clarifier (2000) 6				1970's	
Shallow Well Pumps 8, 11, 15, 24	4-inch	500 GPM	2	1991/2015	
Deep Well Pumps 20 hp 22	4-inch	650 GPM	2	2004/2005	
Chlorine Contact Chamber 15 hp Pump	8-inch	800 GPM	1	2006	
Chlorine Contact Chamber 10 hp Pump	4-inch	350 GPM	1	1995	
Treatment Pond Shack Pump 40 hp 19	6-inch	500 GPM	1	2004	
Chlorine Gas Injector 7, 25			1	2005	
Chlorine Cylinders		1 Ton	2	replaced yearly	
Chlorine Storage Building (W/ One 1 Ton Cylinder)			1		
Chlorine Sensor Alarm Panel				2003	
Control Room (Ground Floor)				2003	
Liquid Chlorine Line				2003	Black Poly
Treatment Pond Aerator		7.5 hp	2	2009	

1. Rebuilt Arm
2. New Dist Arm
3. New Floating Cover & Cleaned Out
4. Repaired Baffles (2004)
5. Clearing & Grubbing
6. Replaced Arms & Gear Box
7. Replaced Each Year
8. Older Pump Rebuilt In 1996
9. Gear Box & Gears Replaced 4/2012
* Other Maintenance & Rebuild Information
10. Dec 09 New V Notch Weir Secondary Clarifier & Rebuilt 10" Center Pipe For Inlet Water
11. 04-10-10 North Shallow Well Pump Rebuilt (Bearing Worn Out).
12. 04-25-10 Circulating Pump For Digestor Seal Blown Out Repaired 5-10-10
13. 05-10-10 Muffin Monster Grinder Pulled, Replacement Reinstalled 8-5-10
14. 01-05-11 Primary Clarifier Sweep Arm Broke, Fixed & Restarted 1-10-11
15. 04-20-11 South Shallow Well Pump Rebuilt & New Shutoff Valve Installed
16. 01-27-12 Primary Clarifier Down, Broken Gear Box & New Ball Gear
17. 02-10-12 Primary Clarifier Drained, New V Notch Weir Installed
02-11-12 Restarted Primary Clarifier All Repairs Complete
18. 09-11-12 Installed New 6-Inch Vaughn Emergency By-Pass Pump
19. 12-31-12 To 01-22-13 M012B Pump Down For Repair

Table 2-32 Wastewater Treatment Plant Structures and Equipment Inventory					
Item Description	**Unit Operation**	**Size**	**Units**	**Year Installed**	**Material of Const.**
20. 06-30-13 To 07-10-13 Repair Flow Meter					
21. 11-16-14 Replace Water Pump Backup Generator Sewer Plant					
22. 12-03-14 Replaced Shut Off & Check Valves 2 Each Deep Well					
23. 2014 Drained & Vactored Out Chlorine Contact Chambers & Put In 3 New Baffles					
24. 10-15-15 One New Vaughn Shallow Well Pump					
25. July 2015 New Chlorine Ejector At Sewer Plant					

5 Water Treatment

5.0 Water Treatment

5.1 Introduction

The Scotia Water Treatment Facility (WTF), constructed in 1966, has historically supplied the domestic water system with high-quality water. The facility is located off a gravel access road on the hillside east of U.S. Highway 101 (Figure 5-1). This chapter describes the WTF's general condition, operation, and performance, and presents recommendations regarding required improvements.

This section also includes an analysis of water demands and capacity. The WTF supplies current domestic usage and commercial and industrial demands for treated water, while operating at less than 100% of its capacity. Based on an analysis of the theoretical capacity of the individual treatment system components, the treatment system is currently operating at approximately 25% of the design capacity.

5.2 Description of Existing Systems

The treatment system is well maintained, but the age of many system components exceeds their design life, and the overall condition is deteriorating. The two in-line multimedia pressure filters operate on the hydraulic head provided by the 1.0-million gallons (MG) raw water tank (Figure 5-1). Pretreatment of the raw water consists of adding an anionic polymer prior to the raw water storage tank. The pretreatment system serves to reduce high raw water turbidities to treatable levels. Under normal conditions, treated water is consistently of a high quality. Recently, it appears that a quantity of water with an elevated amount of carbon reached the raw water intake, compromising the finished water quality, and prompting cleaning and inspection of many system components.

The water treatment system consists of the following processes:

- Coagulation—coagulant addition and rapid mix
- Sedimentation—raw water storage tank
- Filtration—multimedia pressure filters
- Disinfection—gas chlorination

