

Part 2 Operations Evaluation

3.0 Wastewater Characterization

Municipal waste loadings were characterized using flow monitoring and sampling data collected from May 2003 through October 2010.

3.1 Influent Flow Analysis

Influent WWMF flow characteristics were evaluated based on influent flow data provided by MCSD and precipitation data for the period from November 2003 through May 2010. Table 3-1 summarizes the flow data.

Table 3-1 WWMF ¹ Monthly Influent Flow Summary MCSD Wastewater Management Facility							
Month/Year	Influent Flow (MGD) ²						
	2003-04	2004-05	2005-06	2006-07	2007-08	2008-09	2009-10
November	0.925	0.858	0.967	0.892	0.971	0.926	0.940
December	1.135	0.918	1.229	1.128	1.085	1.019	0.962
January	1.203	1.081	1.567	1.146	1.277	1.055	1.182
February	1.280	0.940	1.453	1.333	1.288	1.085	1.144
March	1.126	0.974	1.575	1.325	1.169	1.189	1.226
April	0.988	1.118	1.384	1.145	1.053	1.028	1.270
May 1-14	0.924	0.849	1.129	1.081	0.985	1.093	1.204
Average	1.083	0.962	1.329	1.150	1.118	1.056	1.133
AWWF ³	1.119 MGD						
May 15-31	0.906	0.848	1.039	0.997	0.920	0.985	1.105
June	0.857	0.855	0.980	0.940	0.897	0.925	1.117
July	0.885	0.855	0.908	0.901	0.872	0.872	0.930
August	0.847	0.844	0.887	0.884	0.867	0.879	0.930
September	0.851	0.849	0.897	0.898	0.885	0.883	0.884
October	0.875	0.848	0.899	0.944	0.884	0.894	0.945
Average	0.870	0.850	0.935	0.927	0.888	0.906	0.985
ADWF ⁴	0.909 MGD						
1. WWMF: Wastewater Management Facility 2. MGD: Million Gallons per Day 3. AWWF: Average Wet Weather Flow: The average influent flow during period from November 1 through May 14. 4. ADWF: Average Dry Weather Flow: The average influent flow during period from May 15 through October 31.							

3.1.1 Average Dry and Wet Weather Flows

Average Dry Weather Flow (ADWF) is the average influent flow during the months of May through October. For the purposes of this analysis, the dry season flow has been defined to correspond with the period of prohibited discharge to the Mad River, May 15 through September 30. Due to low regional rainfall averages in October, this month has also been included in the average dry weather flow analysis. Based on analysis of the dry weather season data for May 15, 2003 through October 31, 2010, the ADWF is approximately 0.909 Million Gallons per Day (MGD).

The ADWF can be divided into two descriptive components: base sanitary flow and base infiltration. The portion of treatment plant flow that is entirely attributable to sanitary sewage is known as the base sanitary flow. The base sanitary flow was estimated at 0.830 MGD, based on minimum influent flows during periods of extended dry weather.

The difference between the ADWF and the base sanitary flow is the base infiltration rate. Base infiltration rates depend upon such factors as the quality of material, workmanship, age, and condition in the sewers and building connections; maintenance efforts; and groundwater elevations compared with the elevation of the sewer pipes. A base infiltration rate of 10 to 20 gallons per day (gpd) per Equivalent Dwelling Unit (gpd/EDU) is considered unavoidable infiltration. EDUs are defined as any single-family residential structure.

Based on an ADWF of 0.909 MGD, and an estimated base sanitary flow of 0.830 MGD, the base infiltration rate at the MCSD WWMF was estimated to equal 0.079 MGD. During this period, it was estimated that on average approximately 5,000 Equivalent Dwelling Units (EDUs) contributed wastewater to the collection system, resulting in a base infiltration rate of 16 gpd/EDU.

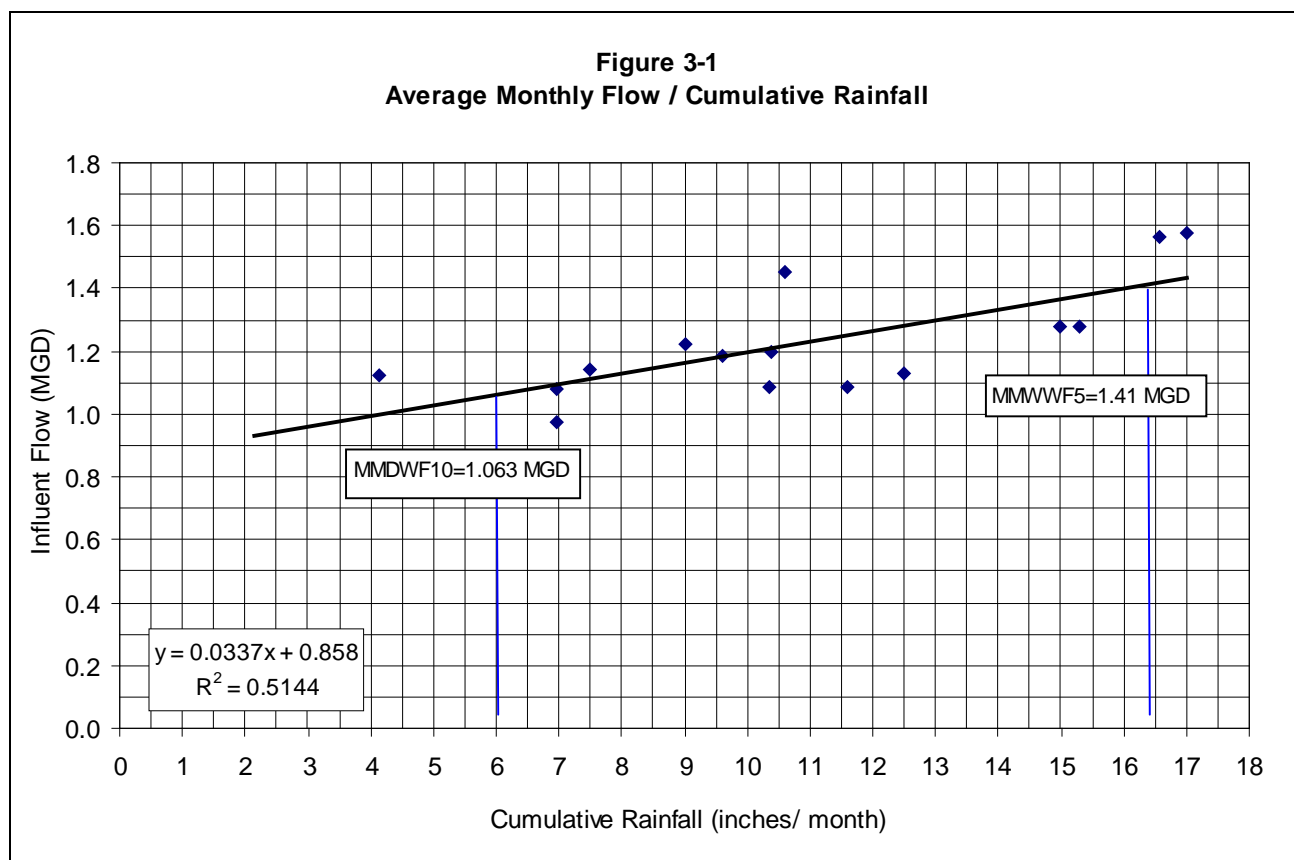
Average Wet Weather Flow (AWWF) is the average influent flow during the months of November through May. For the purposes of this analysis, the wet season flow has been defined to extend until the period of prohibited discharge to the Mad River begins on May 15. Based on analysis of the wet weather season data for November 2003 through May 14, 2010, the AWWF is approximately 1.119 MGD.

