WASTEWATER TREATMENT
CORRECTIVE ACTION REPORT
CITY OF FERNDALE

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1 EXECUTIVE SUMMARY

The City of Ferndale operates a municipal Wastewater Treatment Facility (WWTF) with an average flow of 0.5 MGD and is permitted for a design flow of 1 MGD. The facility provides secondary treatment using stabilization pond processes followed by chlorine disinfection before discharge of effluent to land application or to surface receiving waters. Under the City's discharge permit (NPDES Permit No. CA0022721), wastewater is discharged to overland irrigation May 15 through September 30 and to the Salt River October 1 through May 14. Due to recent physical environment changes in the area, the City cannot remain compliant with its NPDES permit condition requiring a 100:1 dilution when discharging into the Salt River. The City was issued a Cease and Desist Order (CDO No. R1-2003-0049) by the RWQCB on May 15, 2003, with a task list outlining a timetable for compliance. In accordance with the Cease and Desist Order, the City has investigated wastewater treatment and disposal options. The findings of the investigation are summarized in this report.

Several alternatives were investigated. The alternatives focused on different corrective actions that could be implemented at the City's WWTF, and which would protect the beneficial uses of the receiving water. Two alternatives would allow the City to meet its dilution requirement without additional treatment. One of these would involve moving the outfall of the facility to the Eel River where the river flow would adequately dilute (100:1) the WWTF's effluent. Another alternative involves acquiring an area of land large enough to construct a storage pond to retain the WWTF's effluent until sufficient flows in the Salt River are available to discharge at the 100:1 dilution ratio.

Four alternatives explore different treatment technologies that are capable of producing effluent treated to a level that would allow discharge at a reduced dilution ratio. These alternatives could potentially enhance the water quality of the Salt River, and would require a conditional waiver from the Regional Water Quality Control Board for a reduction in the dilution requirement with the advanced treatment. The No Action alternative is also discussed; in this case the City would remain non-compliant with its discharge permit. Doing nothing is not a viable option to the City if it wishes to avoid significant fines and the potential for a moratorium on development.

The existing headworks treatment and pumping equipment is outdated and subject to excessive maintenance and repairs. The site is also low lying and subject to flooding during heavy wet weather, and cannot be easily accessed under these conditions. All of the alternatives, except the No Action Alternative, include an upgrade or replacement of the existing influent pump station and headworks.

Following is a summary of the capital and annual operations and maintenance costs for each alternative:

NO.	ALTERNATIVE	EST. CAPITAL COST	EST. ANNUAL O&M COSTS
1	Pump to Eel River	$3,927,000	$29,000
2	Treatment Marsh	$2,950,000*	$42,000
3	Storage Pond	$3,226,000*	$13,000
4	Tertiary Treatment of Lagoon Effluent	$7,432,000	$59,000
5	Sequencing Batch Reactors	$6,768,000	$36,000
6	Membrane Bioreactor	$8,570,000	$43,000
7	No Action Alternative	$0	$0

*Does not include land acquisition

The analysis of alternatives led to the recommended project centered around a Sequencing Batch Reactor (SBR) treatment process. SBRs are mechanically aerated vessels that could be constructed at the WWTF site without interrupting the operation of the existing treatment plant. This would allow the upgrade to occur while the City continued to process its wastewater and meet most of its discharge requirements. The SBR process is capable of producing consistently high quality effluent that also removes ammonia and nitrogen by-products, and therefore justifies the needed reduction in dilution ratio upon discharge to Francis Creek. The high quality effluent from the SBRs also allows a change in the method of effluent disinfection. An ultraviolet radiation disinfection process can be installed in place of the current gaseous chorine and sulfur dioxide systems that pose health and safety risks and require significant monitoring and handling tasks.

There is currently a shortage in wastewater operations staff at the WWTF. It is recommended that the City add one position to the operations staff, regardless of which alternative is selected, including the No Action Alternative.

2 BACKGROUND AND NEED FOR PROJECT

2.1 Introduction

The City of Ferndale's Wastewater Treatment Facility (WWTF) is located north of the City near the confluence of Francis Creek and the Salt River (Figure 1). The WWTF treats municipal wastewater from the City's 1,500 residents, outlying areas, and several commercial facilities. The effluent is treated through a series of processes and applied to neighboring farmlands as irrigation May 15 through September 30. During the remaining portion of the year, the effluent is discharged to Salt River. Francis Creek is the only tributary of the Salt River, thus the Salt River is considered the receiving water body. The discharge is permitted by the North Coast Regional Water Quality Control Board (RWQCB) through the National Pollution Discharge Elimination System (NPDES Permit No. CA0022721).

Due to morphology in the Eel River Valley, flow in the Salt River has been significantly reduced. Over the last few years, when the WWTF discharges to the Salt River, the flow in the receiving body is often not sufficient to achieve the dilution requirements. Based on the flows measured in Francis Creek, effluent is currently discharged with an average dilution of 3:1. Thus, the City is in violation of the dilution requirement of 100:1 established by the *Water Quality Control Plan for the North Coast Region (Basin Plan)* and its NPDES permit for discharge into the Salt River. The City is actively investigating alternatives to achieve compliance with its disharge permit by complying with the tasks outlined in the City's Cease and Desist Order issued on May 15, 2003.

This report outlines the background of the City's WWTF and effluent discharge. An investigation of corrective action alternatives and the preferred alternative are also included. This report serves as a continuing effort by the City of Ferndale to increase the efficiency of its facility, protect the receiving water body, and establish compliance with its NPDES permit.

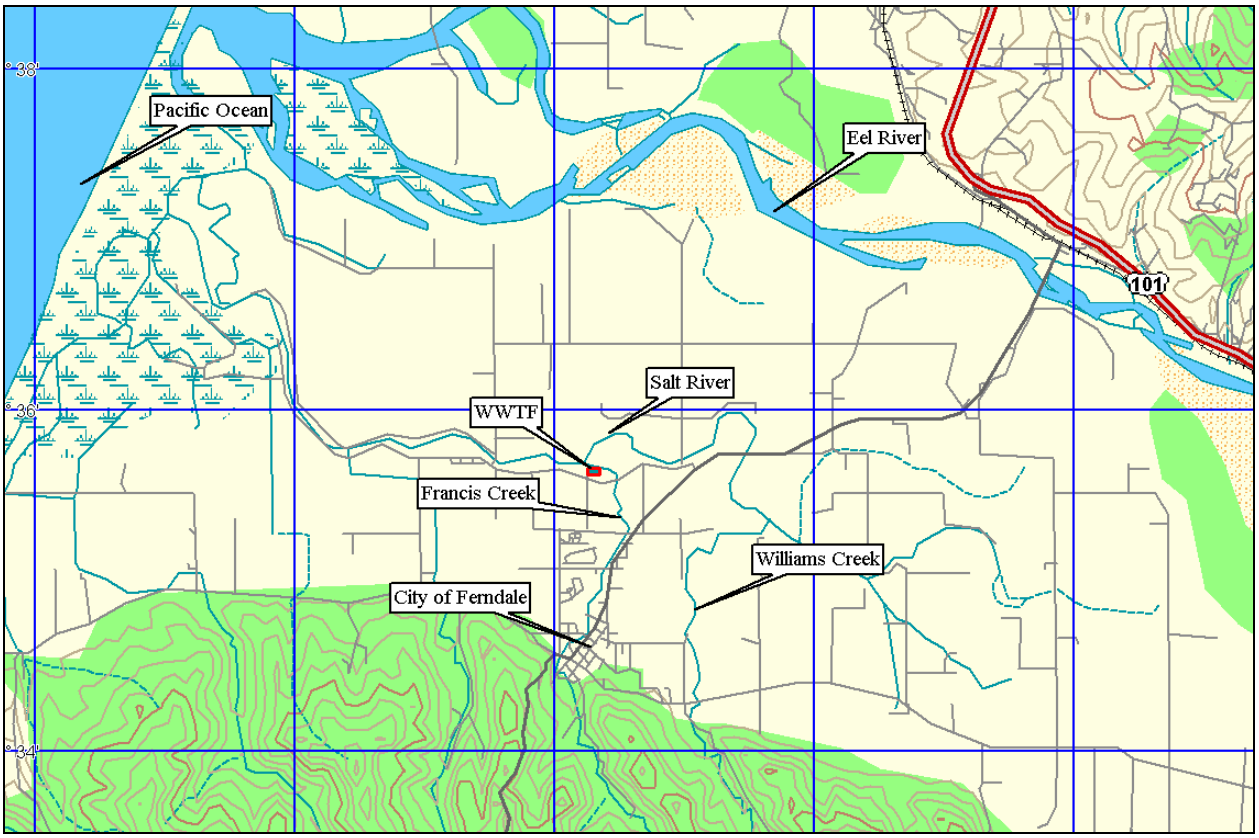


Figure 1: Vicinity Map

2.2 1998 Flooding Event/Impact on Wastewater Facilities

The City's WWTF was operating without serious incident prior to the storms in the spring of 1998. Those storms have since been declared a federal disaster designated as the FEMA 1203 El Niño storm disaster. Sometime in early April 1998, a storm caused Williams Creek to change course due to a debris blockage upstream of its confluence with the Salt River. Williams Creek was the main tributary to the Salt River near the WWTF, but since the blockage, it no longer contributes flow to the Salt River. Currently, the main flow of the Salt River is from Francis Creek, a smaller tributary that flows around the WWTF. Without the contribution of flow from Williams Creek into the Salt River, the 100:1 outfall dilution requirement can no longer be met.

The WWTF does not have adequate storage capacity to hold the effluent until adequate flows are available to discharge. With average autumn influent rate of 0.25 MGD, the WWTF ponds can hold approximately two weeks of storage before they reach maximum capacity. Stream flows are now minimal at best since Francis Creek is a small watershed that rises and quickly subsides with each rain event.

2.3 Noncompliance with Discharge Permit

2.3.1 Discharge Requirements

Effluent discharged from Ferndale's WWTF is required to meet specific standards (Table 1) as indicated in the City's NPDES permit (Appendix A). The effluent has been monitored and recorded since at least 1996. Monitored constituents include average and maximum effluent coliform, average effluent and percent removal of BOD and suspended solids, average and maximum effluent settleable solids, chlorine residual, and pH. With the recent acquisition of equipment, the City also tests for dissolved oxygen and temperature in the aeration pond, chlorine contact basin, and upstream and downstream of the WWTF. Ammonia is also monitored upstream and at the discharge point of the WWTF. The standards for water discharged into the Salt River (during the wet season) and water discharged for irrigation (during the dry season) are the same, with the exception of BOD_5 and the maximum mean daily dry weather flow. The maximum mean daily dry weather flow is not to exceed a monthly average of 1.0 mgd.

Table 1: City of Ferndale's NPDES Effluent Discharge Standards

		DISCHARGE TO CREEK		DISCHARGE TO IRRIGATION	
Constituent	**Unit**	**Monthly Ave.**	**Daily Max.**	**Monthly Ave.**	**Daily Max.**
Coliform	MPN/100ml	23	230	23	230
BOD (20° C, 5 day)	mg/l	30	-	50	80
	lb/day	250	-	417	-
Suspended Solids	mg/l	11	-	11	-
	lb/day	792	0.2	792	0.2
Settleable Solids	ml/l	0.1	-	0.1	-
Hydrogen Ion	pH		6.5<pH<8.5		

Additional standards are as follows:

- 85% removal for BOD_5 and suspended solids between the influent and the effluent (difference in the arithmetic mean taken from 30 consecutive days).
- Minimum of 1.5 mg/L of chlorine residual at the end of the disinfection process.
- Given a minimum detection level of 0.1 mg/L, no detectable levels of chlorine shall be discharged into the Eel River or its tributaries.

The NPDES permit also stipulates the following discharge prohibitions:

- sludge or digester supernatant;
- untreated waste from anywhere in the facility;
- May 15 through September 30; and
- October 1 through May 14 if the effluent flow exceeds 1% of the combined flow of Francis Creek and the Salt River (100:1).

2.3.2 Violations

The City monitors the required parameters and reports to the RWQCB monthly. Water quality violations measured and recorded at the WWTF for a period of record from 1997 to 2002 are summarized in Table 2. Discharge Monitoring Reports (DMRs) identify the water quality violations by type for each month of the year. The records include violations for coliform, chlorine residual, pH, average effluent and percent removal for BOD_5, settleable solids, and suspended solids.

Table 2: Effluent Water Quality Violations for WWTF

	1997	1998	1999	2000	2001	2002	**Total**
Coliform	12	12	1	10	3	3	**41**
Cl_2 Residual	8	0	0	0	0	0	**8**
pH	1	1	3	0	0	0	**5**
BOD (20°, 5-day)- % Removal	1	7	8	0	7	4	**27**
BOD (20°, 5-day)- Ave. Effluent	2	2	0	0	0	0	**4**
Settleable Solids	1	0	0	0	0	0	**1**
Suspended Solids	0	5	0	0	0	0	**5**
TOTAL VIOLATIONS	**25**	**27**	**12**	**10**	**10**	**7**	**91**

The results of the water quality monitoring and reporting of violations indicate that over the last six years, the overall number of violations has decreased. Most of the violations are coliform and % removal of BOD_5. The reductions in violations are likely a result of improvements made to the facility and operator diligence.

The violations associated with the dilution reduction are more frequent that water quality violations. During the period of record, the facility violated the NPDES permit requirement of 100:1 dilution everyday it discharged to the Salt River. The largest dilution recorded was 23:1, but the average was much lower, at 3:1.

2.4 Health and Safety Concerns

A Beneficial Use Analysis was performed by Spencer Engineering and Construction Mangement in November 2002 to assess the downstream uses of the Salt River. The analysis concluded that none of the interviewed parties use the Salt River for recreational activities involving body contact with the water (REC-1). However, the interviews indicated that uses of the Salt River for activities in close proximity to the river, but normally without body contact with the Salt River are prevalent (REC-2). Non-contact recreational uses performed along the Salt River include picnicking, hiking, boating, and hunting. There are no public access areas along the Salt River, thus use by the public is minimized. Landowners also noted that there are no odor problems associated with the WWTF's discharge into the Salt River.

California Department of Fish and Game has indicated that there are several anadromous fish species supported by the Salt River. Species may include the following:

1. coastal cutthroat trout (*Onocorhynchus clarki),*
2. steelhead (*Onocorhynchus mykiss),*
3. Chinook salmon (*Onocorhynchus tshawaytcha)*, and
4. coho salmon (*Onocorhynchus kisutch).*

Of the aforementioned species, cutthroat trout are the only ones not listed as threatened under the Federal Endangered Species Act; however, they are a California Fish Species of Special Concern. The Fish and Game Commission determined that coho salmon in California are a threatened species north of Punta Gorda, and the Department is proceeding in the development of a coho salmon recovery strategy plan.

The California Department of Fish and Game has been solicited for consultation by the RWQCB regarding the issue of the City's request for dilution reduction. The Department issued a request for more information in April 2003 and made several recommendations to the City. Since that time, the requested information has been submitted to the Department and the recommendations have been implemented. Correspondence and meetings have taken place between the Department, the City, and Spencer Engineering during which solutions to the City's dilution violation have been discussed. Currently, the Department has stated that they would "likely" support a dilution reduction if the effluent quality can be treated to a level that exceeds the receiving water body.

3 EXISTING FACILITIES

3.1 System Description and Condition

The City's WWTF was originally constructed in the 1970s and extensively improved over the last 30 years. The WWTF currently discharges treated effluent to the Salt River October 1 through May 14 and applies the effluent to neighboring farmlands as a source of irrigation May15 through September 30. The facility includes headworks, aeration pond, finishing pond, chlorine contact basin, and dechlorination system (Figure 2).

The collection system throughout the City is gravity-fed to the headworks building. The facility's headworks building houses several preliminary process components. A flow meter located within the headworks structure measures the amount of wastewater influent to the facility. The original design included a communitor in the the headworks building to grind and break up the solids before it enters the wet well area, however the communitor has been inoperable for two years. Additional controls in the headworks building include four (4) timers for the aerators located in the aeration pond. The influent pump station, also located in the headworks building, consists of two 25 horsepower (Hp) submersible Flyte pumps that lift the wastewater from the headworks to the aeration pond.

The wastewater enters the five-acre aeration pond where it undergoes biological treatment. The treatment is aided by six submerged fine bubble air diffuser grids. The air diffusers are connected to two 2-inch air headers supplied by two 5 Hp blowers located on the levy of the pond.

From the aeration pond, the wastewater flows through a manually controlled flow gate to the finishing pond. The finishing pond is approximately one acre and provides enhanced treatment of the wastewater. From the finishing pond, the wastewater flows through a control gate to the chlorine contact pond. The control gate can be used to adjust the flow into the chlorine contact basin. The gate can be closed completely to allow the chlorine contact basin to be cleaned.

The chlorine contact basin has undergone several improvements to achieve its current layout. In 1994, a new dividing wall between the finishing pond and the chlorine contact chamber was constructed to replace the original leaking masonry block wall. The most recent changes to the chlorination basin were completed in 2001. At that time, the chlorine contact basin was extensively redesigned and an automated chlorination system was installed. The basin was lined with concrete and the fiberglass baffles were extended to achieve serpentine flow conditions. Currently, the chlorine contact basin is used to inject chlorine to disinfect the wastewater. During winter months, sulfur dioxide is used to neutralize (dechlorinate) the chlorine before it is discharged to the receiving water body.

The chlorine control building houses various instruments to control the treatment of the wastewater. The building also contains various lab equipment to monitor and adjust parameters of the wastewater treatment processes manually if required. All the treatment components are directly linked to an electrical control panel. There is also a generator-powered backup control

panel that can be used in case of power outage to operate the disinfection equipment and the effluent pumps. The alarm system has an automatic dial-up to alert City staff in the case of alarms or power outages at the facility. Additional controls located in the chlorine contact building include the following:

- The chlorine leak detector detects chlorine gas levels and can be adjusted from 0 parts per million (ppm) to 10 ppm. Similarly, the sulfur dioxide detector identifies sulfur dioxide leaks and is adjustable from 0 ppm to 10 ppm.
- The chlorine residual analyzers automatically read the chlorine residual, with the use of buffer solutions, from set points in the chlorine contact chamber.
- The chlorine controllers relay the signal from the chlorine analyzers to the controller. The controller adjusts the chlorine feed rate to maintain the required chlorine residual in the contact pond. The manual chlorine regulator is a back-up system in case of technical difficulties with the automatic chlorine controller.
- The sulfur dioxide controller is given a signal from the second chlorine analyzer and adjusts the amount of sulfur dioxide accordingly to neutralize the chlorine. This device is primarily used in winter months when discharging to Francis Creek. The manual sulfur dioxide regulator is a back-up system in case of technical difficulties with the automatic sulfur dioxide controller.
- The chlorine injector device injects chlorine into the chlorine contact basin.
- The Chlor-o-vac unit injects and mixes the sulfur dioxide at the point of discharge in the chlorine contact basin. The sulfur dioxide injector injects sulfur dioxide at the point of discharge if the Chlor-o-vac unit is offline.

After the wastewater is disinfected and dechlorinated, it is pumped via an effluent pumping system consisting of three pumps to either Francis Creek (wet season) or to land irrigation (dry season). As the effluent is discharged, the flow is measured by an effluent flow meter. The pumps can be controlled automatically by floats located in the contact pond or, when necessary, manually. The control system incorporates the following operational modes:

- One 15 Hp pump is used for low flows at the treatment plant or in combination with the 20 Hp pump during high flows.
- One 20 Hp pump is used for moderate flows at the treatment plant or in combination with the 15 Hp pump during high flows.
- One 10 Hp pump is strictly used during high flow conditions when the other effluent pumps can not keep up with the influent flows.

(ACTUAL FIGURE TO BE INSERTED FROM AUTOCAD)

Figure 2: Site Plan

3.2 Plant Flows and Organic Loading

The WWTF currently treats an average of 0.5 MGD, but historic data indicates that the incoming flow varies from a low of 0.14 MGD during dry weather to a peak day of 3.85 MGD during wet weather events. Flow and design criteria are developed based on flow data for the period from July 1996 to January 2003.

Criteria for choosing and sizing treatment process alternatives are developed based on plant organic loading data obtained during the period from July 1996 to January 2003 along with the hydraulic loading criteria discussed below.

3.2.1 Wastewater Flowrate Terminology

General terms used in the analysis of hydraulic loading to the plant are defined and presented prior to the discussion and results of the statistical analysis. Terms include:

3.2.1.1 Base Sanitary Flow

The minimum repeated flow recorded during the driest months of the year is typically referred to as the Base Sanitary Flow. This flow represents the domestic component of the wastewater in the sanitary sewer system resulting from the use of potable water. The Base Sanitary Flow is estimated to be 0.14 MGD.

3.2.1.2 Base Infiltration and Inflow

The average amount of extraneous water entering the sewer system during the dry season is referred to as base infiltration and inflow (Base I/I). This parameter is determined by subtracting the Base Sanitary Flow from the Average Dry Weather Flow. In general, the Base I/I cannot be removed from the system and an allowance for this flow is typically included in the estimate of flows for each future connection.

3.2.1.3 Average Dry Weather Flow

The average daily flow in the sewer system occurring during the dry season, from May 15 through September 30, is the Average Dry Weather Flow (ADWF). This flow is based on the period when discharge is prohibited. The ADWF for the City's WWTF is estimated to be 0.21 MGD.

3.2.1.4 Average Wet Weather Flow

The average daily flow in the sewer system occurring during the wet season, between the months of Sept-May 15th (period of discharge) is referred to Average Wet Weather Flow (AWWF). The AWWF for the City's WWTF is estimated to be 0.67 MGD.

3.2.1.5 Average Annual Flow

The Average Annual Flow (AAF) is the flow average for the entire year. The AAF for the City's WWTF is estimated to be 0.50 MGD.

3.2.1.6 Maximum Month Dry Weather Flow

The maximum month dry weather flow (MMDWF-10) is the monthly average flow that has a 10% probability of occurrence from May to October in any given year. This flow represents the wettest dry weather season monthly average flow that will probabilistically occur once in ten years. The highest monthly average dry weather flow typically occurs in May.

MMDWF-10 is estimated from a regression of cumulative monthly rainfall versus the mean daily flow for that month. The monthly rainfall with a 10-year reoccurrence interval was equal to 3.99 inches/month based on cumulative monthly precipitation data from a rainfall gauging station in Scotia for the month of May. The MMDWF-10 is estimated to be 0.67 MGD (Figure 3).