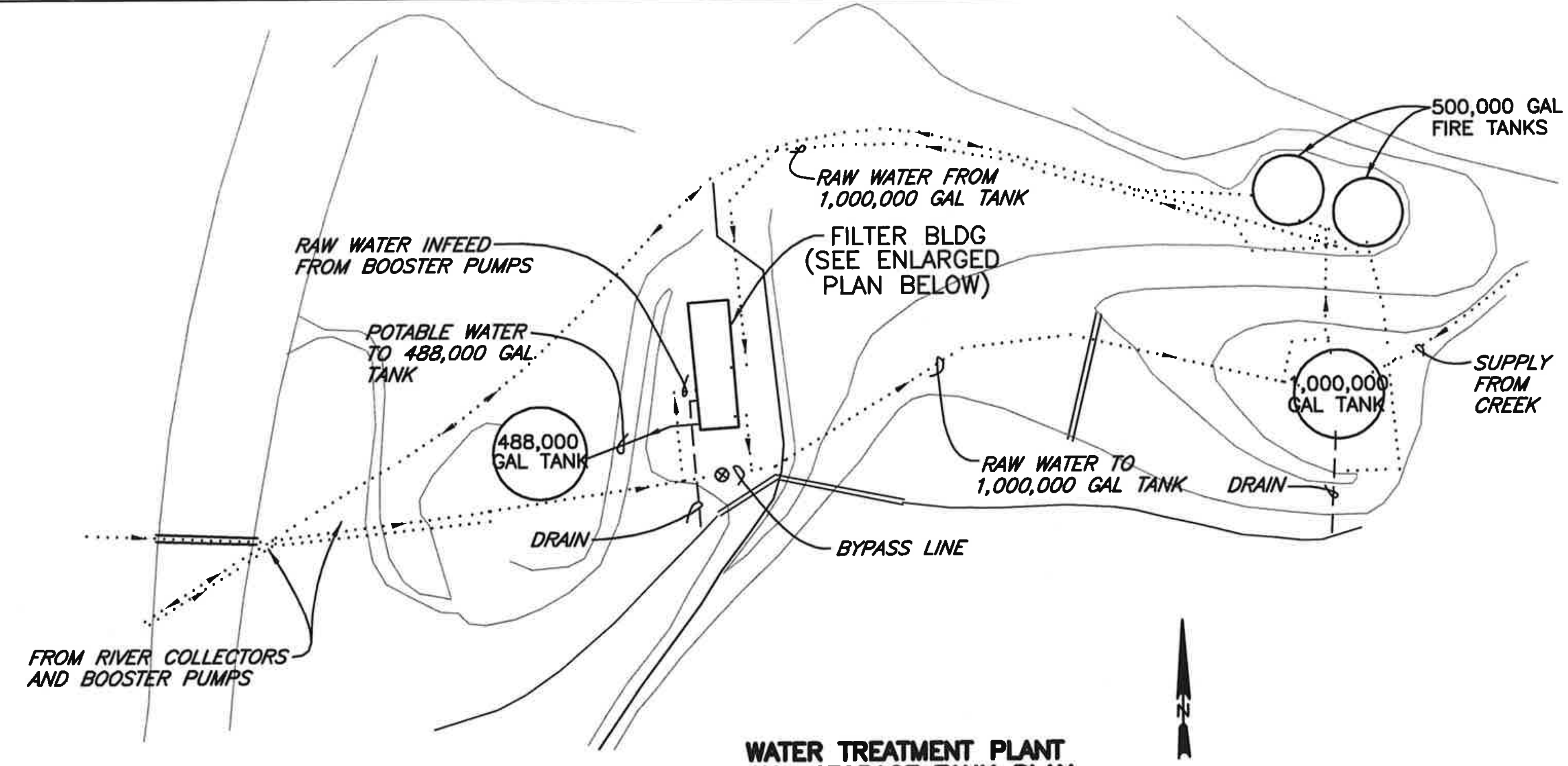
Water from the intake gallery in the Eel River is pumped to a 1.0-MG raw water storage tank by domestic booster pumps. Before discharging to the tank, the water is piped through the WTF where a flocculant is added prior to an in-line mixer. The water flows through the mixer and up to the 1.0-MG raw water tank.

The 1.0-MG raw water tank, which also serves as a sedimentation tank, feeds a pressure filter system at the WTF. Filtered water is disinfected and then flows to the 0.488-MG finish water storage tank. The treatment system does not require any internal pumps, operating on pressure supplied by the upper 1.0-MG tank.

Figure 5-1 schematically illustrates the WTF and filter building. Equipment is summarized in Table 5-1. The facility is well maintained, but in declining condition.