3.1.2 Maximum Monthly Dry and Wet Weather Flows

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

Table 3-2 lists the data points used for the maximum monthly flow analysis and Figure 3-1 presents the graphical representation of flow as a function of cumulative rainfall for the MCSD WWMF.

Table 3-2 Monthly Average Influent Flows and Precipitation MCSD Wastewater Management Facility			
Month	Year	Influent Flow (MGD) ¹	Total Monthly Precipitation (inches)
January	2004	1.200	10.38
February	2004	1.280	15.30
March	2004	1.126	4.13
January	2005	1.081	6.97
January	2006	1.567	16.57
February	2006	1.453	10.60
March	2006	1.575	17.00
December	2006	1.128	12.50
November	2007	0.971	6.96
December	2007	1.085	11.61
January	2008	1.277	15.00
February	2009	1.085	10.36
March	2009	1.189	9.62
February	2010	1.144	7.50
March	2010	1.226	9.00
1. MGD: Million Gallons per Day			



Monthly total precipitation data from the National Weather Service Eureka Woodley Island Station for the period of record (May 1906 - January 2009) was used as a basis for statistical estimation of return intervals. To derive an accurate estimate of rainfall in McKinleyville, the recorded precipitation from the Eureka gage was corrected by a factor relating it to measured data at the MCSD facility. Precipitation data measured at the MCSD facility from May 2003 through January 2009 indicated that the recorded monthly cumulative rainfall in May was an average of 1.3 times that measured at Eureka, and the cumulative monthly rainfall in January was an average of 1.7 times that measured at the weather station.

Maximum Month Dry Weather Flow-10 (MMDWF₁₀) is the maximum monthly average dry weather flow with a 10% probability of occurrence. This flow represents the dry weather season monthly average flow with a recurrence interval of 10 years. For the purposes of this analysis, the dry season flow has been defined to correspond with the period of prohibited discharge to the Mad River, May 15–September 30; and all of October is also included in the dry season period.

A statistical analysis determined the estimated monthly rainfall at the MCSD facility with a 1-in-10-year recurrence interval in May to be 6.08 inches. Based on the linear regression shown in Figure 3-1, this corresponds to a MMDWF₁₀ of 1.063 MGD.

Maximum Month Wet Weather Flow-5 (MMWWF₅) is the maximum monthly wet weather average flow with a 20% probability of occurrence. This flow represents the wet season monthly average flow that is anticipated to have a 5-year recurrence interval.

Based on the monthly total precipitation data, the monthly rainfall with a 1-in-5 year recurrence interval in January is 16.48 inches. Based on the linear regression shown in Figure 3-1, this corresponds to a MMWWF₅ of 1.41 MGD.

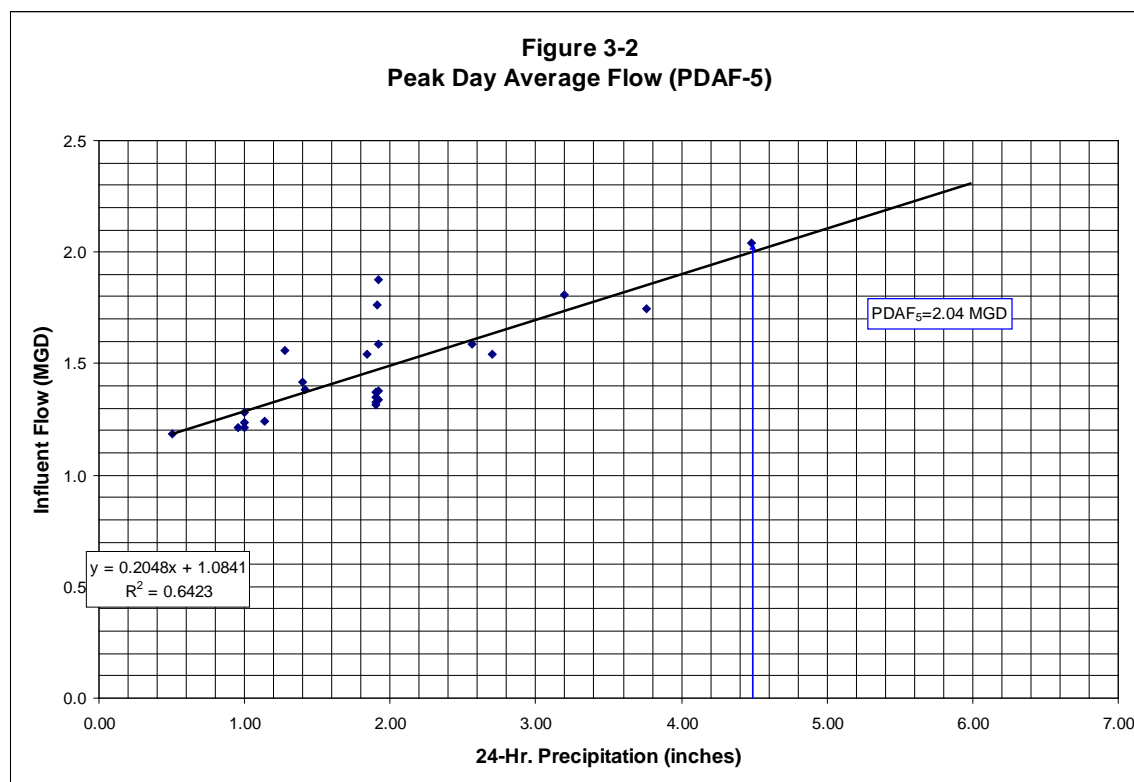
3.1.3 Peak Day Average Flow

Peak Day Average Flow-5 (PDAF₅) is the largest daily flow associated with a 5-year, 24-hour precipitation event. The peak day average flow has a 0.27% probability of occurrence or 1 day in 365 days of any given year. Estimation of peak day flow is based on a regression analysis of daily plant flows during or immediately following significant rainfall events during the wet season.