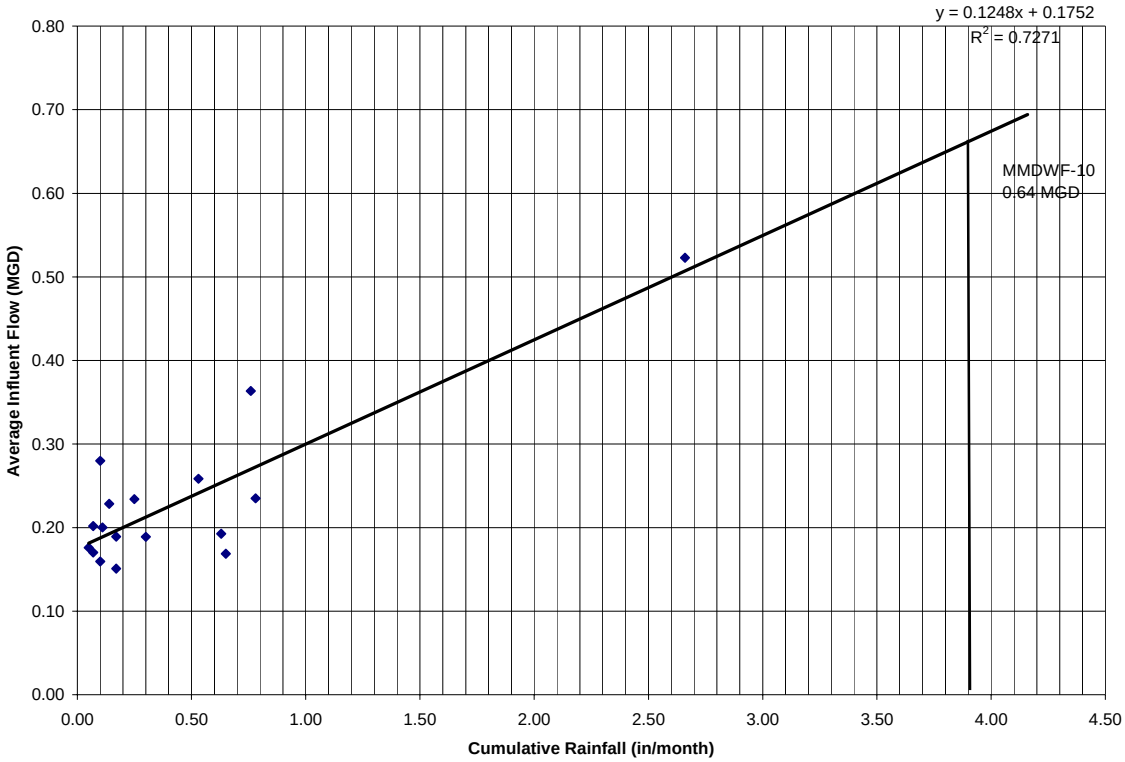


Figure 3: Maximum Month Dry Weather Flow for a Ten-Year Storm Event (MMDWF-10)

3.2.1.7 Maximum Month Wet Weather Flow

The Maximum Monthly Wet Weather Flow (MMWWF-5) is the monthly average flow that has only a 20% probability of occurrence during the wet-weather period in any given year. This flow represents the wettest wet season monthly average flow that is anticipated to have a five-year recurrence interval. In observing the precipitation data for the weather stations at Eureka and Scotia, the month of December had the highest average precipitation.

MMWWF-5 is estimated from a regression of cumulative monthly rainfall versus the mean daily flow for that month. The monthly rainfall with a 5-year reoccurrence interval was equal to 13 inches/month based on cumulative monthly precipitation data from a rainfall gauging station in Scotia for the month of December. The MMWWF-5 is estimated to be 1.47 MGD (Figure 4).

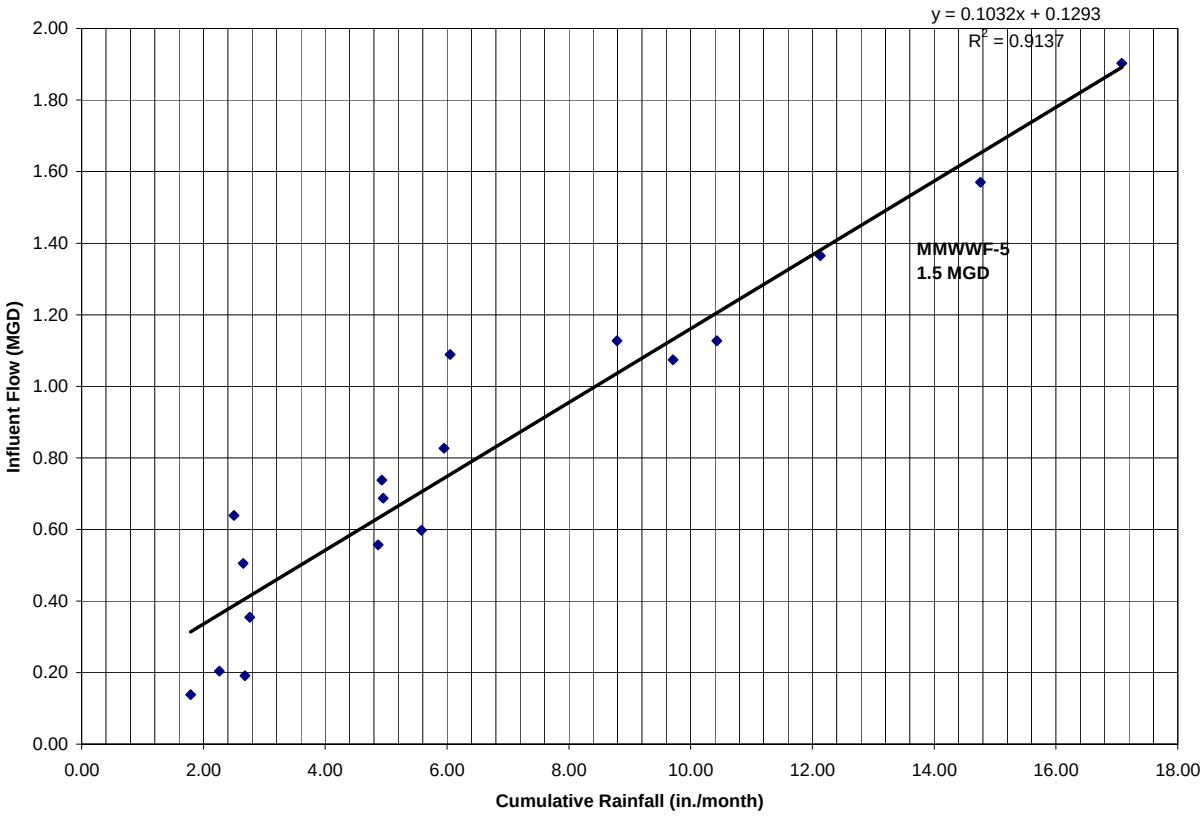


Figure 4: Maximum Month Wet Weather Flow for a Five-Year Storm Event (MMWWF-5)

3.2.1.8 Peak Day Average Flow

The Peak Day Average Flow (PDAF) is the largest daily flow experienced over a 24-hour period during any year. A regression analysis of daily plant flows versus precipitation during or immediately following wet season significant rainfall events is used to estimate the PDAF for a five-year storm event (PDAF-5). Based on precipitation frequency curves for northern California prepared by the National Oceanographic Administrative Association (NOAA) the 24-hour storm event with a 5–year return interval is 4.0 inches. Based on this rainfall and the regression presented in Figure 5, the PDAF-5 is estimated to be 3.70 MGD (Figure 5).

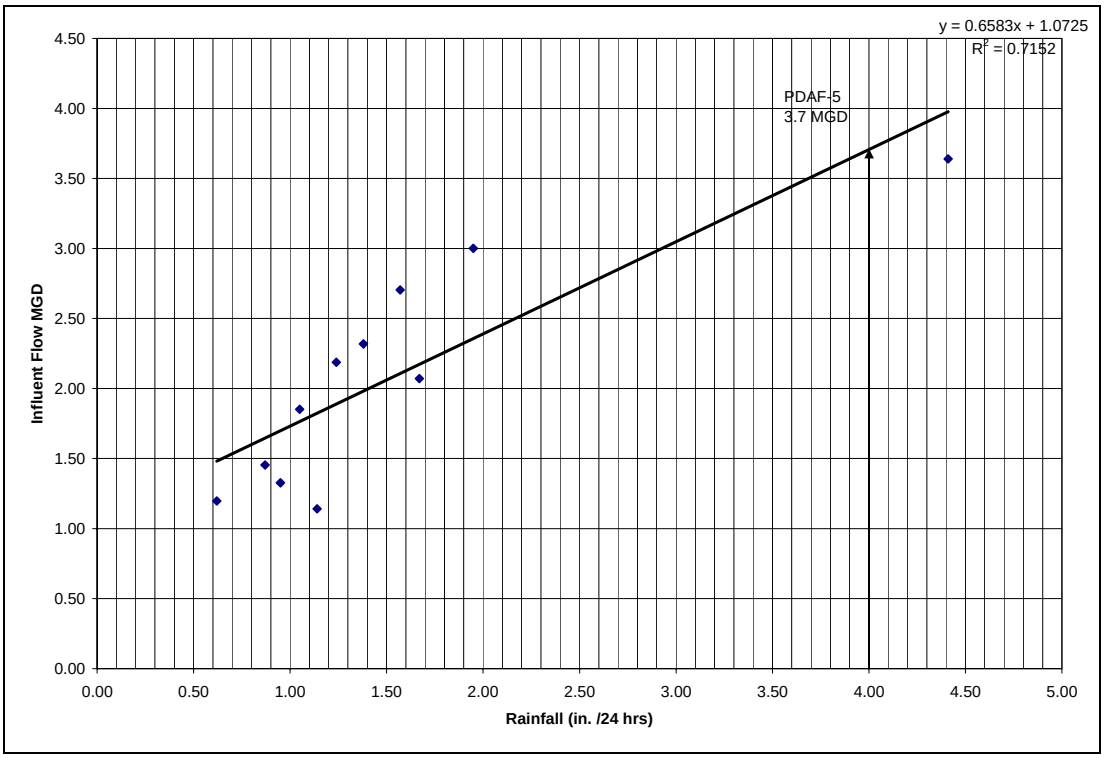


Figure 5: Peak Day Average Flow (PDAF)

3.2.1.9 Peak Instantaneous Flow

The Peak Instantaneous Flow (PIF) is the highest sustained hourly flow rate during wet weather. The PIF can be estimated from a probability projection based on the following probabilities of occurrence:

- An annual average flow is likely to occur 6/12 of the time or with a probability of 50%
- A maximum month flow is likely to occur 1/12 of the time or with a probability of 8.3% (use MMWWF-5 to estimate PIF-5).
- A peak week flow is likely to occur 1/52 of the time or with a probability of 1.9%
- A peak day flow is likely to occur 1 /365 days or with a probability of 0.27% (use PDAF-5 to estimate PIF-5).
- The peak instantaneous flow is likely to occur 1/8,760 hours (i.e. 1 hour in a year) or with a probability of 0.011%

Flow rates and their associated probabilities are graphed on logarithmic probability paper and the projection is used to estimate the PIF-5. The peak instantaneous flow for a five-year storm event PIF-5 at the City of Ferndale WWTF is estimated to be 6.0 MGD (Figure 6). This PIF flow parameter provides the basis for the hydraulic design of pumping facilities.

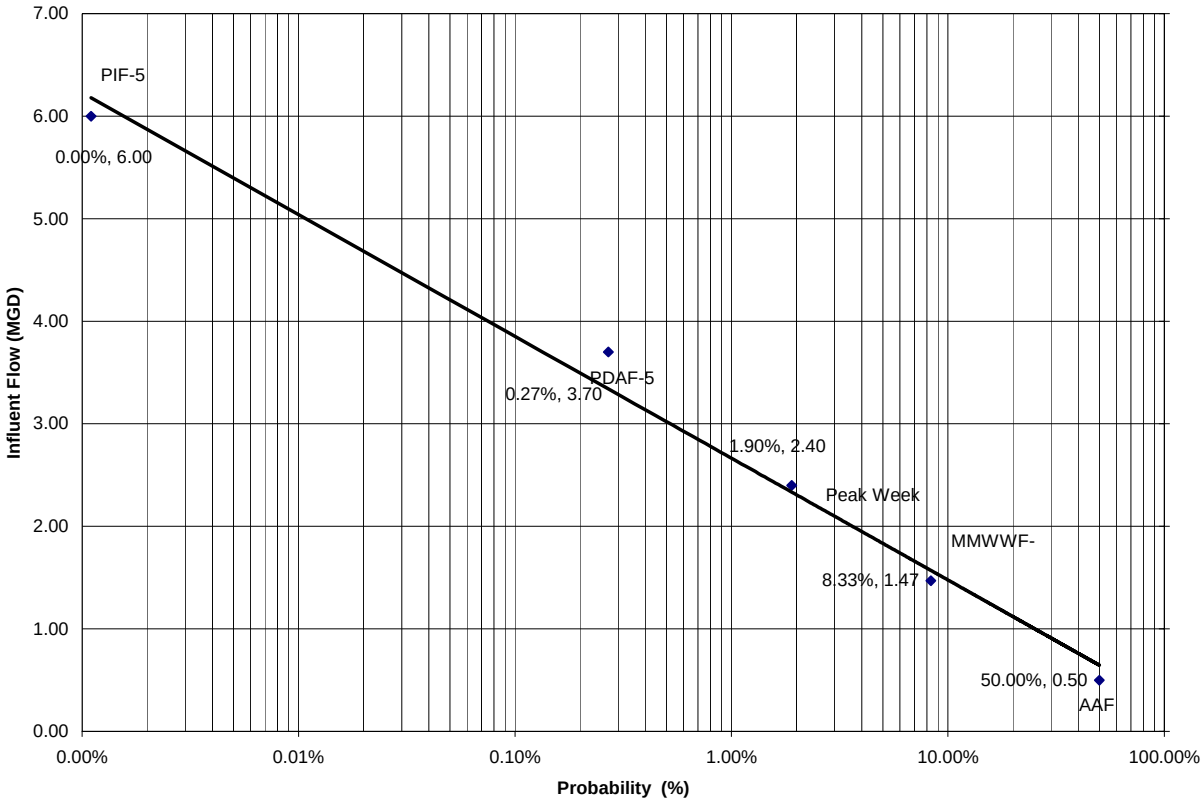


Figure 6: Peak Instantaneous Flow

3.2.2 Flow Summary

The flow data developed in the previous section are summarized in Table 3. The historical average flow rates and maximum monthly flow rates are presented in addition to the values associated with a particular return interval. Values used for design should be based upon a reasonable risk. Flow rates associated with a five-year return interval in the winter (20% chance of failure or overflow) and a 10-year return interval in the summer (10 % chance of failure or overflow) is commonly used to establish minimum baselines for design flows.

In the case of Ferndale, the maximum monthly flowrate associated with a five-year storm MMWWF-5 was exceeded in February 1998 when the average monthly flow was 1.90 MGD. Flows from these storm events caused damage to the facilities outfall and are the basis of the City's application to FEMA for assistance. As a conservative value and based upon past effects, the historical maximum month flow of 2.0 MGD will be used as a design flow for treatment improvements at the Ferndale facility.

Table 3: Influent Flow Summary

Data Type	Flow Type	Flow	MGD
Historical	Average Wet Weather Flow	AWWF	0.67
Historical	Average Dry Weather Flow	ADWF	0.21
Historical	Average Annual Flow	AAF	0.50
Historical	Base Sanitary Flow	Base	0.14
Historical	Maximum Month Wet Weather	MMWWF	1.90
Historical	Peak Week Wet Weather Flow	PWWWF	2.34
Historical	Peak Day Average Flow	PDAF	3.85
Historical	Maximum Month Dry Weather	MMDWF	0.52
Statistical	Maximum Month Dry Weather–10 Yr. Storm	MMDWF-10	0.64
Statistical	Maximum Month Wet Weather-5 Yr. Storm	MMWWF-5	1.47
Statistical	Peak Day Average Flow-5 Yr. Storm Event	PDAF- 5	3.70
Statistical	Peak Instantaneous Flow-5 Yr. Storm Event	PIF-5	6.0

3.2.3 Organic Loading and Performance Summary

Summaries of influent loadings are presented in Tables 4 and 5 and are based on data provided by the City operations staff for biological oxygen demand (BOD) and total suspended solids (TSS) in the year 2002. Estimates of loading in lbs/day are based on a single monthly sample for BOD and TSS and the average monthly flow. Per capita loadings are within the ranges expected for a small city without large industrial contributions.

Effluent quality in terms of BOD and TSS is also included in the Tables 4 and 5 for 2002. Average values for effluent BOD concentrations over the last seven years were 13 mg/l, although maximum values over 20 mg/l were not uncommon. TSS over the same time period averaged 20 mg/l, with maximum values exceeding 50 mg/l frequently. The maximum suspended solids value recorded during this time period was 290 mg/l. Figure 7 and 8 illustrate BOD and TSS concentrations over the period of record.

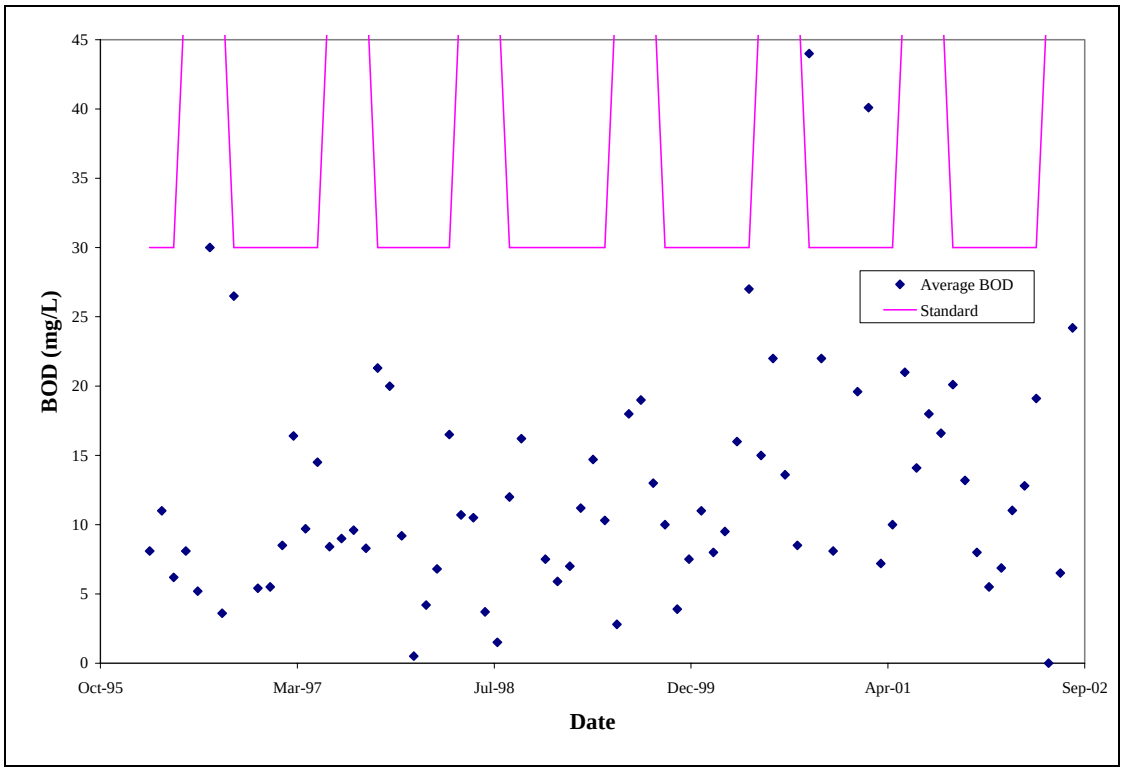


Figure 7: Effluent BOD

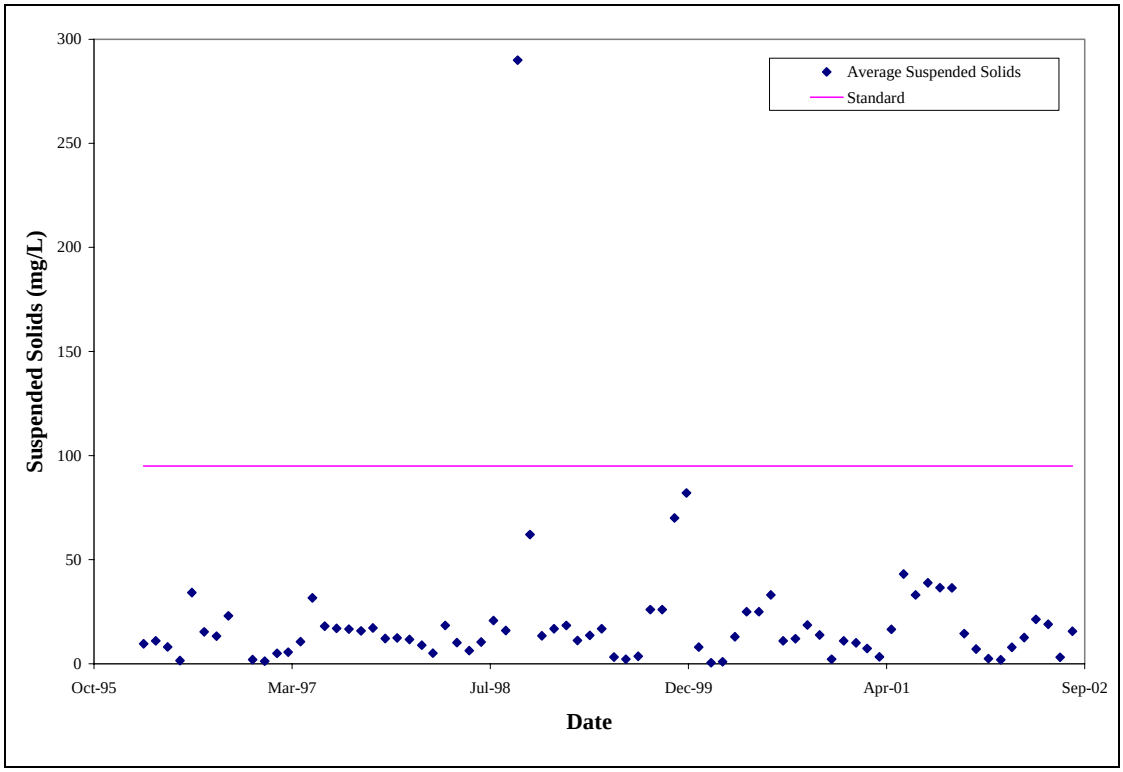


Figure 8: Effluent TSS

Table 4: BOD Loading and Removal

Date	Flow (Ave)	Influent		Effluent		Permit	Removal	Permit
	MGD	mg/l	lbs/day	mg/l	lbs/day	mg/l	%	%
Jan-02	0.84	55.30	387.41	5.50	38.53	30.00	90.05	85.00
Feb-02	0.60	51.00	255.20	6.88	34.40	30.00	86.52	85.00
Mar-02	0.56	90.25	421.50	11.03	51.49	30.00	87.78	85.00
Apr-02	0.36	91.75	275.47	12.80	38.43	30.00	86.05	85.00
May-02	0.24	81.00	162.13	19.10	38.23	30.00	76.42	85.00
Jun-02	0.18	279.00	430.00	5.00	7.71	50.00	98.00	85.00
Jul-02	0.16	165.00	219.35	6.50	8.64	50.00	96.06	85.00
Aug-02	0.18	270.00	396.32	24.20	35.52	50.00	91.04	85.00
Sep-02	0.16	193.00	262.37	7.70	10.47	50.00	96.01	85.00
Ave	0.36	141.81	312.20	10.97	29.27	na	89.77	na
Max	0.84	279.00	430.00	24.20	51.49	na	98.00	na
Min	0.16	51.00	162.13	5.00	7.71	na	76.42	na

Table 5: TSS Loading and Removal

Date	Flow (Ave)	Influent		Effluent		Permit	Removal	Permit
	MGD	mg/l	lbs/day	mg/l	lbs/day	mg/l	%	%
Jan-02	0.84	78.00	546.44	2.40	16.81	95.00	96.00	85.00
Feb-02	0.60	110.00	550.44	1.90	9.51	95.00	96.00	85.00
Mar-02	0.56	112.00	523.08	7.90	36.90	95.00	91.30	85.00
Apr-02	0.36	169.00	507.41	12.60	37.83	95.00	91.00	85.00
May-02	0.24	96.82	193.79	21.30	42.63	95.00	78.00	85.00
Jun-02	0.18	210.00	323.66	19.00	29.28	95.00	91.00	85.00
Jul-02	0.16	235.00	312.41	3.10	4.12	95.00	60.00	85.00
Aug-02	0.18	287.00	421.27	15.60	22.90	95.00	95.00	85.00
Sep-02	0.16	37.30	50.71			95.00	83.00	85.00
Ave	0.36	148.35	381.02	10.48	25.00	na	86.81	na
Max	0.84	287.00	550.44	21.30	42.63	na	96.00	na
Min	0.16	37.30	50.71	1.90	4.12	na	60.00	na

3.3 Existing Hydraulic Profile

Figure 9 illustrates the hydraulic profile of the existing WWTF. The profile is based on drawings prepared for the chlorine contact basin improvements and the facility operator's assistance. Pumps control the water level in the chlorine contact basin and the pump floats regulate operation. The elevations shown on the profile must be confirmed by a topographic field survey before any proposed upgrades can be designed.