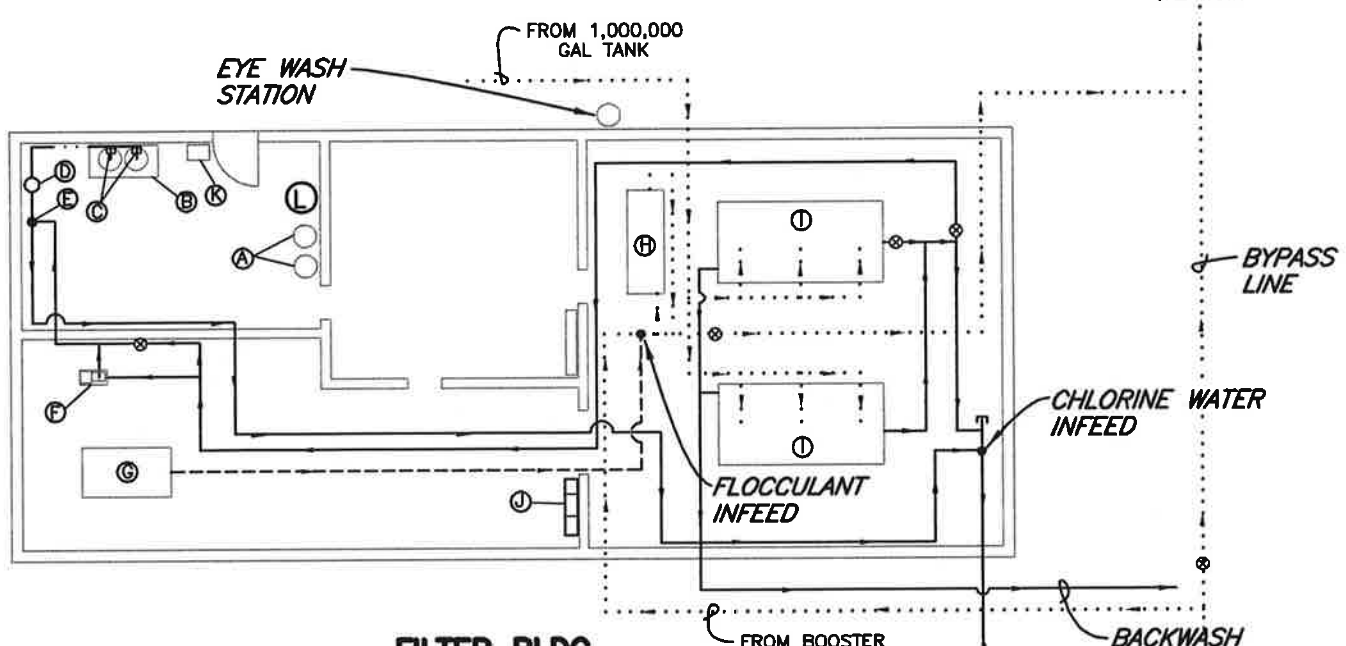
FILTER BUILDING EQUIPMENT

ITEM	DESCRIPTION
A	(2) 150 LB CHLORINE CYLINDERS (STORAGE)
B	(2) CHLORINE CYLINDERS ON SCALE WITH REGULATORS
C	CHLORINE FLOW METER
D	CHLORINE ROTAMETER
E	CHLORINE INJECTOR
F	CHLORINE PUMP
G	FLOCCULANT FEED TANK AND PUMP
H	MIXING TANK
I	SAND FILTERS WITH BACKWASH SYSTEM
J	CHLORINE PUMP CONTROLS
K	EXHAUST FAN WITH LOW INTAKE AND HIGH EXHAUST
L	CHLORINE DETECTOR



WATER TREATMENT PLANT AND STORAGE TANK PLAN
1"=100'±

NO SCALE



FILTER BLDG ENLARGED PLAN
NO SCALE

LEGEND:

— · — · —	CHLORINE LINE
- - - - - -	CHEMICAL LINE
· · · · · · ·	RAW WATER LINE
————	POTABLE WATER LINE

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SHN Consulting Engineers & Geologists, Inc.

Scotia CSD Formation
Detailed Engineering Analysis
Scotia, California

Water Treatment Facility

005161.903

May 2009 | 005161-903-099007-AF-2003-SVY_5-1 | Figure 5-1

Table 5-1 Water Treatment Facility Equipment Assessment TOS Detailed Engineering Analysis				
Item	**Description**	**Size**	**Units**	**Installation Date**
Mixing Tank	Steel in-line baffled	1,100 gallons	1	1968
Sand Filters[1]	8-foot diameter x 30-foot long	240 square feet	2	1966
Filter Media	Sand, deactivated anthracite	NA[2]	NA	1993
Backwash Control	Head loss differential, flow meter	NA	1	1999
Turbidimeter	Hach	NA	2	1992
Flow meter	Velocity, Sparling Series 100	NA	1	2004
Flow recorder	Chart recorder Honeywell	NA	1	1966
Chlorine Detector	Wallace & Tiernan	NA	1	1996
Chlorinator	ST NXT	NA	2	2015
Chlorine Scale	Two 150-pound cylinders	NA	NA	1996
Flocculent Feed Tank[3]	NA	200-gallon	1	1966
Flocculent feed pump	---[4]	---	1	2005
Fluoride Pumps[5]	---	---	2	2002

1. Baffles and media replaced 1993
2. NA: not applicable
3. Offline
4. ---: no data
5. Not in use

5.2.1 Pre-treatment and Sedimentation Tank

The untreated or raw water is pumped to the WTF by the domestic water booster pumps. At the WTF, an anionic polymer is injected to enhance settlement. The polymer is injected directly into the pipe immediately preceding an in-line mixing tank. The mixing tank is a 1,100-gallon horizontal steel tank with internal baffles. The mixer is painted steel and appears to be in good condition.

The pre-treatment system consists of polymer addition, the mixing tank, and the large storage tank. There is no flocculation tank provided. The baffled mixing tank appears well designed for the current flow conditions. A detention time of approximately one minute is provided with one domestic water pump running and is within typical ranges for in-line mixers (30 to 60 seconds).

In the winter months, raw water turbidities from the Eel River intake can exceed 100 nephelometric turbidity units (NTU) and the polymer and large sedimentation tank are necessary to reduce turbidities prior to filtration. The average raw water turbidity from the 1.0-MG tank was 2.32 NTU in 2014, but there is a large range in raw water quality. During 2014, the minimum raw water turbidity was 0.096 NTU and the maximum was 83.99 NTU.

5.2.2 Filtration System

Water from the 1.0-MG raw water tank is filtered in two horizontal cylindrical pressure filters constructed by California Filter Co. in September 1966. Each filter is 30 feet long and 8 feet in diameter, with a surface area of approximately 240 square feet. The filters are constructed of steel with coatings on the interior and exterior to prevent corrosion. Piping is painted ductile iron with a polyethylene coating. The valves that control filter operation are well maintained and have been rebuilt as the operators determine the need from inspections.

5.2.2.1 Filter Operation

The filters operate on line pressure supplied by the 1.0-MG tank. Each filter is divided into four cells with individual inlets and all cells discharge into a common underdrain system. Each filter was originally designed to treat up to 700 gallons per minute (gpm). However, the filters are currently operated to treat approximately 350 gpm through each filter. The inlet to each compartment is located at the top of the tank and each feed line has a pneumatically actuated, hydraulically operated control valve. During backwash, the main backwash valve is open and each cell of the filter is backwashed individually from the common under-drain by closing the influent and opening the waste valve for each respective cell.