Because the increased influent flow to the WWMF during wet weather is highly correlated with rainfall, evaluation of this regression can be used to define peak day flow associated with a specific rainfall event. The PDAF₅ event is determined from a plot of the recorded daily flow that occurred during or 24 hours after a significant rainfall event. Table 3-3 lists the data points used for the peak day average flow analysis.

Table 3-3 Data Points for Peak Daily Average Flow Analysis MCSD Wastewater Management Facility			
Date	Daily Precipitation (inches)	Date	Influent Flow (MGD)¹
2/17/2004	1.84	2/17/2004	1.540
2/26/2004	2.56	2/26/2004	1.590
12/31/2004	1.92	1/1/2005	1.339
12/1/2005	1.42	12/1/2005	1.383
12/28/2005	1.91	12/28/2005	1.765
1/7/2006	1.40	1/7/2006	1.419
1/20/2006	1.92	1/21/2006	1.879
2/27/2006	3.76	2/28/2006	1.747
4/15/2006	1.92	4/15/2006	1.586
2/21/2007	4.48	2/27/2007	2.042
1/5/2008	1.28	1/5/2008	1.559
1/27/2008	1.92	1/28/2008	1.380
1/31/2008	3.20	2/2/2008	1.811
12/28/2008	2.70	12/28/2008	1.544
2/16/2009	0.96	2/16/2009	1.214
2/22/2009	1.14	2/23/2009	1.243
1/19/2010	1.90	1/19/2010	1.313
1/30/2010	1.00	1/30/2010	1.281
2/2/2010	0.50	2/2/2010	1.182
2/14/2010	1.00	2/15/2010	1.215
2/26/2010	1.90	2/26/2010	1.326
3/2/2010	1.90	3/3/2010	1.349
3/12/2010	1.90	3/12/2010	1.374
3/24/2010	1.00	3/25/2010	1.238
1. MGD: Million Gallons per Day			

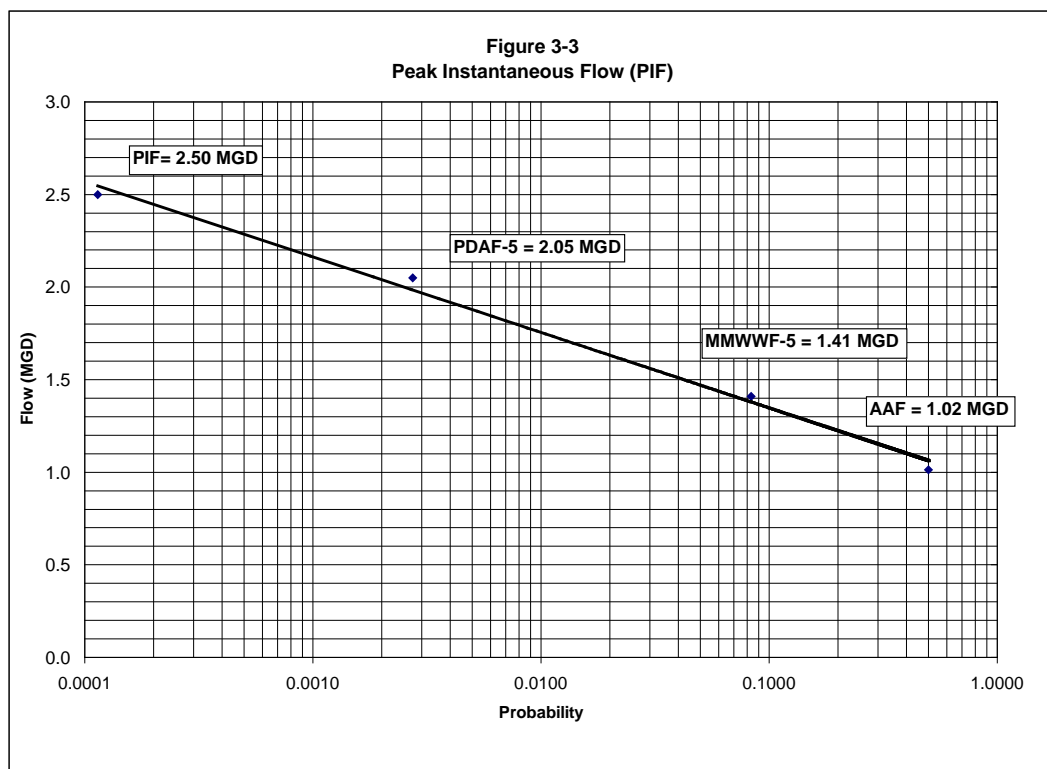
By performing a regression analysis of this data, a linear relationship is established, as shown in Figure 3-2. The PDAF₅ is based on the intercept of this line with the 5-year, 24-hour precipitation event. Based on *Isopluvials of the 5-Yr Precipitation for Northern California* (NOAA, 1973) the 24-hour precipitation with a 5-year recurrence interval is 4.5 inches. Based on the regression analysis shown in Figure 3-2, the resulting PDAF₅ for a 4.5-inch event is equal to 2.045 MGD.



3.1.4 Peak Instantaneous Flow

Peak Instantaneous Flow-5 (PIF₅) is the highest sustained hourly flow resulting from a 5-year storm during high groundwater periods. The PIF is used as the basis of design for the required hydraulic capacity of conveyance and treatment system components.

The PIF₅ has 0.011% probability of occurrence (1 hour in 8,760 hours of the year) and can be extrapolated from a probability plot of the flows derived in the previous section, using logarithmic probability paper. Figure 3-3 shows a graphical representation of a probability plot of Average Annual Flow (AAF), MMWWF₅, and PDAF₅. The PIF₅ for the MCSD WWMF is estimated to be 2.5 MGD.