(ACTUAL FIGURE TO BE INSERTED FROM AUTOCAD)

Figure 9: Existing Hydraulic Profile

3.4 System O&M Concerns

WWTF improvements over the period beginning in 1994 have greatly aided in the operation of the facility, primarily by increasing process automation. However, the operator has indicated several ongoing issues regarding facility operation and maintenance that need attention. Specifically:

1. The chlorine analyzers are very unreliable and require many replacement parts.
2. The two 25 Hp headworks pumps frequently require service, resulting in an approximate cost of $4,000 per year.
3. The effluent discharge pumps only last two to three years and cost $3,000 to $4,000 to replace.
4. The comminutor has been inoperable for two years, a new comminutor/grit removal device is recommended.
5. The facility has neither a bathroom nor a supply of potable water. Potable water would allow the City staff to clean their instruments, wash their hands, and improve the method for cleaning out the chlorine contact chamber (cleaned once a month in the summer only).
6. The sludge accumulating in the aeration pond is currently at a depth of 2 to 3 feet, which is an optimal operational depth; however it is anticipated that the sludge will eventually require removal.
7. There are seasonal algae and duckweed blooms in the aeration pond. The aeration pond is also a host to several species of fish, including goldfish and stickleback.

These are known deficiencies associated with the WWTF and may not necessarily be eligible for FEMA funding. The concerns are incorporated into this report to document them for further investigation.

3.5 Cease & Desist Order

On May 15, 2003, the City was issued a Cease and Desist Order with a request for additional information and monitoring (Appendix B). The Order includes the following time schedule for the City to achieve compliance with its discharge permit:

<u>Task A</u>. *By May 31, 2004, submit a detailed report outlining an alternatives analysis, the proposed method for long-term compliance with Order No. R1-2000-92, and a time schedule for completing specific project milestones. Milestones for gaining compliance with the California Environmental Quality Act and obtaining necessary permits shall be included in the time schedule.*

<u>Task B.</u> *By January 1 and July 1 of each year, until compliance is achieved, submit reports of progress on actions taken to achieve compliance with Order No. R1-2000-92. The progress reports shall include the status of project milestones,*

evaluate the effectiveness of the implemented measures, and assess whether additional measures are necessary to meet the time schedule.

Task C. *By February 2005, achieve full compliance with Order No. R1-2000-92.*

The goal of this report is to fulfill Task A by investigating feasible alternatives and provide a preliminary time schedule for corrective action at the WWTF. This report will also be used as a basis for further addressing environmental and permit issues.

4 ALTERNATIVES ANALYSIS

4.1 General

The following section describes and analyzes six treatment and disposal project alternatives for the City's wastewater treatment facility. There is also a discussion of the No Action Alternative in this section of the report. Two of the options are designed to meet the current 100:1 stream flow to waste flow dilution ratio, including moving the point of discharge to the Eel River, and constructing a storage system that would allow effluent to be stored until creek flows provide the required dilution. The other four options are designed to provide advanced or improved treatment and require a decrease in the current dilution ratio.

One of the primary goals of the four advanced treatment alternatives is to produce a plant effluent that can be discharged to Francis Creek at a reduced dilution ratio without degrading the quality of water in the stream. All four of the described options result in effluent quality that meets or exceeds the current permit limitations in order to prevent such degradation.

4.1.1 Water Quality in Francis Creek

Available water quality data for Francis Creek was provided by City operations staff and is summarized in Table 6. Dissolved oxygen (DO), temperature and ammonia were monitored upstream and downstream of the plant discharge point and give an indication of the impact existing plant effluent has on Francis Creek. The data indicates an average decrease in DO of 1.5 mg/l, an average increase in temperature of 0.1 degree, and an average increase in stream ammonia concentration of 1.5 mg/l, based on a comparison of the upstream data values to the downstream data values. The data record is relatively short, representing the 3-month period from October to December of 2002.

The decrease in dissolved oxygen indicates degradation of stream quality, which should be remedied as part of the proposed treatment process. The impact of effluent ammonia and BOD_5 on the stream is significant and will result in additional oxygen depletion (DO sag) further downstream. The temperature increase is measurable and must be considered in light of the stream's beneficial uses and seasonal fisheries value.

Table 6: Water Quality Parameters and Values in Francis Creek

Date	Upstream Creek				Downstream Creek				Ammonia			
	Morning		Afternoon		Morning		Afternoon		Pond	Eff.	Creek	
	DO	Temp	DO	Temp	DO	Temp	DO	Temp			Up	Down
	mg/l	F°	mg/l	F°	mg/l	F°	mg/l	F°	mg/l	mg/l	mg/l	mg/l
8-Oct-02	7.1	54.7	8.1	60.4					10.5	10.5	0.6	
22-Oct-02			6.5	56.2					10.2	8.9	0.4	
29-Oct-02	5.2	48.9	6.2	55.3					10.0	8.7	0.3	
13-Nov-02	5.0	51.2	6.4	57.7			5.7	57.6				
14-Nov-02	5.3	50.9	6.2	57.4	4.8	51.1	5.8	57.2				
15-Nov-02	5.9	51.4	6.0	57.1	5.1	52.5	4.9	57.9				
18-Nov-02	6.4	56.6	5.5	56.1	5.3	56.9	4.6	57.0				
19-Nov-02	5.9	54.3	5.2	55.5	5.1	54.6	4.4	55.6	10.5	10.0	0.7	2.8
20-Nov-02	5.4	51.8	5.3	56.2	4.3	54.0	4.1	55.2				
27-Nov-02	9.7	51.0			8.2	51.9			10.5	10.5	0.7	2.6
2-Dec-02	11.1	49.9			9.3	50.3			10.3	10.5	0.9	2.8
3-Dec-02	10.1	54.6			9.1	54.0						
4-Dec-02	9.9	54.5			7.4	54.4						
5-Dec-02	10.3	54.3			7.4	54.2						
6-Dec-02	11.1	55.1			7.6	55.7						
9-Dec-02	10.2	58.4							10.5	10.0	n/a	n/a
11-Dec-02	9.4	54.5			8.6	54.4						
12-Dec-02	9.0	58.3			7.6	58.6						
13-Dec-02	9.6	54.5			9.5	54.6						
22-Dec-02	9.7	52.8			9.5	50.3						
23-Dec-02	10.8	50.8			9.7	49.8			10.0	8.5	n/a	n/a
24-Dec-02	10.5	54.3			8.9	54.5						
29-Dec-02	9.9	53.8			9.0	53.8						
30-Dec-02	10.1	53.1			8.7	53.5			9.5	8.5	n/a	n/a
31-Dec-02	10.0	54.1			9.5	54.5						
Averages	8.1	53.0	6.7	56.8	7.6	53.7	4.9	56.8	10.2	9.6	0.6	2.7

4.1.2 Ammonia Removal

Ammonia contained in domestic wastewater such as that generated within the City of Ferndale is a pollutant of concern that demands oxygen and can have very negative effects on a receiving surface water body. In addition to BOD, ammonia utilizes oxygen as biological processes convert ammonia (NH_3) to nitrite (NO_2) and nitrate (NO_3). This process, known as nitrification, consumes oxygen in the receiving water body for significant distances downstream of the point of discharge. In extreme cases, the stream cannot provide the oxygen required to satisfy BOD and nitrification, resulting in a DO deficit that is detrimental to the water body. Ammonia is also toxic to many aquatic organisms including anadromous fish. For these reasons, ammonia is removed from wastewater prior to discharge, and high dilution ratios are often required.

As previously stated, two of the alternatives are designed to meet the current 100:1 dilution ratio, which is a historical requirement that is assumed to be adequate in reducing the negative effects of treated effluent on a receiving water body. The other four alternatives require a significantly reduced dilution ratio, and therefore must treat and reduce ammonia concentrations to levels at or near the existing background levels in the receiving water.

Background ammonia concentrations in Francis Creek are less than 1 mg/l upstream of the plant discharge point based on the 2002 winter monitoring period. Effluent ammonia concentrations must be reduced from the current levels, which averaged 9.6 mg/l during the monitoring period, to a concentration of about 1mg/l, in order to fulfill the goal of zero water quality degradation in the Creek. This is roughly equivalent to an additional 90% reduction of the ammonia discharged from the existing treatment plant.

4.2 Treatment Alternatives Description and Analysis

4.2.1 Eel River Discharge

The first alternative is to relocate the discharge point to a receiving body with sufficient flow to achieve the required 100:1 dilution. The closest water body providing adequate flow for the dilution requirement is the Eel River. An investigation of flows within the Eel River (minimum of 450 cfs at the Scotia, California gauging station) indicates that the river is more than sufficient to provide the 100:1 dilution requirement. Relocation of the discharge point would require approximately two and a half miles of piping to convey the effluent from the WWTF to a discharge point on the Eel River. The pipeline alignment would be east on Port Kenyon Road to Fulmor Road where the pipeline would run north along Fulmor Road to the Eel River. The pipeline alignment would be located within existing rights-of-way dedicated for the roads. The outfall point may be located on private property, requiring the negotiation of an easement by the City although an exact location has not yet been determined (Figure 10).

Two pump stations are included in this alternative. The first pump station would collect the WWTF's effluent and provide pressure-induced flow of the effluent to the Eel River. The second pump station would collect the WWTF's effluent and pump it to the adjacent properties currently used for irrigation application during the dry season. Both pumps would be located within the footprint of the existing WWTF.

The discharge point may be within what may be considered the Eel River estuary, which is prohibited by the *Basin Plan*. An ongoing study by Spencer Engineering is investigating if the proposed outfall location is located within the Eel River estuary. The results of the study would indicate if an exception to this prohibition would be required from the RWQCB or if the outfall location is outside of the Eel River estuary designation. The initial results of the investigation indicate that the proposed discharge location does not have a high enough saline concentration to be considered tidally influenced and would therefore not be considered in the Eel River estuary.

Operationally, this alternative does not include additional treatment technologies or additional operator training. The only additional operation or maintenance required from this alternative

would be for the two new pump stations. Some maintenance would also be required to monitor the condition of the outfall.

The new outfall would require a new NPDES permit which would require extensive environmental review and time. Additional permits include an Army Corps of Engineers Section 401 Permit, a RWQCB Section 404 Permit, and a State Coastal Development Permit. The land required for the piping and the actual outfall location on the bank of the Eel River may be difficult to obtain. Additionally, because the discharge point would be moved to the bank of an actively migrating river, the possibility exists that the river channel could move. If the river channel migrates, relocation of the discharge point may be required or percolation beds could be established on the old river bar and a rapid infiltration gallery could be developed (requiring the reinitiating of the permitting and environmental review process). The environmental review necessary for permitting such a project would be costly and could possibly encounter public opposition.

Advantages:

- The City would be in compliance with its discharge dilution requirement
- Does not include additional treatment technologies
- Does not require additional operator training

Disadvantages:

- Outfall would be on an actively migrating river channel
- Would require additional NPDES and other permitting to discharge into the Eel River
- Moderate operating and maintenance costs
- May face public opposition

Figure 10: Alternative 1- Eel River Discharge

4.2.2 Treatment Marsh

The second alternative is to construct treatment marshes on property adjacent to the existing facility (Figure 11). The treatment marshes would receive secondary treated water from the WWTF and provide additional polishing of wastewater. Wetlands treat wastewater as a result of physical, chemical, and biological processes that occur within the wetland environment. Constructed wetlands are designed to provide a low flow rate and maximize the residence time of water within the system. As the water moves through the wetland, many pollutants either settle and become adsorbed in the sediment, or are transformed by plants and microbes. The plants and microbes absorb the nutrients in the water and use them to facilitate growth. The natural processes occurring in a functional wetland are capable of reducing the concentrations of biochemical oxygen demand, total suspended solids, ammonium nitrogen, total nitrogen, and total phosphorous. Treatment levels are largely a result of environmental factors including water quality, soil properties, and plant type and growth, which can vary seasonally.

Preliminary hydraulic analyses indicate that the required treatment marsh area for the WWTF to discharge at the 100:1 dilution would be over 100 acres. Because acquisition of this much land is infeasible, a reduced area combined with a reduction in the dilution requirement is a more likely option. Further analysis indicates that with a dilution reduction to 20:1, approximately nine-acres of treatment marsh would be required.

The operating depth of the marsh ponds would fluctuate between 3 and 5 feet. At a conservative operating depth of three feet, this would provide a storage volume of about 3.14 million gallons. During late fall and spring flows, which average about 250,000 gallons per day, this would provide storage for about 12 days with no discharge. This is in addition to the 1 to 2 weeks of storage volume available in the existing aeration pond. The treatment marsh would increase the WWTF's storage capacity, provide further wastewater polishing, and improve effluent quality.

The treatment marsh would consist of two cells, with each cell having an open water area and a vegetated portion. The vegetated areas and banks of the marsh will be planted with *scirpus* (bulrush) and *potamogeton* (pondweed) to provide biological treatment. Flow between the two marsh cells and marsh discharge would be facilitated by operation of nine level-control structures. Construction of the marsh would require excavation of approximately 9,500 cubic yards (cy) of material and placement of approximately 8,800 cy of fill material to accommodate the construction of berms and access road. Approximately 73,950 square feet of synthetic liner would be necessary to reinforce marsh berms and prevent leaks. The marsh would have several islands to create habitat for waterfowl. The marsh would be surrounded by a gravel access road to facilitate marsh operation and maintenance. The area surrounding the marsh and access road would be planted with various native trees and plants to create appropriate shade areas for the marsh and further enhance wildlife habitat and marsh aesthetics. The marsh would also be surrounded by chain-link fence and gated (Figure 11).

The proposed site of the marsh is currently owned by a dairy farmer and used as grazing pasture for his dairy cows. The proposed marsh site is currently zoned for agricultural use. The City would acquire the property after the National Environmental Permitting Act (NEPA) and

California Environmental Quality Act (CEQA) permitting process is complete. A lot line adjustment to the City's existing wastewater pump station property would be performed to incorporate the neighboring proposed project site.

The two pump stations to be constructed as part of the treatment marsh alternative would include a marsh pump station and an irrigation pump station. The marsh pump station would collect flows from the existing WWTF chlorine contact basin and pump the collected effluent to the treatment marsh. Approximately 2,500 linear feet of 10-inch diameter force main would be required to transfer flows from the marsh pump station to the treatment marsh. The force main would leave the marsh pump station and run within the pond berms across Port Kenyon Road to the City's existing wastewater pump station yard. From the existing wastewater pump station yard, the force main would continue across the yard and Francis Creek before discharging into the first wetland cell (Figure 11).

The irrigation pump station would collect treatment marsh effluent and pump the effluent to adjacent properties currently used for land irrigation during dry weather. During wet weather, marsh effluent would be discharged to Francis Creek. Flow measuring devices would be installed as part of the project to provide necessary operational data, including volume of WWTF effluent pumped to the marsh, volume of marsh effluent discharged to Francis Creek, and volume of marsh effluent used for land irrigation. The two pump stations would also require electrical improvements such as installing a new underground electrical service to the marsh pump station and new overhead facilities for the irrigation pump station. A gravel access road will be constructed from Market Street, around the marsh and to the irrigation pump station to facilitate maintenance and operation of the pump station and other marsh facilities.

As indicated on the FEMA Flood Insurance Rate Map, the base flood elevation (100-year flood) for the Eel River Valley varies at the site, but is below 22 feet. The completed structural facilities (including treatment marsh berms, and irrigation and marsh pump stations) would be constructed above the 100-year flood elevation as shown in Figure 11. Since the 100-year flood plain extends for many miles towards the Eel River, it is estimated that raising the area required for the marsh berms would have little to no impact on flooding in the surrounding areas.

In 1999, the treatment marshes were chosen as the preferred alternative, but since that time the ability of the proposed marsh to provide adequate treatment has been questioned. Even with the benefits of the treatment marshes, the alternative may not have a high likelihood of producing the consistently high quality effluent required to support the request for a dilution reduction. Additionally, land acquisition, even of this size, appears to be difficult. The project would also require extensive permitting for the excavation and construction of the treatment marsh, re-zoning of the property from Agriculture Exclusive, and mitigation due to the impact on existing wetlands. Additional permits include an Army Corps of Engineers Section 401 Permit, a RWQCB Section 404 Permit, a State Coastal Development Permit, as well as a County General Plan Amendment and Use Permit. The project may also face public opposition over odor and mosquito concerns. If this alternative is chosen as the preferred alternative, it is recommended that a pilot study be performed to determine the likely treated water quality.

Advantages include:

- Low technology
- Low operating costs
- Provides further treatment
- Increases storage capacity
- Enhances habitat

Disadvantages include:

- Requires large area of land
- Requires extensive permitting and re-zoning of land
- Cannot guarantee a specified level of treatment
- May face public opposition
- Possible impact on existing wetlands
- Construction of berms in 100-year Flood Plain

Figure 11: Alternative 2- Treatment Marsh

4.2.3 Storage Pond

The construction of a storage pond would allow the WWTF to provide enough capacity to retain excess wastewater during the discharge season. The discharge would be metered to comply with the 100:1 dilution requirement. No additional treatment would be included with this alternative and the effluent would still be monitored at the current outfall point of the WWTF before it enters the storage pond. The storage pond would be located in close proximity to the WWTF, although an exact location has not yet been determined.

A preliminary hydraulic analysis was performed to calculate the required capacity of a storage pond to release effluent at the 100:1 dilution. The analysis used influent data from the water year of 1999 to determine the water balance and estimate the required volume of the storage pond. The analysis was preliminary and a comprehensive water balance from several water years must be analyzed before final sizing of the storage pond could be calculated. The results of the analysis indicated that a volume of approximately 710,000 cy would be required. Initial sizing of an 11-foot deep pond would require an area of approximately 40 acres. The design of the pond includes the construction of a berm above the flood elevation to prevent overtopping in a storm event. Construction in the flood plain will result in displacement of flood waters to adjacent properties and may require mitigation.

The berms surrounding the pond would be ten feet wide at the top to allow access for service vehicles and would be sloped at 3:1 to allow for berm stability. The berms would be lined with a synthetic fabric to stabilize and limit the movement of fine sediment and water from penetrating the berm.

Permits include an Army Corps of Engineers Section 401 Permit, a RWQCB Section 404 Permit, a State Coastal Development Permit, as well as a County re-zone, General Plan Amendment and Use Permit. Since the outfall point would not change, a new NPDES permit would not be required.

Advantages include:

- The City would be in compliance with its discharge dilution requirement
- No additional technologies would be incorporated into the facility
- Minimal increase in operation and maintenance costs

Disadvantages include:

- Large footprint
- Moderate capital costs associated with land acquisition and construction
- May face public opposition
- Does not provide additional treatment
- Construction of berm within a 100-year Flood Plain
- Property would have to be re-zoned

4.2.4 Tertiary Treatment of Existing Secondary Effluent

This alternative consists of adding advanced treatment onto the existing pond treatment processes with the goal of creating an effluent that could be discharged to Francis Creek at a 3:1 dilution rate (Figure 12). The secondary effluent from the stabilization pond would undergo advanced treatment using pre-treatment processes followed by effluent filtration. Several filtration methods are considered including:

- sand filters;
- micro-filtration; and
- disc filters or fabric filters.

Lagoon effluent is generally more difficult to filter than other types of secondary effluent due to seasonally high levels of suspended solids. This variability in lagoon effluent quality requires that any final filtration process be supplemented with an upstream pre-treatment process. Two pre-treatment processes that have demonstrated long-term, reliable removal of the periodically high suspended solids content in lagoon effluent are polymer addition followed by a dissolved air floatation thickener (DAF), and flocculent addition followed by coagulation and sedimentation. Each process is described below after a discussion of design criteria.

4.2.4.1 Pre-Treatment Options

Advanced treatment processes are sized based on the historical maximum month flow, which is approximately 2.0 MGD (see Table 3). The lagoon would need to be drawn down at the end of the irrigation season and operated throughout the season such that there would be 7 million gallons (MG) of surge storage volume available to provide a buffer during peak day and peak week events. This storage buffer would allow the filtration system to be smaller than would be required to handle peak day flows. The advanced filtration process would likely operate at a continuous rate during the wet season.

Elevated suspended solids in the lagoon effluent are due to algal growth and/or solids washout during high flows. Based on effluent records from the Ferndale facility, the pre-treatment process should be designed for secondary suspended solids of up to 80 mg/l with a high proportion of algae cells. The pre-treatment filter effluent would contain less than 20 mg/l of suspended solids.

The upgraded plant would also need to include processes for dewatering and disposing of the solids removed during the pre-treatment process because algae should not be recycled to the lagoon. Dewatering of the solids from a floatation thickener or a chemical sludge from a sedimentation basin could be accomplished with a drying bed, but because available space is limited, a mechanical dewatering process may need to be investigated.

4.2.4.1.1 Dissolved Air Floatation

A dissolved air floatation thickener (DAF) can be used to effectively remove suspended solids and algae. Separation of the solids is brought about by introducing fine gas bubbles into the liquid phase. The bubbles attach to the particulate matter and carry them to the surface where they can be removed by a skimmer. Air is injected into the process while the liquid is under pressure. A portion of the influent flow is pressurized with a pump, and air is added at the pump suction. The liquid is held under pressure for several minutes, allowing time for the air to dissolve into the solution. The air comes out of solution and adheres to suspended solids creating a buoyant floc after the pressurized flow is released to the floatation tank.