The filters are backwashed twice per week during summer months and require operator initiation of the backwash sequence. During winter months, the backwash frequency increases; and during periods of high turbidity, the filters may be backwashed daily. When the operator initiates the backwash, the raw water inlet valve of cell 1 is closed and the backwash valve opens. This allows the raw water to pass through the three active cells and discharge the filtered unchlorinated water through the cell to be backwashed. When the preprogrammed timer runs down, the backwash valve closes and the filter inlet valve opens. The same sequence then cycles through cell 2, cell 3, and cell 4. At the end of the backwash for cell 4, the backwash control panel opens the filter drain valve for the filter-to-waste cycle.

Currently, each cell is backwashed for a preset time of 12 to 15 minutes. After all cells in each filter are backwashed, a 10-minute filter-to-waste cycle is initiated. It is our understanding that the current backwash rate is approximately 425 gpm, but this flow rate cannot be verified with existing instrumentation. The flow rate for filter-to-waste is unknown and not measureable. Using best estimates, the total flow per backwash cycle is approximately 60,000 gallons. This estimate includes flow to backwash all filter cells and the filter to waste cycle.

The main discharge from the backwashing sequence enters a storm drain system that discharges to the Eel River adjacent to Fireman's Park. The flow from the filter-to-waste is diverted to a drainage south of the WTF, which empties into the Eel River south of the river intake.

5.2.2.2 Filter Performance

The water treatment system consistently produces high-quality water. Filter effluent turbidity (which is recorded daily) indicates that average finished water turbidities in 2014 were less than 0.09 NTU. During this period, the maximum daily turbidity recorded was 0.351 NTU and consistently low finished-water turbidities were maintained even when raw-water turbidity increased significantly.

Treatment system performance is monitored by Hach turbidimeters at the WTF, which provide continuous readings of raw water turbidity and filtered water turbidity. The turbidimeters do not record on a continuous basis. Instantaneous values are recorded by operations staff on the daily filtration report.

5.2.2.3 Filter Condition

A significant volume of filter media was found during a recent cleaning of the finish water tank at the WTF prompting inspection of both pressure filters.

Pressure Filter No. 1 was inspected on August 18, 2015. The most recent backwash prior to this inspection was conducted on Friday, August 14, 2015. Visual inspection of the media in Filter No. 1 revealed multiple issues that may affect the performance of the filters, including mud ball formation, cracks in the media, leaks in the bulkhead, and short circuiting.

Following the inspection of Filter No. 1, samples of backwash water were collected from both filters on September 25, 2015. Turbidity readings were collected during the first flush of water from each cell, then at approximately 5-minute intervals. Backwash water is expected to be turbid, and maximum turbidity values from the cells in Filter No. 1 ranged from 71.7 to 133 NTU. However, maximum turbidity measured from Filter No. 2 was 31.9 NTU, and the maximum turbidity values from cells 3 and 4 were less than 5 NTU. This indicates that either the filters are not filtering solids from the raw water, or the backwash process is not removing collected solids.

A visual inspection of the media in Filter No. 2 was performed on October 1, 2015. The most recent backwash prior to the inspection was on Monday, September 28, 2015. The media condition in Filter No. 2 was found to be similar to that of Filter No. 1, with significant cracking, short circuiting, and corrosion of the bulkheads. Additionally, the majority of the filter media appeared to be large mud balls with very little surface area available for filtration.

It was also noted that the media in both filters was mostly sand, with very little anthracite remaining.

A wall thickness sampling of the tanks was conducted to determine if the wall thicknesses of the filters were suitable for rehabilitation. During the test, the minimum wall thickness measured was 0.11 inches, which is less than 50% of the original thickness, reducing the vessel's pressure rating.

5.2.3 Disinfection System

Filtered water is disinfected with chlorine fed from two, 150-pound cylinders. The chlorination system consists of a scale, a chlorinator with a vacuum regulator and automatic switch-over system, and an ejector system to inject chlorine gas into the solution line. Chlorine solution is injected in the filter effluent line in the filter building and disinfected treated water is then stored in the 0.488-MG finish water storage tank.

Chlorine is applied to the filtered water at an average dosage of approximately 1.04 milligrams per liter (mg/L). The finish water storage tank provides more than adequate detention time for disinfection.

The system feed rates and dosages are monitored on a daily basis to ensure that the chlorine residual is maintained throughout the system and to comply with California Department of Health Services (DHS) requirements. A chlorine residual is obtained from a service in the distribution system on a daily basis. Based on the daily field logs, the residuals average 0.4 mg/L in 2014.

5.3 Regulatory Criteria

5.3.1 Water Rights

The State Water Resources Control Board (SWRCB) Department of Water Resources (DWR) oversees license number 6373 and permit number 3027, which were issued to PALCO on July 7, 1961, and transferred to TOS in 2008 as part of the bankruptcy procedures. Water is permitted to be diverted for domestic and industrial uses, at a specified diversion location.

Diversion of water (up to 4,588,500 gallons per day [gpd]) is allowed by the permit, with no expressed annual quantity limit. Priority rights were established from June 1, 1927, and the proof of diversion was accepted by the DWR in January 15, 1959.

5.3.2 Public Water System Regulations

Drinking water regulations were established in 1974 with the signing of the Safe Drinking Water Act (SDWA).

The RWQCB Division of Drinking Water (DDW) is designated by the EPA as the primary agency to administer and enforce the requirements of the federal SDWA, including the SDWA Amendments of 1996 or the Surface Water Treatment Rule (SWTR). The statutes and regulations adopted by the State of California and the DDW to implement SDWA requirements are contained in Title 22 CCR (California Code of Regulations; related to drinking water).