3.1.5 Influent Flow Analysis Summary

Table 3-4 summarizes the results for the MCSD WWMF influent flow analysis based on data collected from November 2003 through October 2010.

Table 3-4 Influent Flow Analysis Summary of Results¹ MCSD Wastewater Management Facility			
	MGD²	gpd/EDU³	gpcd⁴
Base Sanitary Flow	0.830	166	64
Base Inflow and Infiltration	0.079	16	6
Average Dry Weather Flow (ADWF)	0.909	182	70
Average Wet Weather Flow (AWWF)	1.119	224	87
Average Annual Flow (AAF)	1.022	204	79
Maximum Month Dry Weather Flow (MMDWF-10)	1.063	213	82
Maximum Month Wet Weather Flow (MMWWF-5)	1.413	283	110
Peak Day Average Flow (PDAF-5)	2.045	409	159
Peak Instantaneous Flow (PIF-5)	2.500	500	194
1. Influent flow analysis based on flow data collected November 2003 through October 2010 2. MGD: Million Gallons per Day 3. gpd/EDU: gallons per day per Equivalent Dwelling Unit (EDU); based on an average of 5,000 EDUs served by MCSD during this period 4. gpcd: gallons per capita per day (2.58 persons per household, equivalent population 12,897)			

The EPA has developed guidelines for assessing peak flow data and determining the acceptable amount of I/I in a sewer system (EPA, 1985). The EPA considers infiltration not to be excessive if during periods of high groundwater and dry weather the highest average daily flow recorded over a 7 to 14 day period does not significantly exceed 120 gallons per capita per day (gpcd); inflow is not considered excessive if during a storm event, the highest daily flow recorded is less than or equal to 275 gpcd (EPA, 1985). As shown in Table 3-4, the MMDWF-10 of 82 gpcd meets the guidelines for non-excessive infiltration and the PDAF-5 of 159 gpcd meets the criteria for non-excessive inflow.

3.2 Wastewater Characteristics

The MCSD WWMF serves residential and commercial customers in the unincorporated community of McKinleyville. The community is basically residential and the majority of commercial establishments are service oriented in nature. Commercial water users and wastewater contributors, including restaurants and bars, gas stations, automotive repair and detailing services, wholesale foods, a small brewery, a winery, and a variety of retail shops and offices, account for less than 10% of the wastewater influent that is conveyed to the MCSD WWMF.

3.2.1 Influent Loading

Table 3-5 summarizes influent loadings of Biochemical Oxygen Demand (BOD), Total Suspended Solids (TSS), and Total Ammonia-Nitrogen (NH₃-N). The loadings presented in Table 3-5 are based on monitoring data collected at the MCSD WWMF from 2003 through 2010.

Table 3-5 WWMF¹ Influent BOD², TSS³, and NH₃-N⁴ Loading Summary MCSD Wastewater Management Facility												
	BOD				TSS				NH ₃ -N			
	Average		Max Mo. Ave.		Average		Max Mo. Ave.		Average		Max Mo. Ave.	
Year	mg/L ⁵	ppd ⁶	mg/L	ppd	mg/L	ppd	mg/L	ppd	mg/L	ppd	mg/L	ppd
2003	229	1,892	327	3,035	253	2,168	400	3,964	33	294	37	353
2004	228	1,795	327	2,572	219	1,737	350	2,752	33	263	37	292
2005	238	1,863	305	2,565	247	1,919	410	3,103	34	276	39	336
2006	266	2,379	358	2,729	262	2,383	520	2,315	33	303	40	364
2007	274	2,295	333	2,654	203	1,699	280	2,188	35	301	40	335
2008	291	2,370	340	2,675	204	1,648	245	1,986	35	294	39	352
2009	281	2,203	345	2,603	214	1,729	273	2,335	39	313	43	330
2010	248	2,166	360	2,894	218	1,899	350	2,814	42	375	50	440
Average	257	2,120	---	---	227	1,898	---	---	36	302	---	---
Max Mo.	---	---	360	3,035	---	---	520	3,964	---	---	50	440
1. WWMF: Wastewater Management Facility 2. BOD: Biochemical Oxygen Demand 3. TSS: Total Suspended Solids 4. NH ₃ -N: Ammonia-Nitrogen 5. mg/L: milligrams per Liter 6. ppd: pounds per day												

From 2003 through 2010, BOD loadings averaged 2,120 pounds per day (ppd). Based on an average of 5,000 EDUs served during this period, 0.42 pounds per day are contributed per EDU, which based on 2.58 persons per household, equates to 0.16 pounds per capita per day (ppcd). TSS loadings averaged 1,898 ppd, or approximately 0.38 ppd/EDU (0.15 ppcd). Per capita loadings for BOD and TSS are within the range for typical domestic wastewater, but on the high end of this range (Metcalf and Eddy, 2003).

Ammonia loadings from 2003 through 2010 averaged 302 ppd or approximately 0.06 ppd/EDU (0.023 ppcd), a value that exceeds the published unit loading factors for NH₃-N developed for municipal domestic wastewater. High total nitrogen loadings were confirmed by sampling conducted on the WWMF influent in 2010 for Total Kjeldahl Nitrogen (TKN), combined organic and ammonia nitrogen. Those results indicated a per capita loading of 0.043 ppcd, which also exceeded the published ranges of values.

Table 3-6 summarizes per capita loadings and wastewater strengths for comparison to published values. Average wastewater concentration of BOD, TSS, and NH₃-N were in the medium to strong range as expected for a system with low rates of infiltration. Per capita loadings for NH₃-N are unusually strong.