Flocculant aids such as ferric chloride, alum, or various polymers are used to enhance the floatation process. Jar tests to determine required chemical dosages must be performed on a regular basis because chemical dosages will vary seasonally based on changes in algae populations and secondary loading characteristics. Air application rates will also be affected by changes in secondary effluent and the required operational changes must be determined by trial and error. It should be anticipated that the types of algal populations present in the lagoon will vary seasonally, requiring seasonal process adjustments. Long-term operational experience would ultimately allow prediction of changes in the algal community and optimization of the process and chemical dosing rates.

The wastewater treatment facility in Sunnyvale California uses a DAF thickener to treat secondary pond effluent prior to dual media filters. The combined facility successfully meets effluent permit turbidity limits for either 2 NTU or 10 NTU depending on seasonal discharge requirements. The effluent quality produced by the DAF units is controlled by increased application of cationic polymer. According to the operations director, annual chemical costs are $200/MG of wastewater treated per day. Based on average influent flow to the Ferndale plant, this would translate to an annual cost of $36,500.

If a DAF system is used for pre-treatment preceding a sand filtration system, additional provisions must be made for the addition of a coagulant preceding filtration. Gravity filters must be provided with an influent clearwell, or channel, which is designed for coagulant addition.

4.2.4.1.2 Coagulation/Flocculation/Sedimentation

Chemical coagulants, followed by flocculation and sedimentation have been used to successfully remove algae from pond effluent. Data in the literature indicates that high doses of alum are required to achieve removal rates of 80% and achieve effluent TSS concentrations of less than 20-30 mg/l.

Recommended overflow rates for a sedimentation basin are 0.2 to 0.8 gpm/SF. A 2 MGD process would require an estimated 94-foot diameter clarifier. A flocculation basin would be required, and solids handling should also be included in the process improvements.

4.2.4.2 Advanced Treatment Filtration

Advanced treatment filtration systems that are capable of producing effluent that meets requirements for reclaimed water, and are considered appropriate for upgrading treatment at the existing wastewater treatment facility, are described in this section**.** The preferred alternative for filtration will ultimately be evaluated based on its reliability and cost after being combined with pre-treatment process costs. A pre-treatment process is needed upstream of all of the effluent filtration systems discussed below.

4.2.4.2.1 Continuously Backwashing Sand Filter

Continuously backwashing filters are deep bed, granular media filters available in gravity or upflow configurations with a single type of sand or media consisting of layers of coarse, intermediate, and fine sand. The filters have in common a continuous backwash process whereby the filter media is washed by recycling the sand through an airlift pipe and sand washer. Compressed air is used to supplement the filtrate backwash flow, and serves to suspend and scour the filter media. Continuous backwash filters are also called "continuous contact" filters because flocculation and separation take place within the sand bed.

Continuously backwashing sand filters can be expected to achieve an effluent TSS quality of 5-10 mg/l, when following a DAF unit.

4.2.4.2.2 Microfiltration

The process of microfiltration (MF) is based on separation across a semipermeable membrane. Membranes are constructed with numerous hollow fibers and, during normal operation, feed passes through from the outside of the membrane into the center of the fibers and exits as filtrate. Any particles larger than a pore size range between 0.1 µm to 5 µm will be effectively trapped on the outside of the membrane, including suspended solids, fecal bacteria, viruses, and algae cells.

MF systems are capable of meeting reclaimed water requirements for tertiary filtration, which state that effluent quality must average 0.2 NTU. MF units reliably achieve this quality because of the limited pore size in the semi-permeable membrane. Two types of microfiltration systems are used for reclaimed wastewater:

- Pressurized Membrane Filter-Bundled fibers in a contained system with typical feed pressures of 10-50 psi.
- Submerged Membrane-Vacuum pump draws water through the membrane fibers within modules submerged in open concrete tanks.

The pressurized membrane system is available in skid-mounted units, and is more commonly used for the range of treatment plant flows at the Ferndale facility than the submerged membrane systems.

4.2.4.2.3 Disc Filter

Solids are separated from liquids by filter fabric stretched over disc shaped frames in disc filters. Disc filters have not been approved for tertiary treatment, but have been used to pre-filter lagoon effluent. The purpose of the filtration step is to produce finished water. Because disc filters have not been proven reliable in meeting the anticipated required effluent quality, they are not considered further in this analysis of alternatives.

4.2.4.3 Ammonia Stripping

Most of the nitrogen removal in secondary stabilization lagoons is due to volatilization of ammonia gas (NH_3) and nitrification. Both mechanisms are a function of detention time and temperature in the lagoon. Although lagoon processes may significantly reduce ammonia during the summer months (estimated as 50% reduction), when detention times are long and temperatures are high, lagoons are not efficient at removing ammonia during the colder winter months. During the cooler months, effluent ammonia concentrations from the existing WWTP are typically not much less than the influent ammonia concentrations.

Because biological nitrification is not occurring to any great extent in the lagoon, and the filtration process discussed above will not remove ammonia, an ammonia removal process would be required. One way to reduce ammonia levels in filtered effluent is to construct air-stripping towers following tertiary filtration. In order to strip ammonia from the wastewater, the pH in the flow must be raised to 10.5–11.5. The higher pH levels shift the ammonia/ammonium equilibrium toward the gas phase, allowing sufficient gas-water contact to volatilize the ammonia from solution. This is accomplished in vertical towers where large quantities of air are circulated through water droplets.

The amount of air required to strip out ammonia can be predicted using equilibrium relationships between the dissolved and gas phase of ammonia expressed by Henry's Law constants. However design flows are typically twice that of theoretical flows (EPA, 1993). In addition, ammonia removal efficiencies decrease with decreasing temperatures.

Design considerations include lime coagulation and settling to increase the pH prior to entering the air stripper. The acceptable hydraulic loading for the air stripper is approximately 2 gpm/SF (WPCF, 1982). Excessive hydraulic loading would result in sheeting of water, which in turn would reduce the efficiency of the ammonia removal process.

Air requirements for nitrogen removal are large. Typically 300 to 500 cubic feet per minute of air per gallon of water per minute (cfm/gpm) are required for an air-stripping tower (WPCF, 1982). Once the ammonia has been removed from the wastewater stream, the water effluent from the air stripper requires additional treatment to lower the pH. This is typically done with carbonation (diffusing carbon dioxide through the air stripper effluent).

The existing ammonia concentrations from the pond effluent range from 2 mg/L to 10 mg/L, and the anticipated maximum WWTP design flow is 2 MGD (~1,400 gpm). At this flow rate, the air

stripper (or combination of strippers) would require a cross sectional surface area of 700 SF, with a process blower capable of providing an airflow of 420,000 cfm to 700,000 cfm.

Anticipated maintenance includes periodic removal of scale buildup on the tower media as a result of calcium carbonate ($CaCO_3$) precipitating out of the wastewater stream. A significant consideration in the application of cooling towers is the choice of tower media. Plastic media are commonly used due to the high pH involved.

In summary, an ammonia removal system requires the following process steps:

- Lime coagulation and settling
- Air stripping towers, and
- Carbonation

Estimated costs for such a system are anticipated to range from 1.3 to 2.0 million dollars. Ammonia removal by air stripping can achieve up to 90% removal of ammonia nitrogen. However the EPA states that the place for ammonia air stripping appears to be in warm climate ponds (EPA, 1993). They reported a 50% removal of ammonia nitrogen in unaerated ponds within a 5-day detention time, at a pH of 10.5. Aerated ponds are capable of obtaining a similar removal in approximately a half day.

4.2.4.4 Summary

Successful advanced filtration of the existing lagoon effluent would require pre-treatment because of high TSS concentrations resulting from the presence of algae and other sources of suspended solids. The DAF system is expected to produce the higher quality effluent and provide the more reliable process between the two pre-filtration treatment alternatives.

Both pre-treatment process alternatives would be operator intensive, requiring frequent process and chemical adjustments, and having relatively high operating costs due to chemical addition. Both pre-treatment systems create solids that should be removed from the treatment stream because the return of such solids, especially algae cells, can negatively affect the lagoon treatment process. Because there is not room at the plant site for a facultative sludge lagoon and drying beds are ineffective on the north coast, a mechanical solids handling system would be needed.

DAF is the recommended method of advanced pre-treatment for sand filters or microfiltration when there is the potential for high levels of algae in the secondary effluent, and is therefore the preferred option for pre-treatment if advanced filtration is the selected process. Continuously backwashing sand filters are the most cost-effective of the filtration options discussed and are considered the preferred option for advanced filtration.

Advantages of the advanced treatment of lagoon effluent alternative include:

- Uses existing lagoon process as secondary treatment without major capital upgrades to the existing facilities
- Existing lagoon basin provides some surge capacity
- Effluent filter or membrane options are capable of achieving quality suitable for reclaimed water, assuming they are preceded by the DAF.

Disadvantages of the advanced treatment of lagoon effluent alternative include:

- Secondary effluent requires pre-treatment prior to final filtration.
- Process reliability continues to be subject to seasonal changes.
- High chemical costs for polymer and flocculant are expected.
- Possible necessity to pre-treat final filters with chlorine.
- Ammonia removal with air stripping adds significantly to operational complexity.
- Air stripping requires chemical addition to elevate for pH, which translates into significant operations and maintenance concerns.
- Solids handling processes are required for solids from pre-treatment processes and filter backwashes.
- There are no upgrades to the existing headworks and influent pumps in this alternative.
- The resulting treatment system would be highly Operator intensive.

Figure 12: Alternative 4- Tertiary Treatment of Lagoon Effluent

4.2.5 Sequencing Batch Reactors (SBRs)

This alternative would replace the existing aerated lagoon and polishing pond with a sequencing batch reactor (SBR) treatment system. A sequencing batch reactor is a fill-and-draw activated sludge treatment system in which aeration and sedimentation are carried out sequentially in the same vessel. An SBR should provide improved treatment with a high degree of reliability. The technology has proven it can provide good treatment performance with a high likelihood of trouble free operation when compared to other types of secondary treatment methods.

4.2.5.1 Modifications Required

The SBRs have a small footprint and could be installed at the western end of the plant site, where the existing polishing basin is located. Figure 13 shows the proposed layout and includes the installation of new headworks. Replacing the stabilization lagoon with SBRs would make the area of the lagoon available for a surge basin, facultative sludge lagoon (FSL), and effluent storage for irrigation uses, if needed.

4.2.5.2 Surge Basin

Equalization storage volume created within a portion of the existing lagoon would allow for efficient and cost effective design of the treatment processes. The secondary processes would be designed for the historical maximum month wet weather flow, meaning the surge basin would need to have enough capacity to store surges from the maximum week wet weather flow. The required surge capacity is estimated to be 7 MG (21.5 acre feet).

The surge basin would be drained regularly during the winter months to maintain storage capacity for the next large storm event, and would be completely empty during the summer months. The basin would need to be graded and a gravity line installed allowing wastewater to drain back to the headworks whenever treatment capacity is available. The surge basin would be lined with a synthetic material to minimize cleaning requirements and segregate underlying soil material from wastewater and settled organic solids.

4.2.5.3 Aerobic Digestion

Bio-solids from the SBRs as well as backwash water from the effluent filters would be pumped to one of two aerobic digesters for further solids processing. The digesters would be constructed in the same area as the SBR vessels, as shown in Figure 13. Two digesters are shown to allow for system backup during maintenance and repair, and to ensure adequate solids processing prior to transfer to the facultative sludge lagoon.

4.2.5.4 Facultative Sludge Lagoon (FSL)

A facultative sludge lagoon would be constructed within the westerly portion of the existing lagoon, as shown in Figure 13. Digested bio-solids from the aerobic digester would be pumped to this lagoon and stored for the long term. There is enough room for a lagoon to be sized to store solids for as long as 20 years, and the FSL would provide additional stabilization of the solids. Stored solids are always covered with a clearwater cap to prevent odors from escaping the FSL surface.

4.2.5.5 Transitional Storage

There would be remaining space available in the existing lagoon for treated effluent storage during periods of low surface water flow, and to supplement the supply of irrigation water in the dry months. This storage volume could be useful, for example, during the transition from the irrigation season to the wet weather discharge season in the fall of each year, and again in the late spring during the transition from wet weather discharge to disposal by irrigation. During the summertime, the reservoir would help to regulate the duration and frequency of spray irrigation on agricultural fields.

4.2.5.6 SBR Process

The key element in the SBR process is that the reactor provides both aerator and clarifier tank volumes. Typical SBR operation involves filling a tank with raw wastewater, aerating the wastewater to convert the organics into microbial biomass, providing a period for solids settling, and then discharging the SBR effluent. Two tanks provide continuous treatment; one tank is filling while the other goes through the treatment cycle.

The SBR treatment process is capable of providing BOD, suspended solids, and nutrient removal. The process has demonstrated the ability to consistently provide effluent quality of less than 5 mg/l BOD, less than 5 mg/l TSS, and less than 1 mg/l ammonia (NH_3). Levels at and below 10 mg/l of BOD and TSS each, are typically prescribed in SBR permits.

Aerobic digestion of solids can be added in a third SBR cell, which can be divided into two redundant digester cells. Because there is room to construct an oversized FSL, the aerobic digesters can be sized for shorter detention times than would otherwise be possible.

4.2.5.7 Filtration Process

Sequencing batch reactors are capable of achieving consistently high effluent quality without filtration. However, the City may choose to provide filtration if the RWQCB requires Title 22 effluent. Alternatives that include filtration of SBR effluent would, however, provide a higher quality effluent than the advanced filtration methods described previously.

If filtration of SBR effluent is required, then upflow, continuous backwashing sand filters could be sized using a 5 gpm/SF loading rate. The filters should be designed with the ability to add chemicals to meet Title 22 requirements, but the required chemical dosages are expected to be very low if they are necessary at all.

Membrane filters could also be utilized to polish SBR effluent. Membrane filters are the most reliable tertiary filtration system for producing quality effluent because a semi-permeable membrane with limited pore size filters the effluent. The use of membrane filtration following secondary treatment with SBRs would also eliminate the need for any chemical coagulants. Depending on RWQCB requirements, if filtration of the SBR effluent is necessary, then a cost-benefit analysis comparing upflow continuous backwashing filtration with membrane filtration should be performed.

4.2.5.8 Disinfection

One of the advantages of replacing the stabilization lagoon with an SBR system is the ability to reliably produce high quality effluent, which could be disinfected with an ultraviolet radiation (UV) system. UV systems use power and light instead of oxidative chemicals to disinfect wastewater. The effectiveness of the radiation is a direct function of the quantity of energy, or dose, which is absorbed by the organism. Given the high quality of the effluent from the SBR treatment processes discussed, there would be relatively little interference to the radiation, thereby reducing the UV intensity necessary to achieve the desired level of disinfection.

The advantages of a UV system include the following:

- Simplicity
- Small footprint
- No adverse environmental effects
- Dechlorination is not necessary

4.2.5.9 Summary of SBR Alternative

SBRs are a proven technology that can be employed to reliably achieve a high quality secondary effluent. Reliability, small footprint, and maximizing use of the existing lagoon provide the most important advantages of this alternative.

If tertiary filtration is required to meet Title 22, the filtration processes could be installed at a lower cost than for the other systems considered (i.e. pre-treatment of lagoon effluent). Chemical dosages should be a fraction of the chemical dosages required for lagoon effluent pre-filtration treatment systems. No chemicals should be required for membrane filtration behind the SBR process.

Advantages include:

- Reliable high quality effluent
- Ammonia and nitrate removal
- Straightforward, relatively simple operation
- Small footprint for reactors
- Surge basin to limit peak flows and reduce cost of treatment
- Limited or no chemical use
- Long filter runs between backwash (if filtration is necessary)
- FSL providing long term solids handling and storage

Disadvantages include:

- Secondary process with associated increased complexity of operation
- Solids processing needed

Figure 13: Alternative 5- Sequencing Batch Reactors

4.2.6 Membrane Bioreactors

Membrane bioreactors (MBRs) combine membrane technology with the activated sludge process to provide secondary and tertiary treatment in the same reactor vessel. With this treatment technology, microfiltration modules replace the clarification step with a membrane sheet providing liquid-solid separation. Suspended solids can be removed completely producing very high quality (almost bacteria-free) treated water.

In recent years membrane bioreactors have become more cost effective, and when tertiary treatment is included, the cost of a complete membrane bioreactor may be comparable to the cost of secondary/tertiary treatment processes such as an SBR process followed by microfiltration. This is especially true if the cost of upgrading the disinfection system is included. Membrane bioreactors can provide a reduction of up to 90% in the required disinfection capacity.

4.2.6.1 Process Description

Membrane bioreactors utilize a hollow fiber, ultra filtration membrane immersed within an activated sludge tank with very high mixed liquor. With a pore size less than 0.1 µm the membrane is a complete physical barrier to the mixed liquor solids, bacteria, and most viruses. A vacuum varying between 2 and 9 psi is applied to a head connecting the membrane modules through the use of a centrifugal pump. The treated water is drawn through the hollow fibers and pumped out as high quality effluent. Air in the aeration basin scours the membrane and keeps it free from fouling.

Liquid is periodically pumped back through the membrane in a pulse which, coupled with a membrane air scour system, cleans the membrane by forcing solids away from it. Other components of the treatment system include: pumps for inducing the vacuum, mixed liquor recycle pumps, membrane air scour blowers, chemical feed system for membrane cleaning, and aeration system. Many of the treatment components are similar to those contained in an activated sludge system, although there is no need for a return activated sludge pumping system because there is no clarifier. Mixed liquor recycle pumps keep the reactor solids mixed, and solids are wasted from the recycle stream.

Because of the high mixed liquor and long sludge age of more than 30 days, there is no need for a separate aerobic digester to further aerate the bio-solids. A facultative sludge lagoon would be provided for long-term storage of solids that are wasted from the reactor.

4.2.6.2 Summary

Membrane bioreactors (MBRs) are essentially a clarifier and filter in an activated sludge process containing very high mixed liquor concentrations. Because of the high sludge concentration and reactor capacity (up to 40 times higher than in conventional treatment) the aeration volume is significantly reduced, allowing the entire process to have a small footprint (Figure 14).

Like the SBR process, the need for a secondary clarifier is eliminated. Aeration and clarification can be carried out in a single reactor with the membranes immersed in the aerration basin or alternatively in a separate module following aeration. However unlike an SBR, the ultra filtration membranes in the MBRs provide tertiary filtration. The process can be compared to microfiltration used as a method of tertiary treatment. Performance is considered highly reliable because of the physical barrier provided by the membrane, which in the case of membrane bioreactors is an "ultra-filtration" membrane.

Advantages include:

- Consistently high effluent quality
- Small footprint
- Eliminates or significantly reduces need for disinfection system
- Reduced sludge volumes
- Would not require solids processing line

Disadvantages include:

- New technology with which most operators lack familiarity
- Some operational complexity
- Frequent replacement of membranes required
- High cost
- No long term operational data available (technology less than 5 years)

Figure 14: Alternative 6- Membrane Bioreactor

4.2.7 No Action

The No Action Alternative would cause the City to remain in noncompliance with its NPDES permit. The City's WWTF would continue its violations in effluent quality and dilution. This alternative would not incur additional costs to the City, except for fines placed on the City for its discharge violations.

Advantages include:

- No additional capital costs for the City
- No need for additional land acquisition

Disadvantages include:

- City would remain in violation
- City would continue to be fined for discharge violations
- The State could require the City to limit additional connections to the wastewater system

4.3 Disinfection Process Alternatives

There are several reasons why the City should consider options for improving the disinfection system as part of this project. If improved treatment with sequencing batch reactors is the preferred option, construction would require modification of the existing chlorine basin as was indicated in the layout for this option. Also, one of the advantages of producing higher quality effluent is that other options for disinfection, such as ultraviolet radiation, become feasible.

The WWTF currently uses gaseous chlorine to disinfect treated wastewater prior to discharge to Francis Creek. Gaseous chlorine use raises significant safety concerns with handling and the increased cost of managing a gas chlorine system due to regulatory requirements.[1] This section presents and evaluates three disinfection alternatives for replacing the gas chlorine system, including:

1. Liquid Hypochlorite
2. On-site Hypochlorite Generation, and
3. Ultraviolet Radiation (UV).

These systems are not considered a hazard to public safety and are not subject to the same requirements for accident prevention, risk assessment and monitoring as needed for chlorine gas.

[1] In response, the USEPA prepared risk management program (RMP) regulations that were published as a final RMP rule on June 20, 1996. Under the RMP rule, wastewater facilities that handle greater than 2500 pounds of chlorine must develop a risk management program including a hazard assessment, prevention program, and emergency response plan

4.3.1 Ultraviolet Disinfection

UV systems use power and light instead of oxidative chemicals to disinfect wastewater. UV lamps emit a wavelength of energy that penetrates cell walls and destroys genetic material in bacteria and viruses. The effectiveness of the radiation is a direct function of the quantity of energy, or dose, that is absorbed by the organism.

The advantages of a UV system include:

- Simplicity
- Small footprint
- No adverse environmental impacts
- Dechlorination is not necessary

Disadvantages include:

- Possible degradation of effluent upon exposure to sunlight this is minimized by the application of high dosages
- Effectiveness dependent on the transmissivity of the effluent, which is dependent on effluent water quality parameters
- Need for backup power generation equipment.

4.3.1.1 Technical Requirements

Dosage is the main concern for a UV system and is a function of lamp intensity and transmissivity. Transmissivity is a measure of the ability of UV light to pass through the wastewater, and is affected by water quality parameters such as suspended solids, turbidity, iron, hardness, and the amount of humic material. A decrease in water quality would reduce the applied UV dose and lower the performance results of the system. Consequently, upstream treatment process reliability is extremely important in making the decision to choose a UV disinfection system.

The primary source of UV radiation in the wastewater market is the low-pressure mercury arc lamp because 85% of its output is at the wavelength of 253.4 nm, which is within the optimum wavelength range for germicidal inactivation. Leading suppliers of UV systems recommend high intensity low-pressure systems for small facilities such as the City of Ferndale.

4.3.2 Bulk Hypochlorite

In recent years many wastewater treatment plants have been retrofitted or designed using sodium hypochlorite solution (NaOCl) for disinfection. The selection of hypochlorite is primarily based on eliminating the risks associated with the storage and use of chlorine gas. If appropriate design

practices are employed in design of the system, hypochlorite can be an efficient and effective means of disinfection which has widespread application.

Advantages include:

- Ease of Operation and Control
- Economical
- Safe

Disadvantages include:

- Need for Dechlorination
- Corrosive
- Off-Gassing
- Degradation

4.3.2.1 Technical Requirements

Tanks should be sized for a minimum of two weeks demand during periods of maximum use, and take into account the potential for degradation if hypochlorite that is stored for more than one month. Tanks can be constructed of linear high-density polyethylene or fiberglass reinforced plastic. Fiberglass has a longer life if well specified and properly constructed.