5.3.3 Maximum Contaminant Levels

One of the main elements of the drinking water regulations was the establishment of maximum contaminant levels (MCLs) for inorganic, organic, microbiological, and radionuclide contaminants and turbidity. An MCL is the maximum allowable level of a contaminant in water delivered to the users of a public water system. Concentrations above the MCL for a contaminant are considered violations.

The TOS water system is in compliance with all federal and state regulations and as a condition of its operating permit, prepares a consumer confidence report that includes the levels of any detected contaminants subject to an MCL, unregulated chemicals for which monitoring is required as defined by Title 22 Code of Federal Regulations (CFR) Chapter 17, Article 2, Section 65550, disinfection byproducts or microbial contaminants for which monitoring is required by 40 CFR, and sodium and hardness.

The water system is required to monitor for total coliform twice a month. During 2014, all samples collected tested “absent” for the presence of coliform bacteria.

5.3.4 Surface Water Treatment Rule

The Surface Water Treatment Rule (SWTR) established that surface water must be treated using filtration and disinfection. Title 22 CFR Chapter 17, Article 2, Section 64652 (a) defines the treatment requirements as follows:

> *Each supplier using an approved surface water shall provide multi-barrier treatment that meets the requirements of this chapter and reliably ensures at least:*
>
> *(1) a total of 99.9% reduction of Giardia cysts through filtration and disinfection;*
>
> (2) *a total of 99.99% reduction or viruses through filtration and disinfection.*

5.3.5 Performance Standards

Performance standards for turbidity are defined by Title 22 CFR Chapter 17, Division 4, Article 2, Section 64653 (c)(2), which states that a supplier using conventional or direct filtration treatment that serves fewer than 10,000 persons, the turbidity shall be less than or equal to 0.3 NTU in at least 95 percent of the measurements taken each month, and if monitored using grab samples no sample shall exceed 1 NTU.

Performance standards for disinfection are defined by Title 22 CFR Chapter 17, Division 4, Article 2, Section 64654 (b):

> *Disinfection treatment shall comply with the following performance standards:*
>
> *(1) Water delivered to the distribution system shall not contain a disinfectant residual of less than 0.2 mg/L for more than four hours in any 24 hour period.*
>
> *(2) The residual disinfectant concentrations of samples collected from the distribution system shall be detectable in at least 95% of the samples taken each month that the system serves water to the public. At any sample point in the distribution system, the presence of heterotrophic plate count (HPC) at concentrations less than or equal to 500 colony forming units per milliliter shall be considered equivalent to a detectable disinfectant residual.*

The TOS Scotia water system complies with all required performance standards. Performance of the treatment system is discussed in detail in Section 5.4

5.3.6 Division of Drinking Water Required Documents

On August 24, 2014, the Division of Drinking Water (DDW) supplied information regarding plan and report deficiencies for the Scotia WTF.

The following documents are required, and either do not exist or are outdated:

- Operations manual
- Emergency disinfection plan
- Disinfection monitoring plan
- Filter inspection plan
- Filter inspection annual report
- Cross-connection and backflow prevention program

Additionally, DDW listed the following equipment deficiencies:

- Continuous recording turbidimeter
- Continuous chlorine analyzer
- Finished water chlorine and turbidity alarms

5.3.7 Monitoring

Monitoring requirements for turbidity are defined in CCR, Title 22, Chapter 17, Article 3, Section 64655. The water supplier is required to monitor the turbidity level of the raw water supply by taking and analyzing daily grab samples. To determine compliance with the performance standards for filtered water turbidity, the water system operator is required to obtain samples of the combined filter effluent, prior to clearwell storage, at least once every four hours that the system is in operation and to monitor the turbidity measurements on a continuous basis, recording results every 15 minutes.

At the WTF, the turbidity of the raw water is measured on a continuous basis by two turbidimeters. However, the turbidimeters do not record the data on a continuous basis, so the operators must take grab samples as required to be in compliance.

Each water supplier is required to develop and conduct a monitoring program to measure the parameters that affect the performance of the disinfection process. The requirements for this monitoring program are defined in CCR, Title 22, Chapter 17, Article 3, Section 64656**.** Suppliers serving 500 to 1,000 people may collect and analyze grab samples of disinfectant residual twice each day, provided that any time the residual disinfectant falls below 0.2 mg/L, the supplier shall take a grab sample every four hours until the residual concentration is equal to or greater than 0.2 mg/L. According to the operations supervisor, an approved daily monitoring program is in place and the chlorine residual is monitored on a daily basis at various points in the distribution system.

5.4 Demand and Capacity

5.4.1 Water Demand/Usage

Average daily treated water production for 2014, based on daily treatment plant logs, was 312,067 gpd as summarized in Table 5-2. Additional water demand/usage information can be found in "Chapter 4: Water Distribution," Section 4.3.