Table 3-6 Wastewater Management Facility Influent Composition MCSD Wastewater Management Facility							
	Unit Loadings (ppcd) ¹		Concentration (mg/L) ²				
	MCSD	Published	MCSD		Published		
	Average	Typical	Average	Maximum	Weak	Medium	High
BOD ³	0.16	0.18	257	360	110	220	400
TSS ⁴	0.15	0.20	227	520	100	220	350
NH ₃ -N ⁵	0.023	0.007	36	50	12	25	50
TKN ^{6,7}	0.043	0.027	63	78	20	40	85
1. ppcd: pounds per capita per day 2. mg/L: milligrams per Liter 3. BOD: Biochemical Oxygen Demand, Influent sampling 2003-2010 4. TSS: Total Suspended Solids, Influent sampling 2003-2010 5. NH ₃ -N: Ammonia-Nitrogen, Influent sampling 2003-2010 6. TKN Total Kjeldahl Nitrogen (Free ammonia and organic nitrogen combined) 7. Bi-weekly sampling from 1/2010-12/2010							

3.2.2 Constituents of Concern

The current constituents of concern include the following priority pollutants: 4,4'-Dichlorodiphenyltrichloroethane (DDT), bis(2-ethylhexyl)phthalate, and carbon tetrachloride. These constituents are monitored in compliance with the California Toxics Rule when the facility is discharging to the Mad River. Copper, lead, alpha-1,2,3,4,5,6-hexachlorocyclohexane (alpha-BHC) and 2,3,7,8 Tetrachlorobenzene-p-dioxin (TCDD) congeners were previous constituents of concern that are no longer subject to effluent limitations in the new permit. As discussed in more detail in

Section 5.1 new regulatory requirements for copper have been implemented based on adoption of a Water Effects Ratio (WER) for determining required discharge limitations, and MCSD is currently in compliance with those requirements.

Although it has been determined that copper concentrations in the WWMF effluent do not pose a risk to water quality, high influent concentrations of copper in the wastewater influent are a concern because of possible inhibitory effects on the nitrification process. A recent study recorded 50% inhibition of nitrification during batch tests with influent copper levels of 1 mg/L (WEF, 2002). Based on sampling conducted on the wastewater influent in 2009, influent concentrations of copper are below 0.14 mg/L, 90% of the time, and assuming a linear effect, possible nitrification inhibition would be approximately 10%.

A wide range of values is given in the literature for levels at which copper becomes inhibitory to nitrification, with threshold concentrations of 0.5 micrograms per Liter ($\mu\text{g/L}$) to 500 $\mu\text{g/L}$ cited (Metcalf and Eddy, 1991). If it is determined during pre-design that reduction of copper is necessary to preserve effective nitrification of the influent, pre-treatment in the form of precipitation or chelating agents would be recommended. Based on annual monitoring results provided by HBMWD, the observed concentrations of copper in the WWMF influent can be attributed due to the municipal drinking water, which is drawn from the Mad River (MCSD, 2011).

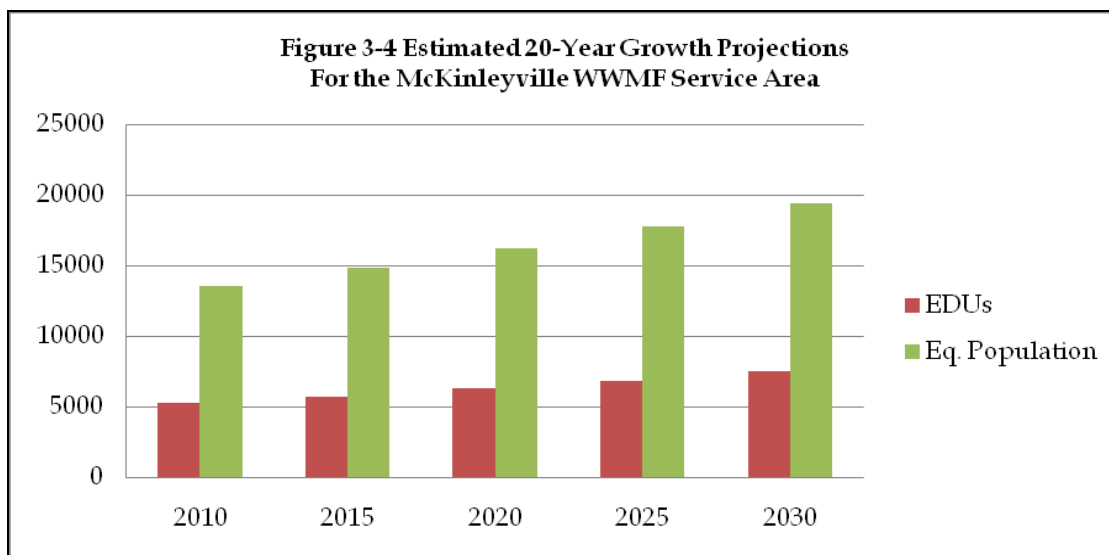
3.3 Flow and Loading Projections

Table 3-7 summarizes influent flow data from the MCSD WWMF and Table 3-8 summarizes the influent loading data.

Table 3-7 Flow Projections MCSD Wastewater Management Facility								
Year	2003-2010			2010	2015	2020	2025	2030
EDUs ¹	4,999			5,267	5,758	6,296	6,883	7,525
eq. population ²	12,897			13,589	14,857	16,243	17,758	19,415
Wastewater Flows	MGD ³	g/EDU/d ⁴	gpcd ⁵	MGD	MGD	MGD	MGD	MGD
Base Sanitary	0.830	166	64	0.874	0.956	1.045	1.143	1.249
Base I/I ⁶	0.079	16	6	0.083	0.091	0.099	0.109	0.119
ADWF ⁷	0.909	182	70	0.958	1.047	1.145	1.252	1.368
AWWF ⁸	1.119	224	87	1.179	1.289	1.409	1.541	1.684
AAF ⁹	1.022	204	79	1.068	1.168	1.277	1.396	1.526
MMDWF ¹⁰	1.063	213	82	1.120	1.224	1.339	1.464	1.600
MMWWF ¹¹	1.413	283	110	1.489	1.628	1.780	1.946	2.127
Peak Day	2.045	409	159	2.155	2.356	2.575	2.816	3.078
PIF ¹²	2.500	500	194	2.634	2.880	3.148	3.442	3.763
1. EDU: Equivalent Dwelling Units 2. eq population: Equivalent Population 3. MGD: Million Gallons per Day 4. g/EDU/d: gallons per Equivalent Dwelling Unit per day 5. gpcd: gallons per capita per day 6. I/I: Infiltration and Inflow 7. ADWF: Average Dry Weather Flow 8. AWWF: Average Wet Weather Flow 9. AAF: Average Annual Flow 10. MMDWF: Maximum Month Dry Weather Flow 11. MMWWF: Maximum Month Wet Weather Flow 12. PIF: Peak Instantaneous Flow								