Hypochlorite, with its high pH and chlorine content, is an aggressive chemical that will exploit any weak spot in a piping system. Each joint must be glued according to manufacturer's instructions if PVC pipe is used. Flexible braided tubing may be used in place of larger pipe for secondary containment. The smallest possible line should be used in design to increase velocity and prevent off-gassing and calcification. Usually the neat hypo is diluted when it must be pumped any distance.

Metering pumps should be positive displacement pumps such as diaphragm or peristaltic pumps. The pumps deliver hypochlorite to the chlorine injection point and must be mixed with the effluent upstream of the contact basin.

4.3.3 Onsite Generation of Hypochlorite

Onsite sodium hypochlorite systems utilize electrolysis to generate a 0.8% hypochlorite solution by combining salt, water and electricity. The process starts by dissolving salt in softened water, and passing the brine solution through an electrolytic cell where a low voltage DC current is supplied. Typical concentration for onsite generated hypochlorite ion is 0.8%.

A consistent supply of chemical solution can be maintained without the necessity of large storage tanks by producing hypochlorite onsite. Because several days' supply is stored, an immediate source of backup power is not as critical as it would be for an ultraviolet disinfection system.

Advantages include:

- Consistency of Supply
- Ease of Operation and Control
- Economical
- Safe
- No need for large storage tanks

Disadvantages include:

- Need for Dechlorination
- Requires 3 phase power

4.3.3.1 Technical Requirements

The technical requirements for onsite generation of hypochlorite are similar to those for bulk hypochlorite except for the advantages resulting from the chlorine being supplied in much lower concentrations. The more dilute solution is less corrosive and less susceptible to scaling or off-gassing.

4.4 Influent Pump Station and New Headworks

The existing pump station structure is in poor condition and can be considered to have outlived its useful design life. Influent is pumped to the primary oxidation ponds with two 25 Hp pumps which frequently require repair. There is currently no pre-screening or grit removal at the existing headworks as the obsolete comminutor (meant to grind up solid material prior to pumping) has not been functional for several years. This section discusses the need for a new influent pumping station and presents alternatives for influent pumping and pre-treatment systems.

4.4.1 Need for New Pump Station

Upgrades to the existing pump station were investigated but are considered infeasible or undesirable for a number of reasons including the following:

- Age and condition of the existing structure and equipment
- Depth of sewer line making installation of pre-treatment processes ahead of the wet well and pumps impractical and resulting in a “confined space” situation that raises significant safety concerns
- Constructability difficulties in keeping the existing pump station in service while upgrades are under construction
- Flooding susceptibility at the site during extreme wet weather events.

For these reasons, a new pump station is recommended. The new influent pump station would be sized to lift wastewater up to above ground headworks for pre-screening and grit removal. The headworks would have sufficient elevation head for the pre-treated influent to flow by gravity to secondary treatment processes. The existing pump station could be left on line while the new facility is under construction. The location of the new influent pump station and headworks would depend on the selected treatment alternative. The preferred location is discussed in more detail in Section 5.

The influent pump station should have a firm capacity equal to the PIF. Firm capacity is defined as the ability to deliver the required flow with one pump off line. This redundancy is necessary to prevent sewage overflows due to the failure of a single pump.

4.4.2 Pre-Screening

Pre-screening removes floatables, rags and other solid materials prior to secondary treatment. There are many designs for mechanical screens that remove solids from the waste stream, including vertical traveling bar screens, vertical traveling fine screens, inclined in-channel screens with screw conveyors, and inclined screens with auger monsters. The following types are most commonly used in facilities the size of the Ferndale WWTF.

- **Drum Screen**: The flow passes through a rotating drum screen and solid material collected on the inside of the screen is carried up an inclined screw or auger. The inclined channel or tube is lined with a somewhat coarser screen through which putrescible organics are washed back into the channel, while the remaining solids are lifted and dewatered for easy disposal.
- **Auger Monster**: This is a proprietary system consisting of a drum screen preceded by a grinder to facilitate the separation of biodegradable matter and coarse solids.
- **Traveling Vertical Screens**: These screens incorporate a series of perforated steel panels traveling on a vertical chain mechanism. The screened solids are dropped into a hopper and fed into a compacter.

These types of screens could be installed in an open channel approximately 1.5 to 2 feet wide. A sampling station would be set up and an automatic sampler installed for composite sampling of the influent wastewater to comply with permit monitoring requirements.

4.4.3 Grit Removal

Grit removal alternatives include aerated grit chambers and proprietary equipment such as the teacup or head cell. The teacup has a hydraulic vortex design that has a limited performance range and is inefficient when operated under varying flows. Grit removal in an aerated grit chamber would be an appropriate pre-treatment methodology as part of the mechanical process because process air would be provided as part of the design for the sequencing batch reactors. However, in addition to a requirement for process air, the disadvantages of aerated grit chambers

include a large surface area requirement, pumping requirements for concentrated grit slurries, and accumulation of floatables.

Based on advantages discussed in the following section the headcell is considered a good choice for grit removal at the Ferndale treatment facility.

4.4.3.1 Head Cell

The head cell is a modular, multi tray solids concentrator that removes grit down to 50 micron in size (200 mesh). Grit removal in the system is based on creating a vortexing action across multiple tray layers that serve as settling basins (similar to settling tubes in clarifiers). Solids caught in the vortex flow are swept down each tray to a central core where they are collected and continuously pumped to a grit washing device. The use of stacked multiple trays creates a large surface area that effectively captures grit in a relatively small footprint.

Advantages of the Head cell system include:

- Highest removal efficiencies among the alternatives considered
- Flexibility to increase capacity after installation
- Low head loss
- Small footprint
- Comparatively low equipment and construction costs

A head cell sized to give 95% grit removal at the PDAF of 3.85 MGD would be approximately 6 feet in diameter. The head cell would be installed in the center of a concrete tank and surrounded by a circular channel. The bottom and sidewalls of the tank are sloped down to allow grit to settle and collect in a bottom hopper. An overflow weir installed across the tank allows effluent to overtop the chamber and flow to the downstream splitter box.

4.4.3.2 Grit Dewatering

Settled grit would be dewatered and concentrated with a cyclonic, auger style classifier that is considered an efficient and cost-effective method for this function. The classifier receives grit from a grit pump that is typically located below the primary grit removal system. Cyclonic classifiers are typically capable of removing 90% of the grit sent to the process. Removed grit is stored in a dumpster onsite and typically disposed of along with screenings. The classifier would be sized for projected grit loadings.

4.5 Construction Costs and Economic Analysis

The estimated construction costs for the six treatment project alternatives are summarized in this section of the report. The estimates are preliminary and are based on the level and detail of planning presented in this report. As planning proceeds, and as site-specific information becomes available, the estimates may require updating. However the estimates can be used to provide a relative measure of the cost impacts of the alternatives considered in this study.

Estimated costs are based on actual construction bidding results from similar work, published cost guides, and other construction cost experience. Reference was made to the as-built drawings and system maps of the existing facilities to determine construction quantities, elevations of the reservoirs and major components, and locations of distribution lines. Where required, estimates are based on preliminary layouts of the proposed improvements.

4.5.1 Construction Costs

Tables 7-12 summarize the capital costs for the first five alternatives. The No Action alternative will not incur additional costs to the City. Operation and maintenance costs must also be considered in determining the apparent best economic alternative among the options.

All alternatives include the cost of a new pump station and headworks. Although these costs will be similar for all alternatives, there is some difference because the location of the new pump station will vary with the various alternatives. This is reflected in different construction cost for the sewer lines discharging to the new wet well.

UV is the preferred method of disinfection but is not suitable for all methods of treatment. A UV system was assumed where appropriate in estimating costs for improvements to the disinfection system, but in cases where the quality of the secondary effluent would not meet minimum requirements for UV disinfection, costs are based on onsite generation of dilute hypochlorite.

Table 7: Construction Cost Estimate for Alternative 1-Pumping to the Eel River

ITEM	UNIT	QUANTITY	UNIT COST	TOTAL COST
Construction	LS	1	$1,689,000	$1,689,000
Additional Mobilization	LS	1	$50,000	$50,000
Control Building	LS	1	$50,000	$50,000
Influent Pump Station	LS	1	$282,500	$282,500
Headworks	LS	1	$365,750	$365,750
Onsite-Chlorine Generation	LS	1	$355,000	$355,000
Total Estimated Construction Cost				**$2,792,250**
Construction Contingency		15%		$418,838
Planning and Pre-Design	LS	1	$50,000	$50,000
Engineering & Surveys		20%		$558,450
Geotechnical	LS	1	$35,000	$35,000
Environmental Document & Permitting	LS	1	$45,000	$45,000
Legal & Administration		1%		$27,923
TOTAL ESTIMATED PROJECT COST				**$3,927,000**

Table 8: Construction Cost Estimate for Alternative 2-Treatment Marsh

ITEM	UNIT	QUANTITY	UNIT COST	TOTAL COST
Construction	LS	1	$970,000	$970,000
Additional Mobilization	LS	1	$50,000	$50,000
Control Building	LS	1	$50,000	$50,000
Influent Pump Station	LS	1	$282,500	$282,500
Headworks	LS	1	$365,750	$365,750
Onsite-Chlorine Generation	LS	1	$355,000	$355,000
Total Estimated Construction Cost				**$2,073,250**
Construction Contingency		15%		$310,988
Planning and Pre-Design	LS	1	$50,000	$50,000
Engineering & Surveys		20%		$414,650
Geotechnical	LS	1	$35,000	$35,000
Environmental Document & Permitting	LS	1	$45,000	$45,000
Legal & Administration		1%		$20,733
TOTAL ESTIMATED PROJECT COST				**$2,950,000**

*THIS COST ESTIMATE DOES NOT INCLUDE LAND ACQUISITION (9 ACRES~$90,000)

Table 9: Construction Cost Estimate for Alternative 3-Storage Pond

ITEM	UNIT	QUANTITY	UNIT COST	TOTAL COST
Construction	LS	1	$1,173,000	$1,173,000
Additional Mobilization	LS	1	$50,000	$50,000
Control Building	LS	1	$50,000	$50,000
Influent Pump Station	LS	1	$282,500	$282,500
Headworks	LS	1	$365,750	$365,750
Onsite-Chlorine Generation	LS	1	$355,000	$355,000
Total Estimated Construction Cost				**$2,276,250**
Construction Contingency		15%		$341,438
Planning and Pre-Design	LS	1	$50,000	$50,000
Engineering & Surveys		20%		$455,250
Geotechnical	LS	1	$35,000	$35,000
Environmental Document & Permitting	LS	1	$45,000	$45,000
Legal & Administration		1%		$22,763
TOTAL ESTIMATED PROJECT COST				**$3,226,000**

*THIS COST ESTIMATE DOES NOT INCLUDE LAND ACQUISITION (40 ACRES~$300,000)

Table 10: Construction Cost Estimate for Alternative 4-Tertiary Treatment of Lagoon Effluent

ITEM	UNIT	QUANTITY	UNIT COST	TOTAL COST
Mobilization	LS	1	$500,000	$500,000
Control Building	LS	1	$50,000	$50,000
Influent PS	LS	1	$282,500	$282,500
Headworks	LS	1	$365,750	$365,750
DAF	LS	1	$864,000	$864,000
Sand Filters	LS	1	$434,500	$434,500
Tertiary Pumping	LS	1	$109,200	$109,200
Additional Construction Cost	LS	1	$158,400	$158,400
Air Stripping Towers	LS	1	$2,250,000	$2,250,000
On-site Hypo-Chloride Generation	LS	1	$355,000	$355,000
Total Estimated Construction Cost				**$5,369,350**
Construction Contingency		15%		$805,403
Planning and Pre-Design	LS	1	$50,000	$50,000
Engineering & Surveys		20%		$1,073,870
Geotechnical	LS	1	$35,000	$35,000
Environmental Document & Permitting	LS	1	$45,000	$45,000
Legal & Administration		1%		$53,694
TOTAL ESTIMATED PROJECT COST				**$7,432,000**

Table 11: Construction Cost Estimate for Alternative 5-Sequencing Batch Reactors

ITEM	UNIT	QUANTITY	UNIT COST	TOTAL COST
Mobilization	LS	1	$400,000	$400,000
Control Building	LS	1	$50,000	$50,000
Influent PS	LS	1	$292,500	$292,500
Headworks	LS	1	$365,750	$365,750
Influent Piping	LS	1	$113,000	$113,000
SBR Reactors, Equalization, Splitter	LS	1	$2,362,650	$2,362,650
Storage Ponds	LS	1	$522,000	$522,000
Additional Cost	LS	1	$180,000	$180,000
UV System	LS	1	$595,000	$595,000
Total Estimated Construction Cost				**$4,880,900**
Construction Contingency		15%		$732,135
Planning and Pre-Design	LS	1	$50,000	$50,000
Engineering & Surveys		20%		$976,180
Geotechnical	LS	1	$35,000	$35,000
Environmental Document & Permitting	LS	1	$45,000	$45,000
Legal & Administration		1%		$48,809
TOTAL ESTIMATED PROJECT COST				**$6,768,000**

Table 12: Construction Cost Estimate for Alternative 6-Membrane Bioreactor

ITEM	UNIT	QUANTITY	UNIT COST	TOTAL COST
Mobilization	LS	1	$450,000	$450,000
Control Building	LS	1	$50,000	$50,000
Influent PS	LS	1	$292,500	$292,500
Headworks	LS	1	$365,750	$365,750
Influent Piping	LS	1	$113,000	$113,000
Membrane Bioreactor	LS	1	$3,593,775	$3,593,775
Storage Ponds	LS	1	$561,223	$561,223
Additional Cost	LS	1	$185,000	$185,000
UV System Cost	LS	1	$595,000	$59,5000
Total Estimated Construction Cost				**$6,206,248**
Construction Contingency		15%		$930,937
Planning and Pre-Design	LS	1	$50,000	$50,000
Engineering & Surveys		20%		$1,241,250
Geotechnical	LS	1	$35,000	$35,000
Environmental Document & Permitting	LS	1	$45,000	$45,000
Legal & Administration		1%		$62,062
TOTAL ESTIMATED PROJECT COST				**$8,570,000**

4.5.2 Operating Costs

Operation and maintenance costs must also be considered in determining the apparent best economic alternative among the options. Table 13 summarizes the estimated annual operating costs for the alternatives.

Table 13: Estimated Annual Operation & Maintenance Costs

	Pump to Eel River	Treatment Marsh	Storage Pond	Tertiary Treatment	SBR	Membrane Bioreactor
Maintenance	$8,000	$30,000	$6,000	$14,000	$10,000	$8,000
Operation	$1,000	$2,000	$1,000	$4,000	$4,000	$4,000
Electrical						
Influent Pumping	-	-	-	$1,000	$4,100	$1,200
Vacuum Pumping	-	-	-	-	-	$10,000
Effluent Pumping	$20,000	$10,000	$6,000	-	-	-
Backwash Pumping	-	-	-	-	-	$5,000
Solids Handling	-	-	-	$5,000	$4,000	$1,000
Aeration	-	-	-	-	$14,000	$14,000
Air Stripping	-	-	-	$35,000	-	-
ESTIMATED TOTAL ANNUAL COST	**$29,000**	**$42,000**	**$13,000**	**$59,000**	**$36,000**	**$43,000**

4.6 Environmental & Permitting Issues

This review identifies the land use and environmental issues associated with the following alternatives for wastewater treatment at the Ferndale WWTF:

- Eel River Discharge
- Treatment Marsh
- Storage Pond
- Advanced Treatment of Secondary Stabilization Pond Effluent
- Sequencing Batch Reactor(s) (SBR) plus Advanced Treatment
- Membrane Bioreactor

4.6.1 Land Use and Zoning

The City of Ferndale owns two parcels within Humboldt County jurisdiction that house the headworks (APN 100-162-20); and the contact basin and ponds (APN 100-161-08). An adjacent parcel is used for spray irrigation of treated wastewater during summer months (APN 100-162-28). The following Humboldt County General Plan land use designations and zoning apply to these parcels:

- Headworks (APN 100-162-20) - General Plan: Public Facility (PF); zoning: PF
- Contact Basin and Ponds (APN 100-161-08) - General Plan: Public Facility (PF); zoning: PF
- Spray irrigation field (APN 100-162-28) - General Plan: Agricultural Exclusive (AE); zoning: AE.

The improvements associated with the above alternatives are concentrated in the Contact Basin and Pond area and are consistent with the General Plan PF designation and are an allowable use in the PF zone. No improvements or work is proposed for the agricultural irrigation field.

4.6.2 Coastal Zone

The contact basin and pond area is located within the Coastal Zone, under Humboldt County local jurisdiction. A coastal development permit (CDP) is required for any development within the coastal zone. "Development" means, on land, in or under water, the placement or erection of any solid material or structure; discharge or disposal of any dredged material or of any gaseous, liquid, solid, or thermal waste; grading, removing, dredging, mining or extraction of any materials; change in the density or intensity of use of land, including subdivisions; construction, reconstruction, demolition, or alteration of the size of any structure; and the removal or harvesting of major vegetation other than for agricultural purposes and timber operations which are in accordance with a timber harvest plan.

"Structure" includes, but is not limited to, any building, road, pipe, flume, conduit, siphon, aqueduct, telephone line, and electrical power transmission and distribution line.

4.6.3 Environmental Issues

A decision by the City Council to proceed with any of the above alternatives would constitute a "discretionary action" and be subject to environmental impact review per the California Environmental Quality Act (CEQA). Similarly, if the project were funded by a federal agency (e.g., FEMA), the project would be subject to environmental impact review per the National Environmental Policy Act (NEPA).

4.4.1.1 Water Quality

The primary environmental issue associated with the above alternatives pertains to the discharge of treated wastewater and its effects on water quality of the receiving waters.

The WWTF treats municipal wastewater from the City's 1,500 residents, outlying areas, and several commercial facilities. The effluent is treated through a series of processes and applied to neighboring farmlands as irrigation May 15 through September 30. During the remaining portion of the year, the effluent is discharged to its receiving body. Francis Creek is a primary contributor to flows in the Salt River, thus the Salt River is considered the receiving water body. The discharge is permitted by the RWQCB through the National Pollution Discharge Elimination System.

One of the primary goals of the advanced treatment alternatives is to produce a plant effluent that can be discharged to Francis Creek at a reduced dilution ratio without degrading the quality of water in the stream. The primary goal of pumping to the Eel River and the storage pond is to allow the WWTF to comply with its dilution requirement without incurring the cost of additional treatment processes.

As discussed in Section 4 of this report, dissolved oxygen (DO), temperature and ammonia were monitored upstream and downstream of the plant discharge point, and give an indication of the impact the existing plant effluent has on Francis Creek. The data indicates an average decrease in DO of 1.5 mg/l, an average increase in temperature of 0.1 degree, and an average increase in stream ammonia concentration of 1.5 mg/l, based on a comparison of the upstream data values to the downstream data values.

The decrease in dissolved oxygen indicates some degradation of stream quality, which should be remedied as part of the proposed treatment process. The impact of the effluent discharge on ammonia concentrations could also be significant and possibly result in an additional oxygen demand in the stream. Each of the advanced treatment alternatives is designed to reduce ammonia to 1 part per million to minimize the risk of negative effects from ammonia in the effluent.

Expected temperature increases are slight but must be considered in light of the stream's beneficial uses and seasonal fisheries value. The discharge of treated wastewater (and resulting possible increase in temperature of receiving waters) occurs during wet weather, and is further limited by the prohibition period from May 16 to September 30 of each year. Treatment plant flows increase during wet weather as the result of I/I (Infiltration and Inflow), and the ratio of wastewater to fresh water decreases. This phenomenon effectively lowers the average temperature of the wastewater in comparison to the dry months when treated effluent is spray irrigated on land. There may be time periods when the creek flows remain relatively low while the weather is wet and preventing disposal by irrigation. The effects of temperature could be more pronounced under such conditions.

Possible mitigation for temperature would be to enhance a reach of Francis Creek adjacent to the plant site by planting willows and/or other riparian, shade producing trees to reduce the ambient

temperature of that portion of the Creek. Planting within the riparian zone of Francis Creek would require consultation with the California Department of Fish and Game (DFG) and may require a Streambed Alteration Agreement with DFG. Francis Creek would be subject to the County's Streamside Management Area Ordinance (see Zone Regulations Section 314-61.1), but depending on what is actually proposed as far as tree planting goes, could be exempted. According to the Eel River Area Plan, Riparian Protection Measures are an allowable activity within the Streamside Management Zone (Michael Wheeler, HumCoPlan, 10/20/03).

Any work in the channel of Francis Creek below ordinary high water would require a permit from the Army Corps of Engineers.

4.4.1.2 Fisheries

Water quality and its effect on fish and fish habitat are not expected to be a significant issue. Francis Creek drains to the Salt River, a tributary of the Eel River. The Salt River has been seriously degraded due to sedimentation in the channel. Alternatives for restoring the Salt River are currently being pursued by the Army Corps of Engineers and the Humboldt County Resource Conservation District.

According to a study prepared by the U.S. Department of Agriculture (*Salt River Local Implementation Plan*, Humboldt County, California, U.S. Department of Agriculture, Soil Conservation Service, March 1993), sedimentation in the estuary of the Eel River adversely affects anadromous fish by filling pools, raising average temperature, lowering dissolved oxygen, and smothering food organisms. Partly as a result of sedimentation and non-point source pollution, the productivity of the Eel River (including the Salt River and Francis Creek) in terms of salmonids has declined considerably.

Other species that may have declined in abundance, or disappeared from the Eel River estuary and river, include Pacific sardines, green sturgeon, eulachon, cutthroat trout and tomcod. Cutthroat trout were found in all Salt River tributaries in 1950 but their absence from samples collected in 1974-75 could be associated with the decreased spawning and rearing habitat in the Salt River drainage (Higgins, 1991 in *Salt River Local Implementation Plan*, 1993).

Restoration of the Salt River will likely result in better conditions for anadromous fish as well as better water quality conditions of receiving waters for wastewater discharge purposes. Discharge of appropriately treated wastewater from the City’s treatment plant could, by increasing flow volumes, enhance the biological value of Francis Creek and the Salt River.

4.7 Preferred Project Alternative

Six alternatives have been investigated to improve the City of Ferndale's WWTF. Two major categories of solutions were investigated. The first type of solutions investigates options for the WWTF to discharge at its required dilution of 100:1, which the City is currently chronically violating. The second type of solution investigates treatment options for improving the WWTF effluent quality in conjunction with a request for a dilution reduction.

The two alternatives for complying with the dilution requirement are relatively low operator intensive and have moderate to low maintenance requirements. The largest source of operation and maintenance increases would be the additional pump stations included in the design to pump to the Eel River and the construction of the storage pond. These two alternatives provide relatively simple designs for establishing compliance with the dilution requirement, but are largely based on acquisition of private land and do not provide additional treatment of wastewater.

Four alternatives for upgrading treatment processes at the Ferndale WWTF are described. Of the four, the treatment marsh would provide the least costly and operator-intensive treatment. However, it would require the acquisition of nine acres and may not provide consistently high quality effluent which the other three alternatives can provide.

The tertiary treatment of the existing lagoon effluent would probably be the most operator-intensive because of the variability of lagoon effluent quality. Frequent adjustment to the pre-treatment process would be required. Because the ability of the tertiary filtration process to produce quality effluent depends upon successful pre-treatment, this alternative is considered the least reliable of the four alternatives, with the exception of the treatment marsh.