Table 5-2 Domestic Water Production TOS Detailed Engineering Analysis			
Date	**Total Usage**		**Maximum Daily Usage (gpd)**
	(gallons per month)	**Average (gpd)[1]**	
January 2014	9,046,000	291,806	506,000
February 2014	7,949,000	283,893	385,000
March 2014	8,793,000	283,645	403,000
April 2014	9,362,000	312,067	464,000
May 2014	12,604,000	406,581	845,000
June 2014	11,287,000	376,233	458,000
July 2014	11,248,000	362,839	483,000
August 2014	10,993,000	354,613	453,000
September 2014	10,161,000	338,700	471,000
October 2014	10,068,000	324,774	324,774
November 2014	7,958,000	265,267	265,267
December 2014	8,835,000	285,000	285,000
Average	9,362,000	312,067	464,000
Maximum	12,604,000	406,581	845,000
1. gpd: gallons per day			

5.4.2 Capacity

Sedimentation Capacity. Design criteria published by the EPA (*EPA Handbook: Optimizing Water Treatment Plant Performance*, 1998 Edition) for sedimentation tanks states that the maximum recommended surface overflow rate (SOR) for a sedimentation basin greater than 14 feet in depth is 0.7 gallons per minute per square foot (gpm/SF). The 1.0-MG storage tank has a diameter of 70 feet and an area of 3,847 SF. Based on the recommended SOR, the tank has a maximum capacity of 2,693 gpm. This would provide 6 hours of detention time. Currently, the peak instantaneous flow to the reservoir is equal to 1,200 gpm, the capacity of a single domestic water booster pump.

Filter Capacity. The filters run 4 to 6 hours per day and process an average of approximately 300,000 gpd of treated water. The surface loading rate under current conditions is approximately 1.8 gpm/SF. Article 5 of the Title 22 CCR relating to drinking water stipulates that for pressure filters, filtration rates shall not exceed 3 gpm/SF for dual media filters. Estimated filter capacities and current and maximum loading rates are summarized in Table 5-3.

Table 5-3 Capacity of Filtration System TOS Detailed Engineering Analysis		
Online Hours	Current Loading at 2 gpm/SF[1] (gpd)[2]	Capacity at 3 gpm/SF (gpd)
8[3]	414,720	622,080
12[3]	622,080	933,120
24[4]	967,680	1,451,520

1. gpd/SF: gallons per day per square foot
2. gpd: gallons per day
3. Assumes backwash for 10% of hours online
4. Capacity based on run time of 70%

CT Capacity. The EPA has published guidelines for determining the CT value (chlorine concentration over time) required to achieve required levels of disinfection. The CT value is equal to the chlorine concentration in mg/L (C) times the actual time (T) that water is in contact with the disinfectant. The limiting CT value is taken as the value that achieves the required reduction (in base-10 logarithm orders, or log) assuming minimum temperature and maximum pH.

Disinfection is the final barrier in the WTF and is responsible for removing any microbial pathogens that pass through previous processes. The SWTR requires that the treatment system (including disinfection) provides a minimum of 99.9%, 3-log removal and/or removal of *Giardia lamblia* cysts and at least 99.99%, 4-log removal and/or removal of viruses. Because the expected log reduction capacity of a conventional filtration system is 2.5 log removal for *Giardia* cysts and 2.0 log removal for viruses, the disinfection system would only be required to provide the remaining 0.5 log and 2.0 log reductions to comply with the federal SDWR (EPA Handbook 1998 Edition). However, it is considered good practice to require that the disinfection system provides at least 1.0 log removal for *Giardia lamblia* cysts, and that value has been used to determine CT value required for disinfection at the Scotia WTF.

Based on an average residual of 0.3 mg/L, a pH of 7.5, and a temperature of 15 degrees Centigrade, the required CT value for a 1-log reduction of *Giardia* cysts is 28 CT units and the required CT value for a 2-log removal of viruses is 2.0 CT units. The requirement for *Giardia* is limiting. Based on a CT of 28 and an average residual of 0.3 mg/L, the required detention time is 93 minutes.

Available contact time is calculated based on the effective volume in the finish water storage tank and in the distribution lines up to the first service. To determine the effective volume, it is necessary to apply a reduction factor that accounts for the effects of short-circuiting in the unbaffled tank. In this analysis, a factor of 0.3 was used (based on published EPA guidelines [EPA, 1989]). The 0.488-MG domestic water tank has an effective volume of 146,000 gallons and at current average feed rates, provides a detention time well in excess of the 93 minutes required.
The capacity of the finish water tank to provide adequate contact time for disinfection at future flow rates was calculated to be 1,569 gpm (146,000 gallons/93 minutes).

Excess Capacity. The treatment system is not currently running at 100% of its capacity. The capacity of the treatment system is estimated to be 1.45 MGD based on the capacity of the filtration system (Table 5-4). Based on the average daily water production (Table 5-2), the system is operating at approximately 25% capacity.

Table 5-4 Capacity of Water Treatment Facility TOS Detailed Engineering Analysis					
Treatment Systems[1]	**Limiting Criteria**		**Theoretical Capacity**		
			gpm[2]	**cfs[3]**	**MGD[4]**
Sedimentation tank	0.7 gpm/SF[5]	6-8 hours	2,693	6.0	3.8
Filtration	3 gpm/SF	---	1,440	3.2	1.45[1]
Disinfection[6]	93 minutes Detention		1,569	3.49	2.26

1. Assumes 24-hour run time with 30% allowance for backwash and downtime
2. gpm: gallons per minute
3. cfs: cubic feet per second
4. MGD: million gallons per day
5. SF: square foot
6. Based on volume of domestic storage tank times 0.3, does not include distribution system volume

5.5 Improvements

The Scotia WTF was constructed in 1966 and has been well maintained. The WTF is currently in compliance with current state and federal regulations and consistently provides high-quality drinking water. However, due to the condition of the pressure filters, it is uncertain if the plant will be able to maintain compliance during winter storms when influent turbidity rises.

An updated summary of deficiencies and performance limiting factors is provided below. The recommended capital improvements associated with these issues of concern have been categorized as follows:

- Priority One: improvements considered immediately necessary to maintain compliance with applicable regulations and consumer confidence
- Priority Two: improvements necessary for the long-term viability of the plant, but not an immediate issue with regard to water quality or compliance
- Operational Improvements: improvements that are recommended for operational reliability during the 20-year planning period

These capital improvements and associated costs are described in Tables 5-5, 5-6, and 5-7 in Sections 5.5.1, 5.5.2, and 5.5.3, respectively.