Table 3-8 Loading Projections MCSD Wastewater Management Facility								
Year	2003-2010			2010	2015	2020	2025	2030
EDUs ¹	4,999			5,267	5,758	6,296	6,883	7,525
eq. population ²	12,897			13,589	14,857	16,243	17,758	19,415
Wastewater Loads	ppd ³	ppd/EDU ⁴	ppcd ⁵	ppd	ppd	ppd	ppd	ppd
Average Day BOD ⁶	2,120	0.42	0.16	2,234	2,442	2,670	2,919	3,191
Max Mo. BOD	3,035	0.61	0.24	3,198	3,496	3,822	4,179	4,569
Max Day BOD	4,111	0.82	0.32	4,331	4,736	5,177	5,660	6,188
Ave NFR ⁷	1,898	0.38	0.15	2,000	2,186	2,390	2,613	2,857
Max Mo. NFR	3,964	0.75	0.29	3,964	4,334	4,738	5,180	5,664
Max Day NFR	5,305	1.01	0.39	5,305	5,800	6,341	6,933	7,580
Average Day TKN ⁸	558	0.11	0.04	588	643	703	768	840
Max TKN	702	0.14	0.05	740	809	884	967	1,057
Max Day TKN	809	0.16	0.06	852	932	1,019	1,114	1,218
Average Day NH ₄ -N ⁹	302	0.06	0.02	318	348	380	416	455
Max. Mo NH ₄ -N	440	0.09	0.03	464	507	554	606	662
Max Day NH ₄ -N	450	0.09	0.03	474	518	567	620	677
1. EDU: Equivalent Dwelling Units 2. eq population: Equivalent Population 3. ppd: pounds per day 4. ppd/EDU: pounds per day per Equivalent Dwelling Unit 5. ppcd: pounds per capita per day 6. BOD: Biochemical Oxygen Demand 7. NFR: Non-Filterable Residue 8. TKN: Total Kjeldahl Nitrogen 9. NH ₄ -N: Ammonium-Nitrogen								

Projections are based on an alternative growth rate projection of 1.8% annual increase, cited as the probable growth rate projection in the McKinleyville Community Plan (Humboldt County, 2002). Figure 3-4 shows the estimated 20-year growth projections for the MCSD WWMF service area.



4.0 Existing Wastewater Facilities

4.1 Wastewater Collection System

This section of the facilities plan provides an overview of the sanitary sewer collection and conveyance systems serving MCSD. Information presented in this section is based on discussion with MCSD staff and our review of the following documents:

- Community Infrastructure and Services Technical Report (W&K, 2008)
- MCSD Sanitary Sewer Management Plan (FES, 2011)
- MCSD Municipal Services Review (LAFCo, 2009)
- MCSD Budget for the Fiscal Year Ending June 30, 2012 (MCSD, 2011)

4.1.1 System Description

MCSD maintains approximately 65 miles of sewer mains (MCSD, 2010). The collection system consists of approximately 63 miles of gravity sewer mains, 2 miles of pressure mains, 900 sanitary manholes, and five pump stations. Gravity sewer lines range in size from 6-inch lines to 24-inch lines, with the majority of the system (76%) comprised of 6-inch lines. An overview of the MCSD collection system was included as Figure 1-4. The community sewer collection and conveyance system is owned and operated by MCSD and services the sewer collection area shown in Figure 1-5.

MCSD maintains a radio telemetry system that allows all key facilities to be monitored constantly from the MCSD field office. The sewer facilities are connected to the computer system by radio telemetry. Upgrading of the system from land-based telephone lines to radio telemetry was started in 2003 and completed in 2009 (MCSD, 2011).

4.1.2 Lift Stations

Five lift stations have been constructed to convey wastewater from the collection system tributary areas to the MCSD WWMF. A summary evaluation of the lift stations is provided in Table 4-1. The maximum pump capacities reported in the analysis were determined by draw down testing performed in 2009 by SHN and additional testing completed by the District in 2011.

Average pumping rates and total pump capacity shown for each lift station were based on the data collected from the pump tests in 2009 and 2011. The process included calculating the incoming flow rate to the pump station using the time and height difference in water surface elevations noted during the pump tests. The average pumping rate was then determined by calculating the volume of effluent pumped out based on changes in the water surface elevation during the pump test and adding to that volume the amount of effluent that came into the wet well while the pump was running. The combined volume was divided by the pumping time to arrive at the average pump flow rates. Pump tests at each lift station were conducted individually for each pump and while running multiple pumps simultaneously.

**Table 4-1
Lift Station Summary
MCSD Wastewater Management Facility**

Description	PS #1	PS #2	PS #3	PS #4	PS #5
Name	B Street	Letz Lane	Kelly Street	Hiller Road	Fisher Road
System Type	Duplex, self-priming	Triplex, self-priming	Duplex, self-priming	Duplex, self-priming	Quad, flooded suction
Number of Pumps	2	3	2	2	4
Pump Type(s)	Centrifugal, non-clog	Centrifugal, non-clog	Centrifugal, non-clog	Centrifugal, non-clog	Centrifugal, non-clog
Pump Make(s)	Gorman Rupp	Gorman Rupp (All Pumps)	Gorman Rupp	Gorman Rupp	Worthington (All Pumps)
Pump Model(s)	2, T3A3B	1, T8A3B 2, T4A-5	2, T3A3B	2, T6A3B	2, 4MFV-11 2, 4MFV-15
Pump hp ¹	5	15 (P1 & P2) 50 (P3)	5	20	30 (P1 & P2) 100 (P3 & P4)
Pump Size(s)	3-inch	4-inch (P1 & P2) 8-inch (P3)	3-inch	6-inch	4-inch (P1 & P2) 8-inch (P3 & P4)
Motor Make(s)	Allis Chambers	Allis Chambers (P1) Baldor (P2) US Electric (P3)	Allis Chambers	General Electric	General Electric (All Pumps)
Motor Volts	240	480 (All Pumps)	240	240	480 (All Pumps)
Firm Capacity ²	182 gpm ³	673 gpm	125 gpm	836 gpm	1,614 gpm
Overflow point	Low spot @ wetwell	---	---	To Fisher @ 9.5	---
Auxiliary power type	35 kw Generator	125 kw Generator	Portable	Not Required (passive overflow)	170 kw Generator
Force main length	1,457 ft ⁽⁴⁾	1,716 ft	30 ft	1,298 ft	5,960 ft
Force main size	10-inch	10-inch	6-inch	12-inch	12-inch
Discharge manhole	Manhole at Park and A	Manhole at Murray	Manhole at Eucalyptus	Headworks	Headworks

1. hp: Horsepower
2. Firm capacity was estimated based on pump tests conducted in 2009 and 2011, and assumes the largest pump at each lift station is offline.
3. gpm: gallons per minute
4. ft: feet

Improvements to the lift stations over the last 10 years have been focused mainly on the Fisher Road pump station. In 2001, the Fisher lift station flow meter was upgraded. In 2002, the grinder/communiter at the Fisher lift Station was replaced. In 2005 a restoration project at the Fisher lift station was completed to rehabilitate wet well valves, doors, light fixtures and exterior facilities (MCSD, 2010). The Hiller Road lift station was also upgraded in 2001 to increase capacity.