MBRs reliably produce a very high quality effluent because of the small pore size permeability of the membranes, have a small footprint, and require little or no disinfection. System complexity should however be taken into account in determining overall process reliability as should the possibility of potential fouling of the membranes and the need for maintenance with chemical backflushing. Long-term experience with MBRs is lacking, and the new technology is a high cost item.

The installation of SBRs to replace the secondary lagoon process assures a reliable, high quality, secondary effluent. SBRs are proven technology and have become easier to operate in recent years due to improvements to the mechanical and controls systems. The timed sequence of fill, aeration, and decant can be reliably automated thus simplifying operation. The overall consistency of the process makes operation of this system more straightforward than operation of the processes needed to provide advanced treatment of lagoon effluent.

The SBR process is also considered easier to operate and understand than the membrane bioreactor process. This is partially due to the fact that SBRs are based on the traditional activated sludge process that is familiar to most operators. By contrast the membrane bioreactor

is a sophisticated system that would require the operator to become familiar and skilled in a new technology.

A set of criteria were established to determine the preferred alternative. The criteria include capital costs, operation and maintenance costs, environmental review and permitting, funding agency acceptance, and public acceptance (Table 14). Each of the alternatives was compared based on the criteria, and from the criteria the preferred alternative was determined.

Because of the hydrologic and hydraulic conditions affecting Francis Creek, we do not believe the current discharge dilution requirement can be complied with. However, it is our opinion that a treatment system that would increase the quality of effluent to a level greater than the receiving body would justify a conditional dilution reduction. The installation of SBRs to replace the secondary lagoon process assures a reliable, high quality, secondary effluent and, of the six treatment alternatives, is considered the preferred option.

Table 14: Corrective Action Alternatives for Wastewater Treatment

5 PROPOSED PROJECT

5.1 General Project Description

The installation of sequencing batch reactors (SBRs) to replace the secondary lagoon process assures a reliable, high quality, secondary effluent. One of the main advantages of SBRs is a small footprint that allows the existing plant to continue processing community wastewater while the upgrades are under construction. The project would maximize use of the existing lagoon, which would be modified to provide three functions for the upgraded plant: 1) peak flow equalization storage, 2) a facultative sludge lagoon for solids storage and digestion, and 3) finished effluent storage for irrigation purposes.

The proposed layout is shown in Figure 15. Figure 15 includes two reactors with associated blower and pumping facilities, two aerobic digesters, and room for one additional SBR reactor in the future. All new headworks treatment processes would be located above the blower room and the influent pump station. An influent screen would be needed to preclude inorganic material from the SBRs, and a grit removal process would be located downstream of the screens. Influent pumps would be sized to boost the wastewater the lift needed to reach the headworks processes and the splitter box for distribution to secondary treatment. The splitter box would control the direction of flow depending on the SBR operation cycle and whether there is a need to route excess flow to the surge basin.

Effluent filters are also shown on the proposed layout, and are considered a potential construction item depending on system performance and RWQCB requirements. It is recommended that the project be based initially on improving secondary treatment through the addition of SBRs <u>without</u> effluent filtration. Effluent filters could be added if required based on treatment plant performance and water quality parameters in the receiving stream at the point of discharge.

Other project elements include an ultraviolet radiation disinfection system that would be located downstream of the SBRs in the vicinity of the existing chlorine contact chamber, and a new control building for electrical, instrumentation and process control, and general operations of the plant including laboratory, storage, and operator office functions.

Figure 15: Recommended Project Site Plan

5.2 Design Criteria

5.2.1 Target Discharge Quality

The reliability of the SBR process was emphasized in the analysis of alternatives. Sequencing batch reactors are a proven process that has been shown to consistently produce BOD and TSS effluent quality in the 5 to 10 mg/l range and, when designed to nitrify, reduce effluent ammonia concentrations to less than 1 mg/l. Design of an SBR system would be based on the ability to consistently achieve this effluent quality.

5.2.2 Design Flow

A surge basin created within a portion of the existing lagoon would allow for efficient and cost effective design of the treatment processes. Secondary and tertiary treatment units should be sized for the maximum month flow and the surge basin sized to handle the difference between maximum design flow for the treatment units and an extended high flow event.

The historical maximum month flow at the treatment facility during the period from 1996 to 2002 was 1.90 MGD. As stated in the influent flow analysis in Section 3, it is considered appropriate to design for the historical maximum, leading to the preliminary sizing of treatment processes based on a 2.0 MGD design flow.

An extended high flow event is based on a peak week, which includes several days of peak day flows. The equalization volume required to contain this event is approximately 7.0 million gallons, which can be accommodated in the surge basin shown in Figure 15.

5.2.3 Summary of Design Criteria

Table 15 summarizes the design criteria developed for the preferred project. Besides design flow and required effluent quality, the table summarizes the criteria used for each of the treatment process components and gives some preliminary estimates of required volumes. A more complete description of these design elements is contained in Section 5.3.

Table 15: Summary of Design Criteria for SBR Process

Design Flow	
Peak Day	2.0 MGD
Ave Day	0.5 MGD
BOD Loading	
Average	382 lbs /day
Peak-(2 MGD-50 mg/l)	1000 lbs/ day
Required Effluent Quality	
Effluent BOD	10 mg/l
Effluent TSS	10 mg/l
Effluent Ammonia	1 mg/l
SBR Reactors	
# of Basins	2
LxWxH	134'x50x24'
Total Volume	2.20 MG
Retention Time	26.4 hrs
Decant Volume	0.25 MG
Equalization Volume	0.20 MG
Aerobic Digestion	
# of Basins	2
Loading	0.1 lbs volatile solids/day
Surge Basin	
Volume	7 MG
Facultative Sludge Lagoon (FSL)	
Loading	0.1 lbs volatile solids/1,000 CF/day
Irrigation Storage	To be determined

5.3 Project Elements

5.3.1 Headworks Location

City staff described several disadvantages with the existing headworks site including lack of access during significant wet weather events, and general age of building and equipment. It was decided that the new influent pump station should be moved to the north side of Port Kenyon Road where it could be consolidated with the other treatment processes on a single site. The recommended project calls for the pump station and headworks to be constructed adjacent to the SBR reactors as shown on the site plan (see Figure 15). The existing pump station would stay on line during construction, continuing to pump influent to the primary oxidation pond for treatment, and be abandoned as appropriate after the new facility was completed.

5.3.2 Influent Sewer Lines

Two alternatives were investigated for connecting the existing collection system to the new wet-well. The two alternatives are shown in Figure 16. Alternative 1 assumes new gravity lines would be constructed from the manhole just upstream of the existing wet well to a new manhole in front of the proposed influent pump station. The invert elevation of the existing manhole is 4.69 feet. Assuming a minimum 0.4 % slope along the new gravity line heading west in Port Kenyon Road, the invert elevation at the new manhole would be approximately 0.00 feet MSL.

Alternative 2 consists of the construction of a new lift station on the east bank of Francis Creek, just upstream of the existing inverted siphon under the creek. This alternative would require the construction of approximately 1,400 lineal feet of new force main and a new gravity line from the manhole on California Street. Based on the hydraulic gradeline from this manhole the new wet well could be constructed at a higher elevation than would be required in Alternative 1.

The costs of the two alternatives are compared in Table 16. Alternative 1 would have added cost for the deeper wet well and the deep sewer line extension. Alternative 2 would have added cost for the new lift station and force main. Alternative 2 offers the added benefit of eliminating the existing inverted siphon beneath the creek, and may be preferred by the City as a remedy to ongoing maintenance problems with the siphon.

Table 16: Estimated Construction Costs for New Influent Pump Station and Headworks Alternatives

Item Description	Alternative 1	Alternative 2
Mobilization	$120,000	$120,000
Control Building	$50,000	$50,000
Influent Pump Station & Wet Well	$292,500	$282,500
Headworks	$365,750	$365,750
Booster Pump & Force Main	$0	$230,000
Gravity Sewer Extension	$113,000	$0
Total Construction	**$941,250**	**$1,048,250**

Figure 16: Influent Sewer Alternatives

5.3.3 Influent Pump Station

The hydraulic grade line shown in Figure 17 illustrates the necessity of installing a new influent pump station as part of the proposed project. In fact, because of deficiencies in the existing pump station, a new pump station and headworks is a recommended component of each of the proposed alternatives. As discussed in Section 4.4, upgrading the existing pump station was considered infeasible for a variety of factors. Instead it is recommended that a new pump station be constructed to pump flow up to the headworks for pre-screening and grit removal. The headworks would have sufficient elevation head for the influent to flow by gravity to secondary treatment processes from the flow splitter box near the entrance to the SBR basins. The splitter box would allow diversion of the flow to either one or both SBR reactor basins or, if flow exceeds 2 MGD, would direct excess flow to the surge basin.

It is recommended that the influent pump station have a firm capacity equal to the PIF of 6.0 MGD. Firm capacity is defined as the ability to deliver the required flow with one pump off line. This redundancy is necessary to prevent sewage overflows due to the failure of a single pump. It is recommended that three submersible pumps be installed with two being capable of meeting the PIF. Each of the three pumps would lift approximately 3.0 MGD. This design includes smaller pumps so that one pump can accommodate peak daily flow and two pumps can match the PIF. Variable frequency drives may be required on one or two of the pumps to match low flow conditions.

5.3.4 Headworks

The new headworks would be constructed at an elevation that provides sufficient hydraulic head for flow into the SBR reactors (or the surge basin). As indicated on the hydraulic profile shown on Figure 17, the influent would be pumped into a pre-screening channel, then would flow by gravity through a grit removal system and into the splitter box. The multi-leveled headworks structure would be constructed over the blower pump room, which would be located on the same level as the SBR reactors, and would share a common wall with them (see Figure 15).

5.3.4.1 Pre-Screening

One of the deficiencies in the existing treatment facility is the lack of a pre-treatment process for removing solid material such as rags, grit, and floatable, inorganic material. Designs for pre-screening units have been discussed in Section 4. It is recommended that in-channel screen be installed with a rotating inclined screw that lifts solids while allowing for soft organics to be washed back into the channel through a coarse screen. (Some systems such as the “auger monster” are provided with a grinder prior to the lifting screw to facilitate separation of biodegradable material and coarse solids).

Coarse solids lifted by the screw are fed into a screenings compactor to remove additional water and the dry solids drop into a dumpster for disposal to a landfill. The pre-screenings channel will

be constructed on a false floor, with space underneath for a dumpster at ground level. The dumpster will be placed on rails so that it can be easily accessed.

5.3.4.2 Grit removal

The head cell as discussed in Section 4 is proven technology for grit removal and is recommended as part of the proposed project. The head cell is a modular, multi tray solids concentrator. The flow distribution header evenly distributes influent onto conical trays and a vortex flow pattern develops. Solids settle on each tray, and then are swept down to the center underflow collection chamber.

A head cell would be placed in the center of a concrete tank and surrounded by circular channel which would extend around the circumference of the head cell. The bottom and sidewalls of the tank would be sloped down and an overflow weir installed across the tank. The overflow weir would overtop to a splitter box.

Grit pumped off the bottom of the concentrator would be pumped to cyclonic concentrator and auger style classifier. Installation of the classifier would allow grit to fall into a dumpster for transport to the landfill. If the classifier was installed on the same level as the in-channel screen grit could be collected in the same dumpster as the screenings.

5.3.4.1 Design Criteria

Pre-screening would be accomplished by a screen designed to handle the range of flows from current average dry weather flow (ADWF) to peak instantaneous flow (PIF). The channels would be designed to handle projected PIF without overflowing while maintaining 2 foot per second velocity at peak hourly flows during dry weather conditions.

Sizing of the grit removal system (called a head cell) is based on the ability to achieve 95% removal at peak day flows (PDAF). The removal at PIF would be reduced to approximately 75%. A head cell that would give 95% grit removal at a PDAF of 3.85 MGD would be approximately 6 feet in diameter and 16 feet deep. The concrete tank surrounding the cell would have an approximate diameter of 10 feet.

5.3.5 SBR Biological Treatment Process

The advantages of the SBR process have been summarized in the previous section and include the elimination of a secondary clarifier and RAS pumping equipment, high tolerance for peak flows and shock loading, clarification under ideal quiescent conditions, and process flexibility.

As the name implies, the SBR process is operated in a periodic (batch) manner, and utilizes a single tank for both bio-oxidation and sedimentation of suspended microorganisms. A typical operation sequence consists of five steps: fill, react, settle, decant, and idle. The treatment

sequence begins with the introduction of wastewater into a partially filled tank containing settled mixed liquor from a previous cycle (fill phase). In the react phase, aeration/mixing are provided for a length of time sufficient to produce effluent of the desired quality. In some instances, the fill and react phase are combined in an aerated fill step.

The mixed liquor is allowed to settle quiescently for another set period of time after the react phase, after which the clear supernatant liquid is decanted as effluent. The system is ready for refilling after the decant phase is completed. During the settle and decant phases in one tank, the other tank operates in the fill and react phases. Automated controls are utilized to concurrently operate equipment required for each treatment basin because the process continually cycles.

5.3.6 Flow Equalization

An effluent equalization basin is located downstream from the SBR units and shares a common wall with the treatment basins. This basin helps to even out the discharge from the SBR basins to the disinfection process. An automatic valve controls the discharge rate of the equalization basin.

The equalization basin is sized based on the ability of the basin to hold the volume of decant from one of the SBR reactors while the other is filling. The peak decant rate is 6,410 gpm based on preliminary design of the SBR units. The required equalization volume is 0.2 MG based on a peak discharge rate of 2.0 MGD (1440gpm).

5.3.7 UV Disinfection System

One of the advantages of replacing the stabilization lagoon with an SBR system is the ability to reliably produce high quality effluent, which can be disinfected with ultraviolet radiation. It is recommended that the City convert from gas chlorination disinfection to UV as part of the preferred project. Construction of the SBR reactors would require that the existing contact basin be modified, and the City could take the opportunity to upgrade its disinfection process and eliminate the safety and monitoring issues related to hazardous substances such as gaseous chlorine and sulfur dioxide. UV is environmentally friendly and does not require dechlorination. Other advantages include simplicity of operation and a relatively small footprint. Among the disinfection system options discussed in Section 4, UV was the disinfection alternative preferred by City staff.

Preliminary design of the UV system was based upon the following criteria: a peak flow of 2.0 MG, expected suspended solids concentrations in the SBR reactor effluent of less than or equal to 5 mg/l, a minimum transmissivity of 45%, and a required disinfection limit of less than 23 MPN /100 ml. Based on these criteria it was estimated that 7 modules of 40 bulbs each would be necessary to provide the required dose. It is recommended that a redundant module be supplied, and therefore the proposed design would have eight modules installed in a channel 48 feet long by 2 feet wide and 6 feet deep. The estimated construction costs for the system are presented in Table 17.

Table 17: Estimated UV System Construction Cost

UV System Equipment	$385,000
Mechanical	$60,000
Electrical/Controls	$80,000
UV Channel	$50,000
Site Work	$10,000
Yard Piping	$10,000
Total	**$595,000**

The City's NPDES permit requires that the effluent average less than 23 MPN/100 ml total coliform with a daily maximum of 230 MPN/100 total coliform. According to a leading manufacturer of UV systems, the expected finished water total coliform count should be in the range of 18 to 25 MPN/100 ml (assuming recommended dosages of effluent suspended solids are around less than or equal to 5 mg/ l).

Effluent limitations for disinfection at the Ferndale facility are very stringent and the City should verify that there would be no problem meeting these limits. One method of doing this would be to work with the UV manufacturer and request they perform total coliform testing of SBR effluent in locations where they have UV systems installed. Depending on the outcome of this "pilot testing" procedure, the need for an effluent filtration process could be determined.

5.3.8 Surge Basin

Equalization storage volume created within a portion of the existing lagoon would allow for efficient and cost effective design of the treatment processes. The secondary processes would be designed for the historical maximum month wet weather flow, meaning the surge basin would need to have enough capacity to store surges from the maximum week wet weather flow. The required surge capacity is estimated to be 7.0 MG (21.5 acre feet).

The surge basin would be drained regularly during the winter months to maintain storage capacity for the next large storm event. It would likely be kept completely empty during the summer months. The basin would therefore need to be graded and a gravity line installed such that wastewater could drain back to the headworks whenever treatment capacity is available. The surge basin would be lined with a synthetic material to minimize cleaning requirements, segregate underlying soil material from wastewater and settled organic solids, and protect the quality of underlying groundwater and adjacent surface water.

5.3.9 Aerobic Digestion

Biosolids from the SBRs (and backwash water from the effluent filters, if needed) would be pumped to one of two aerobic digesters. As shown in Figure 15, each of the digesters would be roughly half of the volume of an SBR reactor. Providing two digesters allows for settling and decant of one while the other is undergoing aeration, and provides backup for routine maintenance.

5.3.10 Facultative Sludge Lagoon (FSL)

A facultative sludge lagoon would be constructed within the westerly portion of the existing stabilization pond, as shown in Figure 11. Digested biosolids from the aerobic digesters would be pumped to this lagoon and stored for the long term. There is enough room in the stabilization pond for a lagoon to be created and sized to store solids for as long as 20 years. The FSL would provide additional stabilization of the solids, although the digesters would provide the majority of solids processing. Stored solids would be covered with a clear water cap to prevent odors from escaping the FSL surface.

5.3.11 Transitional Storage

There would be remaining space available in the existing stabilization pond for treated effluent storage during periods of low surface water flow, and to supplement the supply of irrigation water during the dry season. This storage volume provides a buffer and could be useful, for example, during the transition from the irrigation season to the wet weather discharge season in the fall of each year, and again in the late spring during the transition from wet weather discharge to disposal by irrigation. The reservoir could also help regulate the duration and frequency of spray irrigation on agricultural fields during summer and other dry periods of the year.

5.4 Hydraulic Profile

Figure 17 is a partial hydraulic profile of the proposed treatment plant improvements. The minimum required water level in the splitter box was estimated based upon the preliminary design of the SBR reactor, the required volume in the equalization tank following the reactors, and the maximum water level in the existing contact basin. Preliminary design of the reactors indicates that they will need to be 24 feet high with a sidewater depth of 22 feet. The decant volume is contained in the upper 5 feet of each of the reactors, and must be allowed to drain into the equalization tank without surcharging. The equalization tank would have a sidewater depth of 12 feet and would drain into the UV channel for radiation prior to discharge.

The static level to which the influent pumps must be able to lift the influent flow is 35 feet based on preliminary design criteria for the reactors and equalization basin, and on the estimated head losses through the screenings channel and grit removal process.

Figure 17: Recommended Project Hydraulic Profile

5.5 Construction Costs

The costs of the proposed projects include all physical components that are necessary for upgrading treatment at the Ferndale WWTF. The estimated costs include allowances for all of the physical components discussed in the project description.

5.5.1 Contingencies

A contingency factor equal to 15% of the estimated construction cost has been added to the overall project cost. In recognition that the cost estimates presented are based on conceptual planning, allowances must be made for variations in final quantities, bidding market conditions, adverse construction conditions, unanticipated specialized investigations and studies, and other factors which cannot be foreseen at this time but could increase final costs.

5.5.2 Engineering

The cost of engineering services for major projects typically includes special investigations, a pre-design report, surveying, foundation investigations, 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 typically range from 15 to 25 percent of the contract cost when all of the above services are provided. The lower percentage applies to large cost projects without complicated mechanical systems. The higher percentage applies to smaller, complicated projects. The estimated engineering cost for design and construction phase services related to the preferred project is allowed at 20% of the construction cost for planning purposes.

Additional engineering services may be required for City specialized projects. This could include geotechnical evaluations, structural evaluations, and other specialized consulting activities. Due to the nature of potential upgrade projects, required engineering support may be essential during the construction phase of the work. In some cases, details, specifications, and contract administration services may be appropriate for the development of some projects. The cost for these services would depend on the individual project and the level of support required.

5.5.3 Legal, Administrative, and Financing

An allowance of one percent (1%) of construction cost has been added for legal and administrative services. This allowance is intended to include internal project planning and budgeting, grant administration, liaison, interest on term loan financing, legal services, review fees, legal advertising, and other expenses associated with the project.

5.6 Annual Operating Budget

A comparison of annual operating costs for six alternatives (not including the No Action alternative) was presented as part of the alternative cost evaluations in Section 4.5. Increases in annual operating cost would be based on increased power costs for aeration and the additional pumping which would be required. The recommended SBR project had the mid-range increase in annual operating costs among the three mechanical treatment process alternatives.

The evaluation of annual operating costs did not include the cost of additional operations staff because the need to hire an additional operator was assumed for all alternatives. Currently the City employs only one operator full time. The City Manager, who is a certified operator, provides backup to his chief operator. The City has indicated that there is currently a need for at least one additional operator at the wastewater facility. In addition to providing adequate staffing, money for training should be included in the annual operating budget and operators should be encouraged to seek state certification.

SBRs are proven technology and have become easier to operate in recent years as a result of improvements to the mechanical and controls systems. The timed sequence of fill, aeration, and decant can be reliably automated, thus simplifying operation. The overall consistency of the process makes operation a straightforward process that should not need frequent adjustment after the initial start-up period.

Operations staff currently carry pagers to notify them of alarm conditions at the plant such as power failures, or high wet well levels at the influent pump station. The proposed improvements to treatment at the facility would require that, in addition to existing alarms, the operator in charge be notified of any critical system failures at the treatment plant which affect the SBR process. These additional alarms would include blower failure, valve failure, sludge pump failure, or high level alarms. This would ensure that the treatment plant process is not compromised and eliminate the need for continuous onsite monitoring.

5.7 Project Schedule

This report serves to satisfy Task A contained in the City's Cease and Desist Order, which is outlined in Section 3.4. Task A requires the City to outline an alternatives analysis and identify a time schedule for corrective action. Table 18 summarizes the time schedule for the recommended project improvements.

Table 18: Proposed Time Schedule for Implementing Correction Actions

Action	Action Completion Date
City Selection of Preferred Alternative	January 2004
Funding Agency Review and Approval	April 2004
Environmental Review and Permitting	October 2004
Survey and Final Design	September 2004 to March 2005
Bidding Process	April 2005
Construction	June 2005 to June 2006

5 CONCLUSIONS AND RECOMMENDATIONS

Based on the investigation performed by Spencer Engineering and Construction Management, Inc. and SHN Consulting Engineers and Geologists, Inc., it is our recommendation that the City of Ferndale move forward with the implementation of the SBR project. The SBR process is relatively expensive; however, it provides advanced treatment to ensure the protection of the receiving water body while incurring relatively low maintenance and operation costs to the City.

In reviewing the recommendations in this report, the City is encouraged to visit operating SBR plant such as the plant located in Rogue River, Oregon.

It is important to note that the City is concurrently upgrading its collection system. The goal of the improvements is to decrease the volume of I/I introduced to the WWTF during the wet season. Reduction of I/I would correct some of the surges the WWTF has experienced in past winters.