5.5.1 Priority One Improvements

Required capital improvements identified as a Priority 1 include rehabilitation of pressure Filter No. 2, installation of a backwash recovery system, replacement of the finish water distribution line from the clear well to the distribution system, new turbidimeters, chlorine residual analyzer, and a remote alarm system.

5.5.1.1 Rehabilitation of Filter No. 2

Although the treatment plant is producing quality water, the condition of both filters indicates that adequate filtration may not be attainable during the winter months when raw water turbidity increases. Observations made during the sampling of the backwash water indicate that it is unlikely that Filter No. 2 is sufficiently reducing turbidity. Although the condition of the two filters is similar, it appears that Filter No. 1 is reducing turbidity, and can continue to function in the short term.

As an interim solution to full rehabilitation or replacement of both filters, a partial rehabilitation of Filter No. 2 is proposed to provide adequate effluent quality in the immediate future.

5.5.1.2 Turbidimeters

The existing turbidimeters on the raw water and finished water monitor do not record turbidity. Installing turbidimeters that have continuous monitoring capability is considered a priority for operation and compliance. Installation of continuous recording turbidimeters was also requested by DDW.

5.5.1.3 Alarm System

According to the operator, there is no alarm for a system malfunction or equipment failure at the treatment facility. The chlorine detector provides a local alarm to notify system operators that chlorine-gas has been detected and that self-contained breathing apparatus must be employed before entering the area. Because this alarm is not transmitted to on-call personnel, the problem cannot be addressed immediately.

Equipment failures that potentially effect water treatment or personnel safety must be monitored. Examples of equipment alarms that would provide warning of water system malfunction include valve failure, failure of the polymer pump, chlorine system malfunction (for example, loss of vacuum), chlorine gas detention, and low reservoir level. A remote alarm system is proposed as a Priority 1 improvement. An inexpensive auto-dialer system can be used to warn water system personnel of WTF emergencies that require immediate response.

5.5.1.4 Repair of Water Line to Distribution System

Recently, the water line from the clear well to the distribution system was found to be damaged. As an emergency repair, a 6-inch water line was installed overland from the finished water tank drain line through a Caltrans culvert and connected to the distribution system on the west side of U.S. Highway 101.

A water line is proposed to be installed through the original containment pipes from the finished water tank outlet to the original point of connection.

5.5.1.5 Backwash Recovery System

The current discharge of the backwash water enters a storm drain system that ultimately discharges to the Eel River. This is an unpermitted discharge, and the discharge occurs throughout the year, including during the summer prohibition period. A backwash recovery system is recommended to bring the plant into compliance with the Clean Water Act and other relevant regulations.

There are two potential options for disposing of the spent backwash water. If regulatory approval can be obtained, it is proposed that the discharge be routed to the log pond for disposal. This option is relatively inexpensive, and the cost estimate is included in Table 5-5. If discharge to the log pond cannot be permitted, the installation of a 100,000-gallon backwash tank that will allow settling of solids is proposed. Additional infrastructure for this option includes a pump and piping to recycle clarified backwash water to the raw water tank, and drying beds for solids handling.

5.5.1.6 Flocculation Tank

The current flocculation system consists of polymer injection and a 1,100-gallon mixing tank. After mixing, the water is pumped to the raw water tank for the settling of solids. However, as part of the repair of the distribution system, the fire suppression system will be connected to the raw water tank, which necessitates separation of fire suppression water in the raw water tank from the flocculent.

A 15,000-gallon flocculent mixing tank is proposed. Water will be piped from the raw water tank to the proposed mixing tank, then to the pressure filters. The tank will provide adequate contact time, and its position in the system will stop the flocculent addition to the raw water tank.

5.5.1.7 Chlorine Analyzer with Alarm

Chlorine residuals are measured at the Scotia Fire Station. To meet regulations, these measurements should be taken at a point in the system before the first customer connection. It is recommended that a continuous recording chlorine analyzer with an alarm be installed in an appropriate location. A residual chlorine analyzer with alarm has been requested by DDW.

Table 5-5
Estimated Costs, Water Treatment and Storage Priority One Upgrade (Rev. 10/7/15)
TOS Detailed Engineering Analysis

Item (Unit Type)	Unit(s)	Quantity	Unit Cost	Total Cost
Mobilization/Demobilization[1]	LS[2]	1	$50,000	$50,000
New Turbidimeters[1]	LS	1	$12,000	$12,000
Remote Alarm System[1]	LS	1	$12,000	$12,000
Improvements to Chlorination System[1]	LS	1	$25,000	$25,000
Backwash Recovery System[3]	LS	1	$35,000	$35,000
Flocculation Tank	LS	1	$200,000	$200,000
Pressure Filter No. 2 Rehabilitation	LS	1	$45,000	$45,000
Distribution Line Repair	LF[4]	500	$100	$50,000
Water Treatment and Storage System Priority One Upgrade Cost Subtotal				**$429,000**
Engineering[5] (20%)				$85,800
Contingency (20%)				$85,800
Total Water Treatment and Storage System Priority 1 Upgrade Cost, Call:				**$600,000**

1. Cost estimate from the Town of Scotia Detailed Engineering Analysis (SHN, 2009), adjusted for inflation
2. LS: lump sum
3. Estimate assumes discharge to the log pond will be permitted
4. LF: linear foot
5. Engineering includes design, permitting, and construction management for the project.