4.1.3 Age and Condition

MCSD's wastewater collection system was installed in the mid-1980s and has been well maintained. District staff has placed an operational priority on investigating and monitoring I/I of groundwater and storm runoff into the collection system. Smoke testing of the collection system is completed periodically to test for leaks and misconnections (MCSD, 2010). Each winter, the District also monitors wet weather flows at various manhole locations and expends the necessary resources to reduce I/I during wet weather. Overall, the MCSD collection system experiences some of the lowest I/I rates in the County (W&K, 2008).

4.1.4 Assessment

4.1.4.1 Collection System

Detailed review of the collection system was last completed in 2004. Pipe replacement has been on hold pending further engineering analysis. Projected growth in McKinleyville raises questions about the adequacy of the collection system capacity. MCSD is investigating the potential impacts to the collection system capacity based on various development projections provided by the County. Further discussion of the collection system analysis developed for this effort is presented in Section 6.

Based on staff observations, and as demonstrated by preliminary model results, no surcharging occurs within the collection system under existing dry weather conditions. However there were previous deficiencies identified in the system including the capacity of the main trunk lines, such as the Thiel Avenue line under Hiller Park, and the Widow White Creek line under the freeway (W&K, 2008). The District has considered adding additional capacity either by addition of parallel lines or pulling and replacing the existing lines (W&K, 2008).

4.1.4.2 Lift Stations

The pumps at each lift station are the original pumps installed when the system was constructed. The age of the pumps at each lift station is a known deficiency, but there has been no known failure to date (MCSD, 2011). MCSD staff provides semi-annual maintenance for each pump including adjustments and rehabilitation of the pump volute, as necessary. There have been no motor upgrades since the Hiller lift station sheaves and motors were upgraded in 2001.

4.2 Wastewater Treatment

The MCSD WWMF is a facultative pond system followed by wetlands treatment and chlorine disinfection. The overall site plan for the existing treatment system is depicted schematically in Figure 1-3.

4.2.1 System Description

4.2.1.1 Headworks

Wastewater from the Hiller and Fisher lift stations is pumped into a splitter box at the head of the facultative pond system where downward opening slide gates split the flow between the primary treatment ponds. Metering is provided by flow meters on the two force mains.

There is no functional pre-treatment of the influent, screening, or grit removal prior to discharge to primary treatment Ponds 1A and 1B. There are two grinders located in the existing system—one grinder is located at the Fisher lift station and the second grinder at the treatment plant. The grinders reduce large inorganic solids to small particles so that they may pass through the treatment system. Allowing the inorganic solids to remain in the flow stream increases the sludge volumes due to the inorganic solids not breaking down during treatment.

4.2.1.2 Facultative Pond System

Parallel primary treatment Ponds 1A and 1B are followed by secondary Ponds 2 and 3, generally operated in series. Supplemental aeration is supplied by 12, 5 horsepower (hp) turbine aerators: five each in Pond 1A and Pond 1B and two in Pond 2.

4.2.1.3 Wetlands Treatment

The secondary ponds are followed by two wetlands treatment cells operated in series. Wetlands 4 and 5 were constructed in 2005, to provide enhanced BOD and nutrient removal.

4.2.1.4 Disinfection

Secondary effluent from the wetlands treatment cells discharges to the chlorine contact basin. Following disinfection, effluent is gravity-fed to the Mad River for discharge, applied at one of four land reclamation sites, and/or discharged to the percolation ponds. During the period of discharge to the Mad River effluent is dechlorinated prior to discharge using sulfur dioxide injected in the effluent channel of the chlorine contact basin.

4.2.2 Secondary Treatment Capacity

The following analysis of secondary treatment capacity is based on the treatment system's capacity to treat organic loadings in the form of BOD. Generally, suspended solids or Non-Filterable Residue (NFR) can be assumed to track fairly closely with BOD in a pond system with the exception that excessive algae growth can lead to high levels of suspended solids in the pond effluent.

4.2.2.1 Facultative Ponds

Facultative ponds are designed using empirically derived surface loading rates coupled with a more detailed kinetic analysis, such as the equation developed by Wherner-Wilhelm to determine

required detention times. Facultative lagoons with supplemental surface aeration (such as, the ones at MCSD) have a greater allowable surface or areal loading rate and the design may be controlled by the required detention time as calculated with the Wherner-Wilhelm equation.

Table 4-2 summarizes surface or areal loading on the MCSD facultative pond system. Typical loading rates for the facultative pond with supplemental aeration are 50-180 pounds/acre/day (Metcalf and Eddy, 1991). The pond loading on the MCSD ponds during the maximum month currently exceeds typical values and in 2030, average BOD loading will exceed the typical range of values.

Table 4-2 Areal BOD¹ Loading Rates MCSD Wastewater Management Facility									
Area	Acres	2010				2030			
		Average		Max Month		Average		Max Month	
		ppd ²	ppd/ac ³	ppd	ppd/ac	ppd	ppd/ac	ppd	ppd/ac
Ponds 1A & 1B	11.2	2,234	199	4,433	396	3,191	285	6,673	596
Combined ⁴	16.1	2,234	139	4,433	275	3,191	198	6,673	414
1. BOD: Biochemical Oxygen Demand 2. ppd: pounds per day 3. ppd/ac: pounds per day per acre 4. Ponds 1A, 1B, 2, and 3									

The first order removal rate equation developed by Wherner-Wilhelm is used to predict BOD removal rates based on available detention time and temperature assuming a flow through pattern between plug-flow and complete mix. Table 4-3 summarizes the results of an analysis showing detention times and expected removal rate.