Upon review and approval of the Corrective Action Report, the report should be submitted to FEMA for consideration of funding the chosen improvements as mitigation to the effects caused by the 1998 flooding event. FEMA may only approve a portion of the project so other funding options should also be investigated. It is imperative that the City comply with the task list outlined in the Cease and Desist Order and the terms established in its NPDES to correct the wastewater discharge violations and curb additional fines from the State.

6 REFERENCES

U.S. EPA. 1993. Nitrogen Control. EPA/625/R-93/010. Washington D.C. (September).

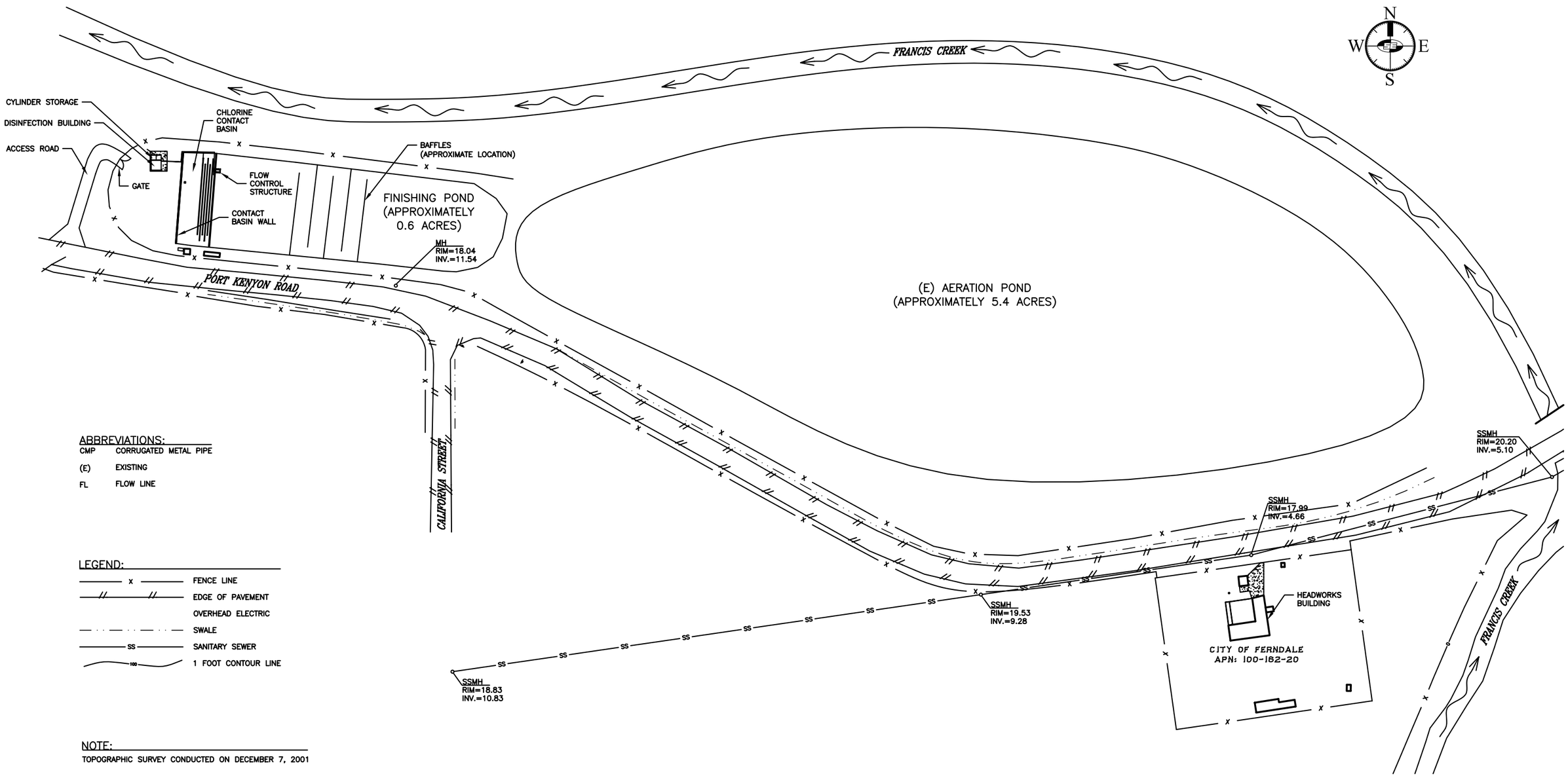
WPCF 1982. Wastewater Treatment Plant Design, Water Pollution Control Federation Manual of Practice No. 8. Washington D.C.

7 APPENDICES

7.1 Appendix A: NPDES Permit No. CA0022721

7.2 Appendix B: CDO No. R1-2003-0049

7.3 Appendix C: RWQCB Correspondence



ABBREVIATIONS:

CMP	CORRUGATED METAL PIPE
(E)	EXISTING
FL	FLOW LINE

LEGEND:

- FENCE LINE
- EDGE OF PAVEMENT
- OVERHEAD ELECTRIC
- SWALE
- SANITARY SEWER
- 1 FOOT CONTOUR LINE

NOTE:

TOPOGRAPHIC SURVEY CONDUCTED ON DECEMBER 7, 2001

PLAN

SCALE: 1"=50'

HALF SCALE

SPENCER ENGINEERING & CONSTRUCTION MANAGEMENT, INC.
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Designed: N. FIORENTINO
Drawn: J. BAKER
Checked: T. S. KELLY

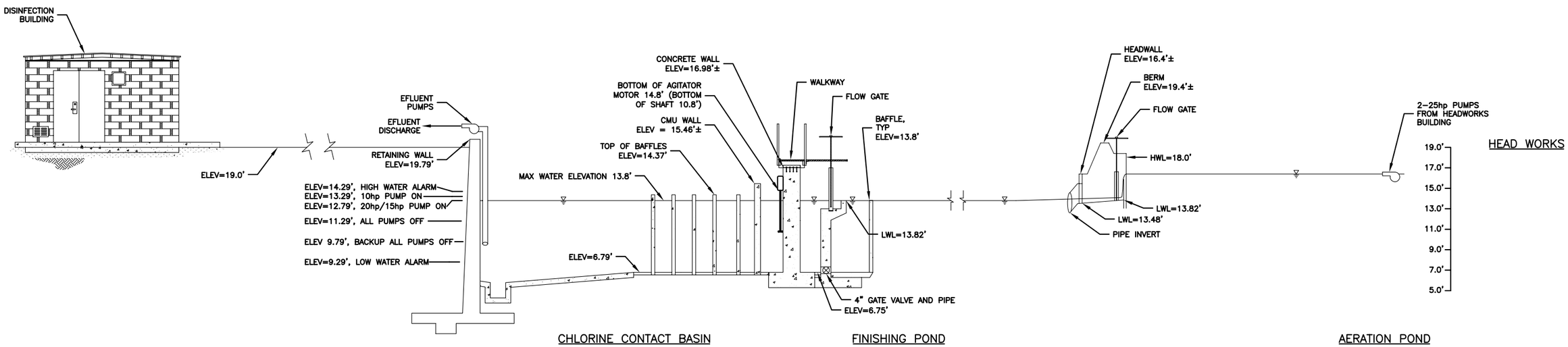
PREPARED FOR:
CITY OF FERNDALE

FIGURE 2 – SITE PLAN
WASTEWATER TREATMENT PLANT

File Name: EXISTING 8-5-03
Date: NOV. 24, 2003

0 1"

Job No. 98132.1
Sheet No. PAGE 10



EXISTING HYDRAULIC PROFILE

VERTICAL SCALE 1"=5'
HORIZONTAL N.T.S.

HALF SCALE

ABBREVIATIONS:

(E)	EXISTING
ELEV	ELEVATION
HWL	HIGH WATER LEVEL
LWL	LOW WATER LEVEL
TYP	TYPICAL

LEGEND:

CONCRETE	
GATE VALVE	⊗
PUMP	

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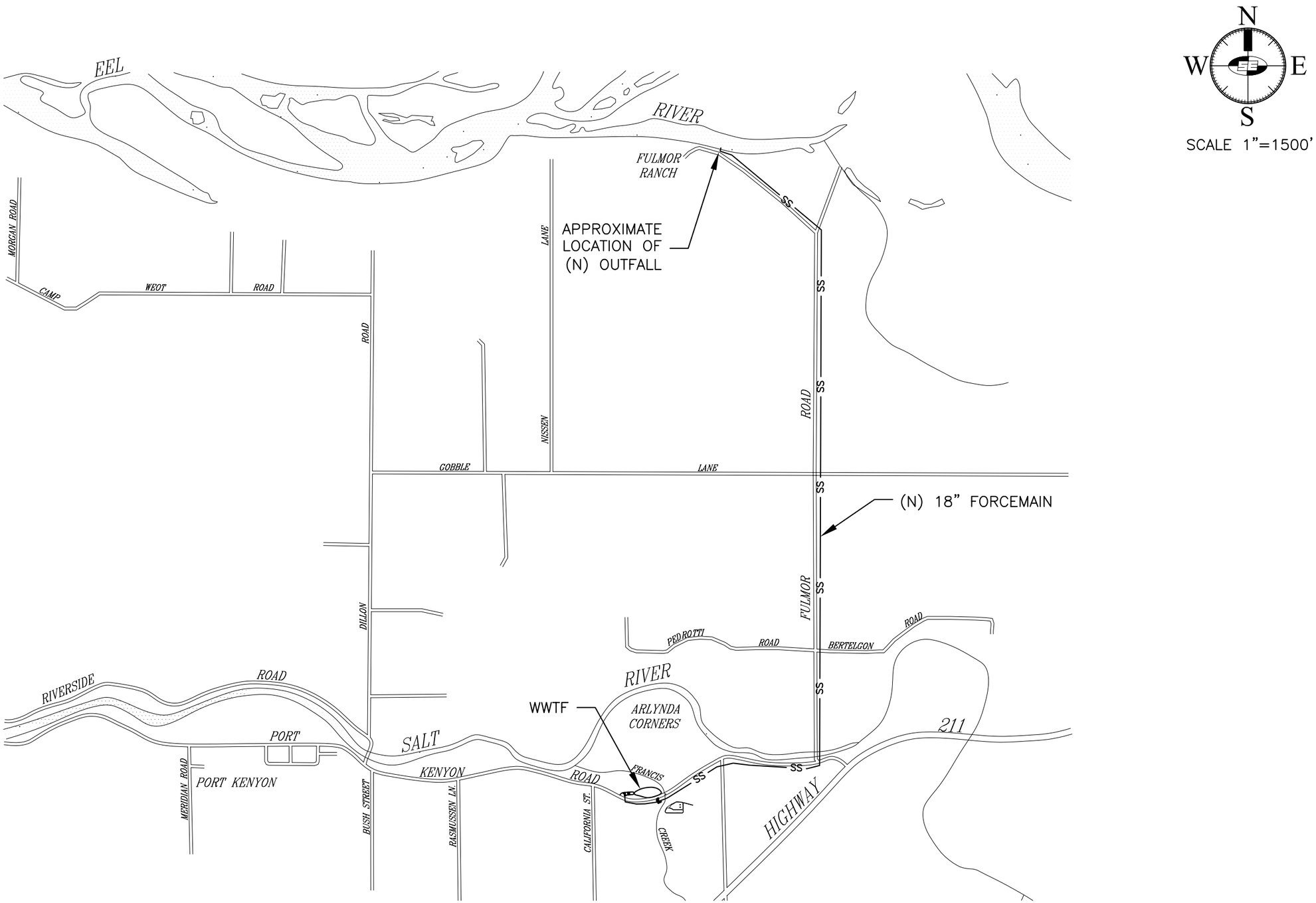
PREPARED FOR:
CITY OF FERNDALE

FIGURE 9: EXISTING HYDRAULIC PROFILE
WASTEWATER TREATMENT FACILITY

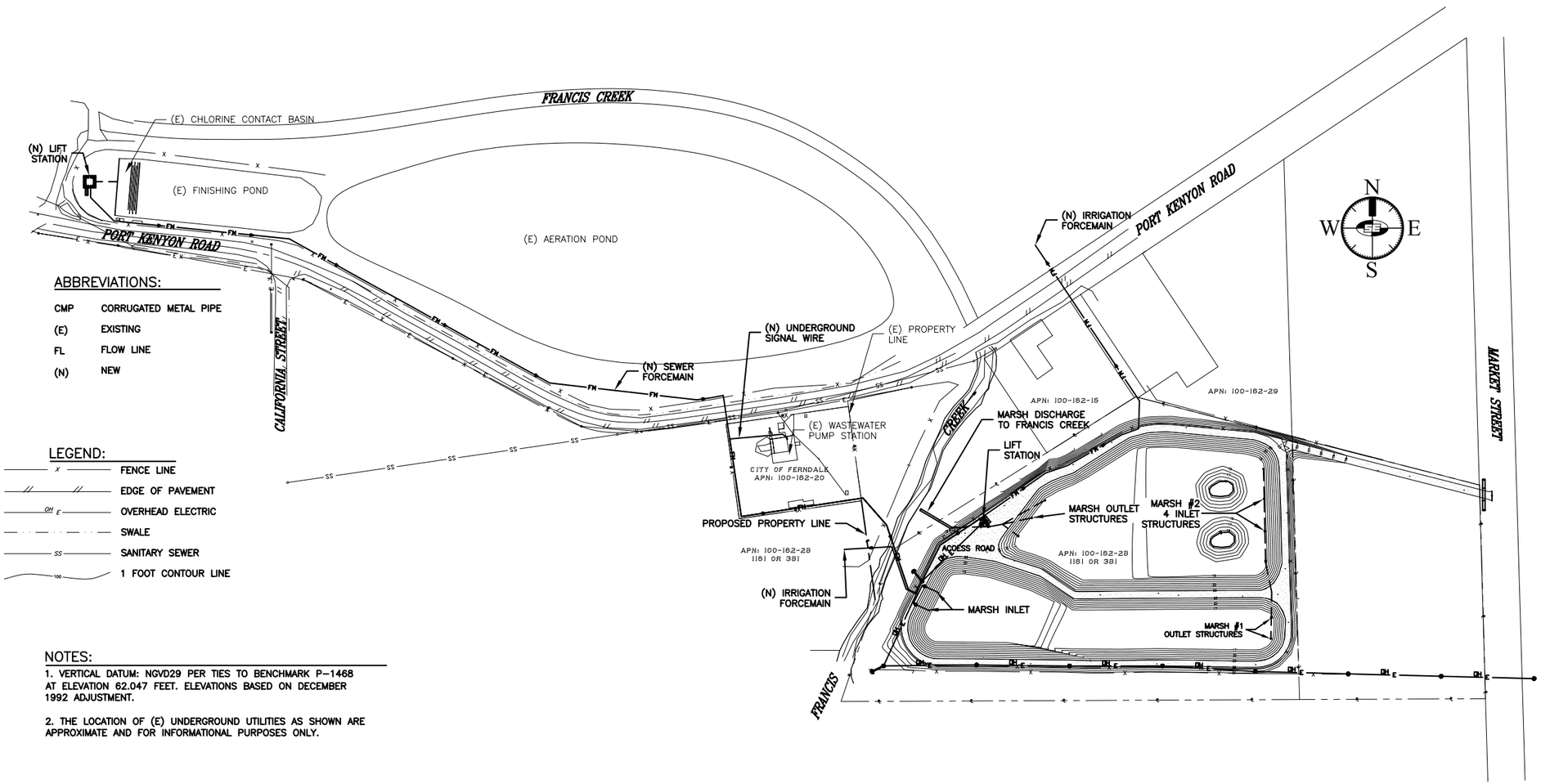
File Name: HYDRAULIC
Date: 10/24/03

0 1"

Job No. 98132.1
Sheet No. PAGE 20



SPENCER ENGINEERING & CONSTRUCTION MANAGEMENT, INC. 1585 HEARTWOOD DRIVE, SUITE F•McKINLEYVILLE, CA 95519 TELEPHONE (707) 839-4336 • FAX (707) 839-4012	Designed: N. FIORENTINO Drawn: J. BAKER Checked: T. S. KELLY	PREPARED FOR: CITY OF FERNDALE	FIGURE 10 - ALTERNATIVE 1 - EEL RIVER DISCHARGE WWTF CORRECTIVE ACTION REPORT	File Name: EEL-RIVER-ALT Date: NOV. 24, 2003	BAR IS ONE INCH ON ORIGINAL DRAWING 0 — 1" IF NOT ONE INCH ON THIS SHEET, ADJUST SCALES ACCORDINGLY
				Job No. 98132.1	Sheet No. PAGE 27



ABBREVIATIONS:

CMP	CORRUGATED METAL PIPE
(E)	EXISTING
FL	FLOW LINE
(N)	NEW

LEGEND:

- FENCE LINE
- EDGE OF PAVEMENT
- OVERHEAD ELECTRIC
- SWALE
- SANITARY SEWER
- 1 FOOT CONTOUR LINE

NOTES:

1. VERTICAL DATUM: NGVD29 PER TIES TO BENCHMARK P-1468 AT ELEVATION 62.047 FEET. ELEVATIONS BASED ON DECEMBER 1992 ADJUSTMENT.

2. THE LOCATION OF (E) UNDERGROUND UTILITIES AS SHOWN ARE APPROXIMATE AND FOR INFORMATIONAL PURPOSES ONLY.

TREATMENT MARSH LAYOUT

SCALE: 1"=100'

HALF SCALE

SPENCER ENGINEERING & CONSTRUCTION MANAGEMENT, INC.
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Drawn: J. BAKER
Checked: T. S KELLY

PREPARED FOR:
CITY OF FERNDALE

FIGURE 11 – ALTERNATIVE 2 – TREATMENT MARSH
FERNDALE WWTF CORRECTIVE ACTION REPORT

File Name: 98132_LAYOUT
Date: 11/24/03

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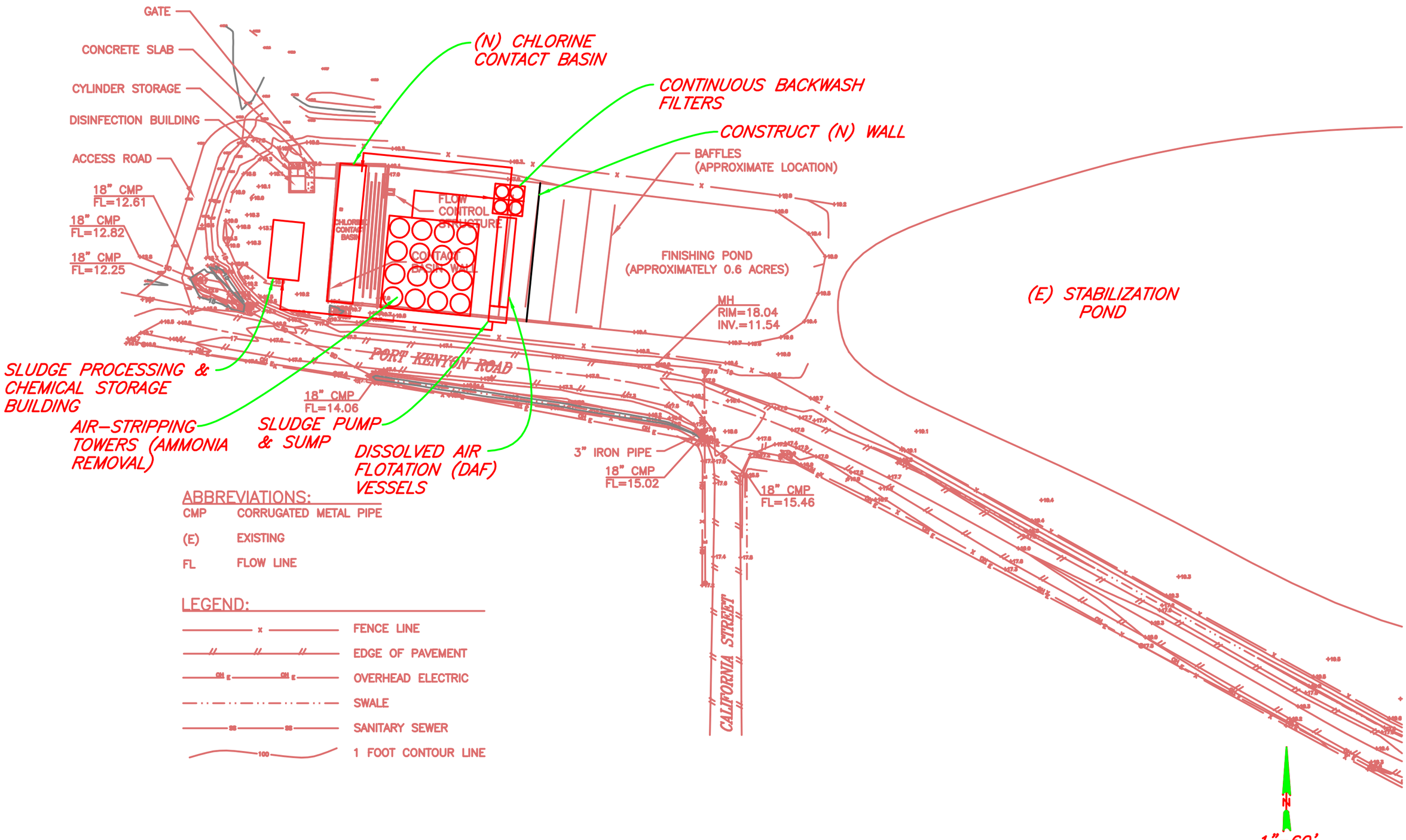