5.5.2 Priority Two Improvements

Priority two improvements are necessary for the long-term viability of the plant, but not an immediate issue with regard to water quality or compliance.

5.5.2.1 Replacement of Filter Plant

The existing pressure filters have exceeded their useful life. Good maintenance has kept them in functioning condition, and until recently, they have produced high-quality water that has met regulatory requirements. However, the results of recent inspections indicate that there is significant damage to the filter bulkheads and laterals. Although rehabilitation of the filters is possible, replacement of the filters, associated piping, and controls is a more cost-effective approach with a longer life span.

5.5.2.2 Raw Water Tank Solids Removal

Currently, the raw water tank must be emptied and taken out of service for the removal of solids. Installation of a suction header to allow removal of solids without emptying the tank is recommended.

5.5.2.3 Seismic Retrofit

The 1.0-MG raw water storage tank and 0.488-MG finish water storage tank are inadequately tied to their foundations to resist loads imposed by the design earthquake. It is recommended that a new reinforced concrete foundation collar be installed around the raw water tank, and that a series of tie-down saddles be welded to the bottom of the tank with hold-down bolts extending into the foundation. Similarly, the 0.488-MG tank seismic retrofit will also be included in the Community Services District (CSD)'s priority improvements.

Table 5-6
Estimated Cost of Water Treatment and Priority Two Upgrades (Rev. 10/7/15)
TOS Detailed Engineering Analysis

Item (Unit Type)	Unit(s)	Quantity	Unit Cost	Total Cost
Filter Plant Replacement	LS[1]	1	$860,000	$860,000
Raw Water Tank Suction Header	LS	1	$20,000	$20,000
Seismic Retrofit of 0.488-MG[2] Tank[3]	LS	1	$175,000	$175,000
Seismic Retrofit of 1.0-MG Tank[3]	LS	1	$265,000	$265,000
Water Treatment and Storage Priority Two Upgrades Subtotal				**$1,320,000**
Engineering[4] (20%)				$264,000
Contingency (30%)				$396,000
Total Water Treatment and Storage Priority Two Upgrades Cost, Call:				**$2,000,000**

1. LS: lump sum
2. MG: million gallons
3. Cost estimate from the Town of Scotia Detailed Engineering Analysis (SHN, 2009), adjusted for inflation
4. Engineering includes design, permitting, and construction management for the project.

5.5.3 Operational Improvements

5.5.3.1 Issues of Operation

This section lists the performance limiting factors that were identified for the CSD formation Below each problem is a recommendation in *Italics* that may reduce or eliminate the problem.

Issue 1:	There is no central location where the storage tank levels are monitored. Monitoring of reservoir levels would simplify tracking of water volumes in the system, and when combined with pump and flow meter data, would help to identify major leaks.
Recommendation 1:	*Assess existing telemetry system and upgrade to provide monitoring capability.*
Issue 2:	There is no supervisory control and data acquisition (SCADA) system or other means of continuously monitoring water quality and flows at the WTF; all readings and measurements are done manually on a daily basis by the individual operators.
Recommendation 2:	*Install a SCADA system that monitors the WTF and water storage facilities, controls the treatment process, records water quality and production on a continuous basis, and sounds alarms and/or shuts down the treatment system in the event of an equipment malfunction. The SCADA system will provide continuous information on pump operation, water tank levels, water quality and flow rates, chlorine doses and residuals, coagulant doses, and plant operation including backwash cycles, as well as other operational monitoring and controls. The system will also provide a computerized interface to allow operators to easily control the facility processes, and alarms and shut-downs for system malfunctions and equipment failures.*
Issue 3:	The gas chlorination system has not been assessed for compliance with the *California Fire Code* and Article 80 of the *Uniform Fire Code*.
Recommendation 3:	*Have the system inspected by the Fire Marshal to determine compliance with Article 80 of the Uniform Fire Code (NFPA, 2006), which requires facilities using 150-pound cylinders not equipped with scrubber systems to have the following controls:* • *Approved containment vessels or containment systems* • *Protected valve outlets* • *Gas detection system* • *Approved automatic--closing fail-safe valve* *Switching to hypochlorite is considered as an alternative to upgrading the existing gas chlorination system.*

5.5.3.2 Opinion of Probable Cost

Estimated cost for the capital improvements discussed as issues of concern are itemized in Table 5-7. A more thorough evaluation of the existing systems will be required prior to design of the proposed capital improvements; therefore, these cost estimates are preliminary.

Table 5-7
Estimated Cost of Water Treatment and Storage Operational Needs (Rev. 10/7/2015)
TOS Detailed Engineering Analysis

Item (Unit Type)	Unit(s)	Quantity	Unit Cost[1]	Total Cost
Improvements to Reservoir Telemetry	LS[2]	1	$65,000	$65,000
SCADA[3] System	LS	1	$130,000	$130,000
Water Treatment and Storage Operational Needs Subtotal				**$195,000**
Engineering[4] (20%)				$39,000
Contingency (20%)				$39,000
Total Water Treatment and Storage Operational Needs Cost, Call[5]:				**$273,000**

1. Cost estimate from the Town of Scotia Detailed Engineering Analysis (SHN,2009), adjusted for inflation
2. LS: lump sum
3. SCADA: supervisory control and data acquisition
4. Engineering includes design, permitting, and construction management for the project
5. Not included in initial capital improvement program

6.0 References

SHN Consulting Engineers & Geologists, inc. (May 2009). Town of Scotia Community Service District, Detailed Engineering Analysis, Revision 3, Development of the Scotia Community Services District LAFCo Application, (Appendix A to the Municipal Service Review). Eureka, CA:SHN.