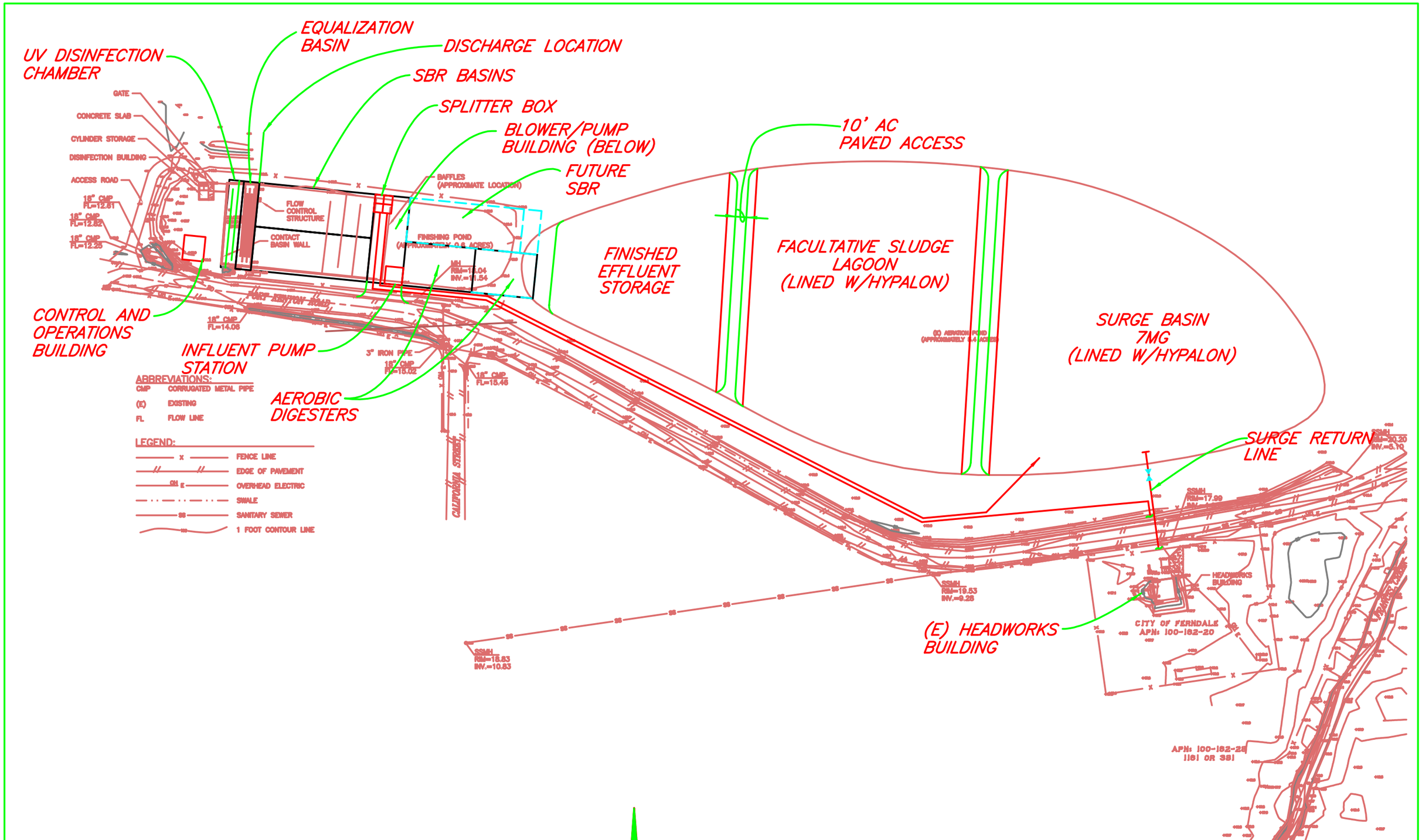
ABBREVIATIONS:

CMP	CORRUGATED METAL PIPE
(E)	EXISTING
FL	FLOW LINE

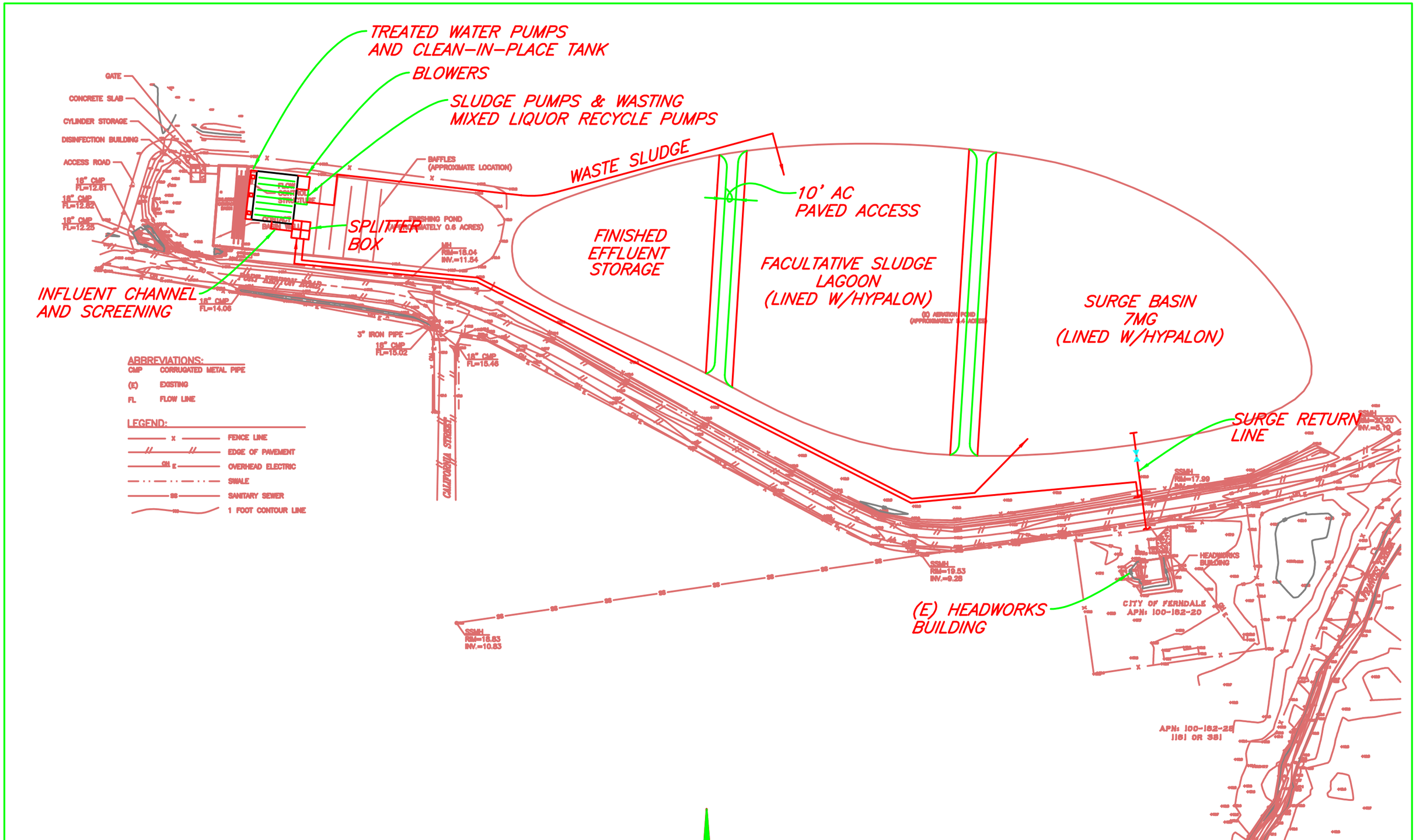
LEGEND:

- FENCE LINE
- EDGE OF PAVEMENT
- OVERHEAD ELECTRIC
- SWALE
- SANITARY SEWER
- 1 FOOT CONTOUR LINE

SHN Consulting Engineers & Geologists, Inc.	City of Ferndale USDA Preliminary Engineering Report Ferndale, California	Alternative 4 – Teriary Treatment of Lagoon Effluent
	November, 2003	003222-wwtf-f-12	figure 12



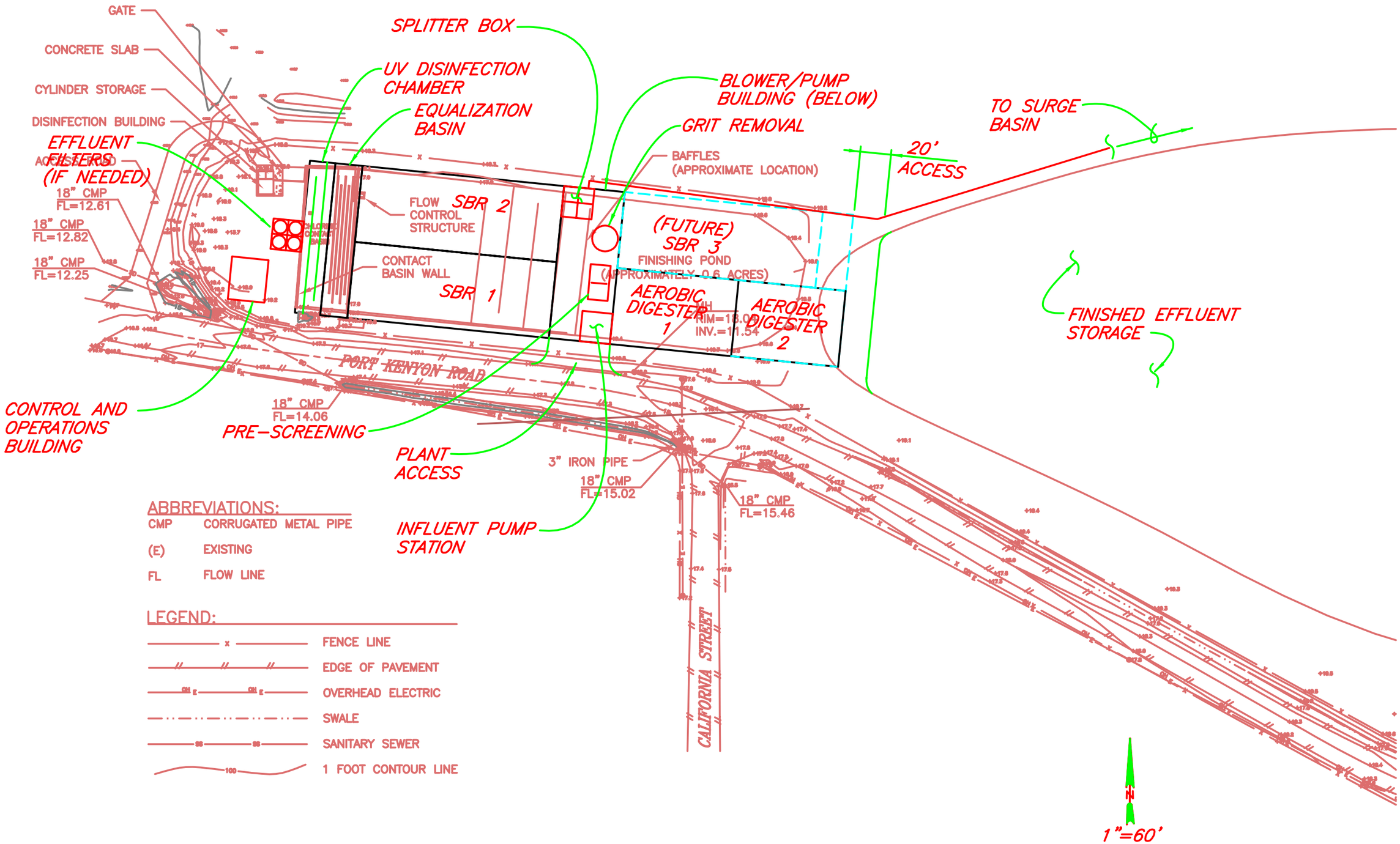
SHN Consulting Engineers & Geologists, Inc.	City of Ferndale WWTF USDA preliminary Engineering Report Ferndale, California	Alternative 5 – Sequencing Batch Reactors
	June, 2004	98132-6_ALT-5	Figure 13



SHN Consulting Engineers & Geologists, Inc.	City of Ferndale USDA Preliminary Engineering Report Ferndale California	Alternative 6 – Membrane Bioreactor SHN 003222
	November, 2003	003222–wwtf–f–14	Figure 14

TABLE 14: CORRECTIVE ACTION ALTERNATIVES FOR WASTEWATER TREATMENT

Alt.	Description	Advantages	Disadvantages	Capital Cost Est.	Annual O&M Cost Est.	Land Required	Meets Water Quality Criteria	Environmental	Public Acceptance	RWQCB Acceptance	FEMA Acceptance
1	Pump effluent to outfall on Eel River to obtain required dilution.	Moving effluent outfall to new location with adequate flow to obtain required dilution.	Moderate operating costs associated with a lift station and approx. 2 miles of pressure piping to Eel River. Discharge point would be in an actively migrating river channel. Does not solve existing poor treatment problems of the WWTF. No ammonia removal.	$3,927,000	Moderate ($29,000)	Possible at Outfall	Questionable	Possible local opposition. Permits include State Coastal Development Permit, ACOE Section 401 Permit, RWQCB Section 404 Permit, and new NPDES Permit	Questionable	Likely	Likely
2	Improve wastewater effluent by adding a Constructed Wetland to the existing system (approximately 9 acres).	Would add storage capacity to system to meet 20:1 dilution requirements. Would likely result in permanent permit modification, adding to system reliability and capacity. Moderately expensive system to maintain and operate. Provides enhanced marsh habitat.	Would require large amount of pasture land to be taken out of production (could be partially offset by land being a public attraction similar to Arcata's treatment marsh/wildlife sanctuary). Cannot guarantee a specified level of treatment. Project may include an unwilling landowner. Would take wetland designated land. Potential mosquito issue. Does not solve existing WWTF problems.	$2,950,000 (cost does not include $90,000 for land acquisition)	Moderate-High ($42,000)	~ 9 acres	Questionable	Is not an anticipated problem based on preliminary discussions with FEMA, RWQCB, and Army Corps of Engr. Will require County re-zone, General Plan Amendment and Use Permit, State Coastal Development Permit, ACOE Section 401 Permit, and RWQCB Section 404 Permit.	Likely	Unknown	Likely
3	Comply with dilution requirement by adding a Storage Pond to the existing system (40 acres).	Would add storage capacity to system to meet the 100:1 dilution requirements. Would add minimal operation or maintenance costs to the system.	Would require an extensive area of land (40 acres). Does not provide additional treatment to the system. Project may include an unwilling landowner. Would take wetland designated land. Potential mosquito issue. Does not solve existing WWTF problems.	$3226,000 (cost does not include $300,000 for land acquisition)	Low ($13,000)	~40 acres	Questionable	Will require County re-zone, General Plan Amendment and Use Permit, State Coastal Development Permit, ACOE Section 401 Permit, and RWQCB Section 404 Permit.	Moderate	Likely	Likely
4	Tertiary Treatment of Lagoon Effluent	Uses existing lagoon process as secondary treatment without major capital upgrade to the existing facilities. Existing lagoon basin provides some surge capacity. Effluent filter or membrane options are capable of achieving quality suitable for reclaimed water, assuming they are preceded by the DAF. 90% removal of ammonia. Enhances water quality in the Salt River.	Secondary effluent requires pre-treatment prior to final filtration. Process reliability continues to be subject to seasonal changes. High chemical costs for polymer and flocculant are expected. Ammonia removal with air stripping adds significantly to operational complexity. Air stripping requires chemical addition to elevate for pH, which translates into significant operations and maintenance concerns. Solids handling processes are required for solids from pre treatment processes and filter backwashes. The treatment system would be highly operator intensive.	$7,432,000	High ($59,000)	No	Likely	Will require a State Coastal Development Permit and an NPDES Permit.	Questionable	Likely	Questionable
5	Sequencing Batch Reactors (SBRs)	Reliable high quality effluent. Ammonia (90%) and nitrate removal. Straightforward, relatively simple operation. Small footprint for reactors. Surge basin to limit peak flows and reduce cost of treatment. Limited or no chemical use. Uses existing lagoon to provide long term solids handling and storage (FSL). Enhances water quality in the Salt River.	Secondary process with associated increased complexity of operation. The system requries the addition of solids processing.	$6,768,000	Moderate-High ($36,000)	No	Likely	Will require a State Coastal Development Permit and an NPDES Permit.	Likely	Likely	Questionable
6	Membrane Bioreactors	The system produces consistently high effluent quality and is contained in a small footprint. The system eliminates or significantly reduces need for disinfection system. 90% removal of ammonia. Reduced sludge volumes. Would not require solids processing line. Enhances water quality in the Salt River.	This is a new technology with which most operators lack familiarity The system has some operational complexity and chemical cleaning of membranes is required. High capital cost. No long term operational data available (technology less than 5 years)	$8,570,000	Moderate-High ($43,000)	No	Likely	Will require a State Coastal Development Permit and an NPDES Permit.	Questionable	Likely	Questionable
7	No Action	No additional capital costs for the City or need for additional land acquisition.	The City would remain in violation and would continue to be fined for discharge violations.	$0	None	No	No	City would remain in non-compliance.	Questionable	Unlikely	Unlikely



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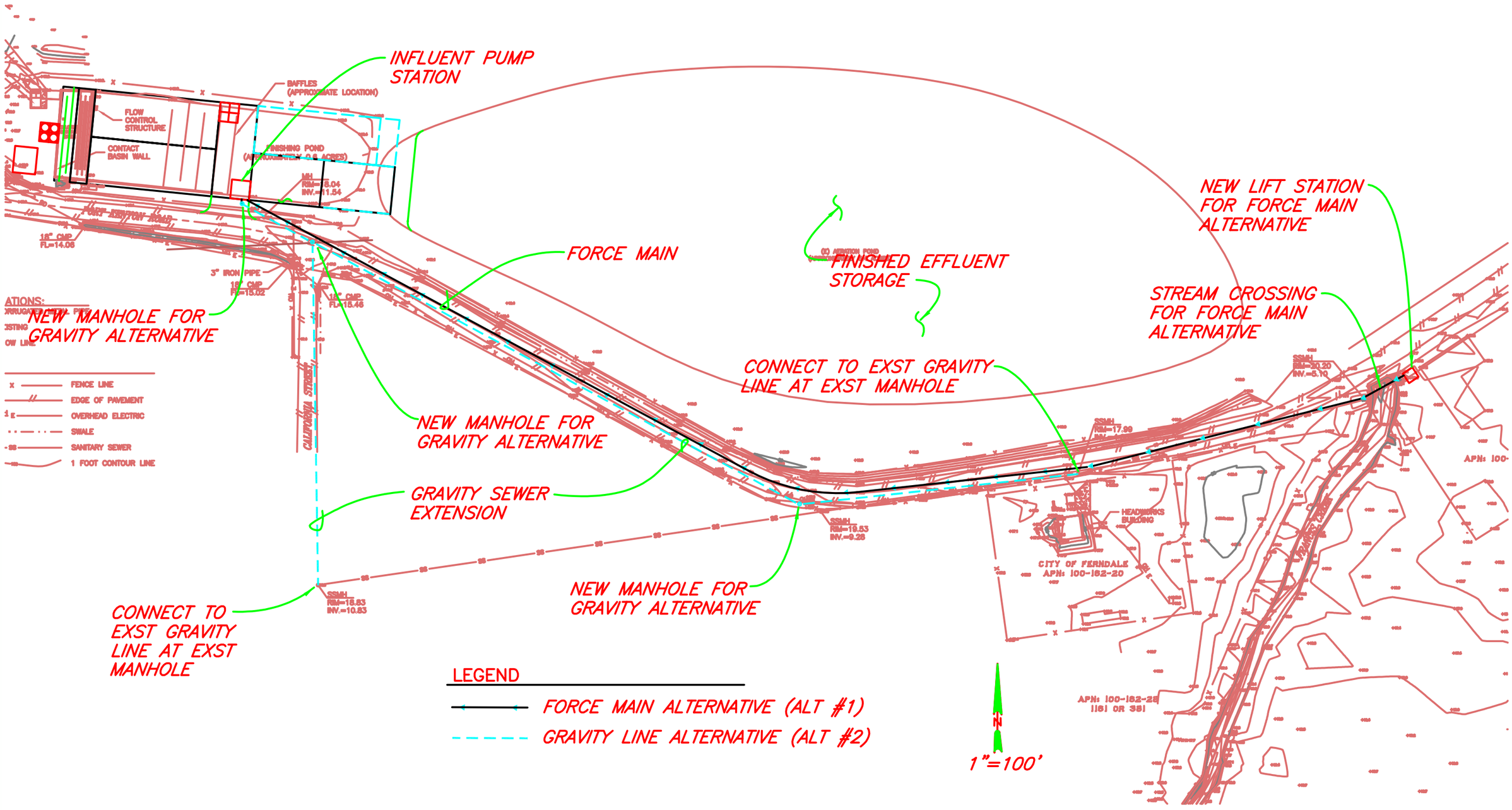
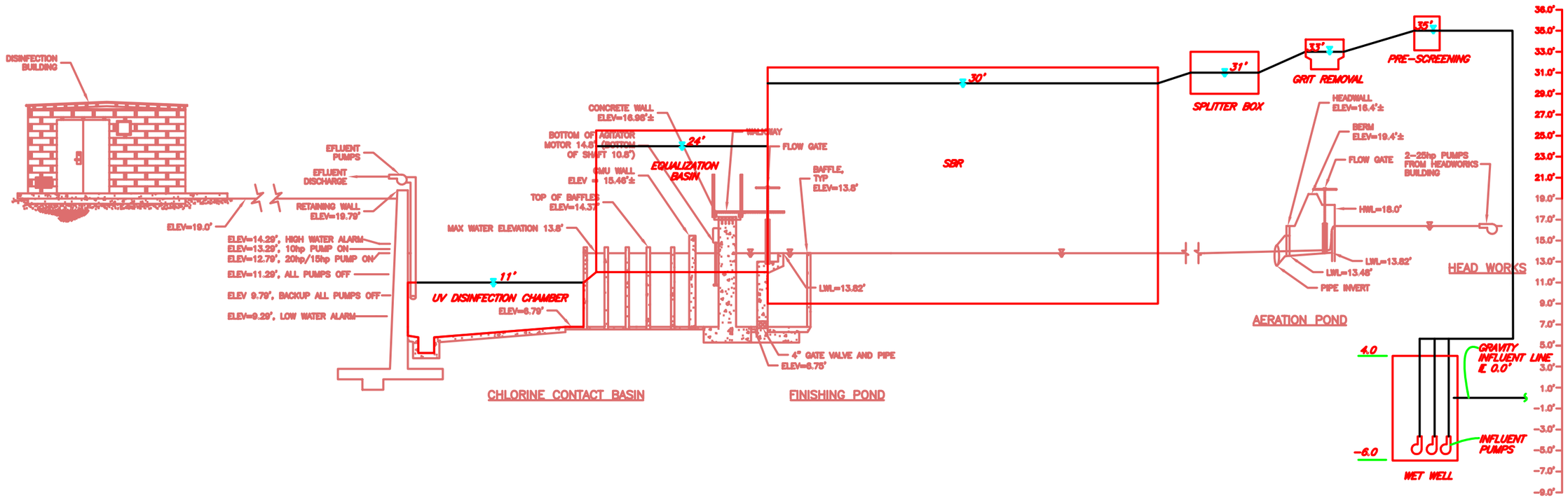
- SANITARY SEWER
- 1 FOOT CONTOUR LINE

SHN Consulting Engineers & Geologists, Inc.	City of Ferndale WWTF Corrective Action Plan Ferndale, California	Recomended Project Site Plan SBR- Treatment Process SHN 003222
	November, 2003	003222-wwtf-f-15	Figure 15



SHN Consulting Engineers & Geologists, Inc.	City of Ferndale WWTF Corrective Action Plan Ferndale, California	Influent Sewer Alternatives SBR– Treatment Process SHN 003222
	November, 2003	003222–wwtf–f–16	Figure 16



HYDRAULIC PROFILE

VERTICAL SCALE 1"=5'
HORIZONTAL N.T.S.

ABBREVIATIONS:

(E)	EXISTING
ELEV	ELEVATION
HWL	HIGH WATER LEVEL
LWL	LOW WATER LEVEL
TYP	TYPICAL

LEGEND:

CONCRETE	
GATE VALVE	
PUMP	

HALF SCALE

SHN Consulting Engineers & Geologists, Inc.	City of Ferndale WWTF Corrective Action Plan Ferndale, California	Recommended Project Hydraulic Profile SBR- Treatment Process
	November, 2003	003222-wwtf-f-17	Figure 17