Table 4-3 Theoretical BOD¹ Removal Rates (Wherner-Wilhelm) MCSD Wastewater Facilities Plan												
Ponds	2010						2030					
	MMDWF ² = 1.1 MGD ³			MMWWF ⁴ = 1.5 MGD			MMDWF = 1.6 MGD			MMWWF = 2.1 MGD		
	DT ⁵	KT ⁶	Re ⁷	DT	KT	Re	DT	KT	Re	DT	KT	Re
	days		%	days		%	days		%	days		%
P1	20.1	4.02	87	15.2	2.13	73	14.10	2.82	80	10.6	1.48	65
P2	3.6	0.72	NC ⁸	2.7	0.38	NC	2.50	0.50	NC	1.9	0.27	NC
P3	3.8	0.76	NC	2.8	0.39	NC	2.70	0.54	NC	2	0.28	NC
All	27.5	5.50	90	20.7	2.90	80	19.3	2.70	79	14.5	2.03	72
1. BOD: Biochemical Oxygen Demand 2. MMDWF: Maximum Month Dry Weather Flow 3. MGD: Million Gallons per Day 4. MMWWF: Maximum Month Wet Weather Flow 5. DT: Detention Time 6. KT: Removal Rate--First order rate constant for BOD removal 0.25 d-1 corrected for temperature. Winter k=0.14 d-1 Summer = 0.20 d-1 7. Re: Percent Removal 8. NC: Not Calculated												

The analysis is presented for maximum month flows, which define the limiting conditions. Higher BOD removal rates are to be expected during low flows and higher temperatures. For example, theoretical rates at ADWF and 18.8 °C range from 80 to 90%.

Based on the analysis, the majority of the BOD removal is expected to occur in Ponds 1A and 1B. Theoretical predictions were comparable to BOD removal rates recorded in the MCSD WWMF capacity study (OLA, December 2000). During this study, conducted in the spring and summer of 2000, BOD removal in Ponds 1A and 1B ranged from 50 to 85%. Overall, BOD removal rates of 78 to 90% recorded during the capacity study were also comparable to the theoretical predictions.

Table 4-4 presents the results of theoretical BOD removal rates based on temperature and detention time applied to average influent BOD concentrations. The current NPDES permit requires that the facility meets an average monthly limit of 45 mg/L BOD year round. The loading analysis indicates that the secondary ponds are at the limits of their capacity to provide the required BOD removal reliably without additional BOD removal in the wetlands during the winter months.

Table 4-4 Theoretical BOD¹ Removal Rates Based on Temperature and Detention Time MCSD Wastewater Management Facility											
2010						2030					
MMDWF ² = 1.1 MGD ³			MMWWF ⁴ = 1.5 MGD			MMDWF = 1.6 MGD			MMWWF = 2.1 MGD		
Infl. ⁵	Re. ⁶	Effl. ⁷	Infl.	Re.	Effl.	Infl.	Re.	Effl.	Infl.	Re.	Effl.
mg/L ⁷	%	mg/L	mg/L	%	mg/L	mg/L	%	mg/L	mg/L	%	mg/L
272	90%	27	244	80%	49	272	79%	57	244	72%	68
1. BOD: Biochemical Oxygen Demand						5. Infl.: Influent					
2. MMDWF: Maximum Month Dry Weather Flow						6. Re.: Percent Removal					
3. MGD: Million Gallons per Day						7. Effl.: Effluent					
4. MMWWF: Maximum Month Wet Weather Flow											

4.2.2.2 Wetlands

Wetland organic loading rates based on the expected effluent quality from the facultative pond system are presented in Table 4-5. Loading rates for the approximate 6 acres of existing wetlands range from 43 ppd/ac at current flows to 226 ppd/ac at future flows and loadings.

Table 4-5 Organic Loading Rate on Wetlands Cells MCSD Wastewater Management Facility											
2010						2030					
MMDWF ¹ = 1.1 MGD ²			MMWWF ³ = 1.5 MGD			MMDWF = 1.6 MGD			MMWWF = 2.1 MGD		
FS Re. ⁴	Loading		FS Re.	Loading		FS Re.	Loading		FS Re.	Loading	
%	ppd ⁵	ppd/ac ⁶	%	ppd	ppd/ac	%	ppd	ppd/ac	%	ppd	ppd/ac
90	254	43	80	676	114	79	744	126	72	1335	226
1. MMDWF: Maximum Month Dry Weather Flow						4. FS Re.: Removal in facultative system preceding wetlands					
2. MGD: million gallons per day						5. ppd: pounds per day					
3. MMWWF: Maximum Month Wet Weather Flow						6. ppd/ac: pounds per day per acre					

A range of loading rates is presented in the literature for areal loading on wetlands. There is general agreement that loading rates should not exceed 100 ppd/ac if aerobic conditions are to be maintained near the surface and odors minimized (Crites & Tchobanoglous, 1998). Typical BOD loading rates on wetlands are in the range of 50-70 ppd/ac (Tchobanoglous, 1987). The EPA design manual for constructed wetlands treatment (EPA, 2000,) is more conservative in its approach, stating that organic loading rates in the range of 10 to 25 pounds BOD/acre/day to free water surface wetlands have been shown to meet secondary effluent standards of 30 mg/L BOD and TSS effectively. Even though the current NPDES permit does not require that the MCSD WWMF meet secondary standards of 30 mg/L for both BOD and TSS, the organic loading rates on the wetlands are high enough to call into doubt the ability of the combined facultative ponds and wetlands system to meet the permit requirements of 45 mg/L BOD and TSS reliably, especially as loadings increase.

Enlarging the wetland cells to provide secondary treatment and enhanced treatment for nutrient removal is discussed as an alternative in the evaluation of treatment alternatives in Section 6. As part of that analysis, biological rate constants for wetlands treatment are discussed and used to provide specific criteria for how large treatment wetlands would need to be to meet the current permit and/or future secondary requirements.

4.2.2.3 Nutrient Removal

Currently, removal rates for ammonia in the facultative pond system range from 20 to 50%. The mechanisms of ammonia removal in pond systems and reasons for its wide range are discussed in more detail in subsequent sections. The values presented here are used to provide an estimate of nitrogen loading on the wetland cells as part of a discussion regarding the capacity of wetland cells to achieve required removal rates reliably.

According to the EPA design manual for wetlands, the maximum Total Nitrogen (TN) loadings on a free water surface wetland to sustain an effluent TN of less than 10, can conservatively be set to 5 kilograms per hectare (kg/ha) (4.5 ppd/ac) (EPA, 2000). At a removal rate of 25%, the lower end of the range for the facultative pond system, ammonia loading on wetlands Ponds 4 and 5 is approximately 231 ppd or 39 ppd/ac. Under these conditions, current loading on the wetlands is seven times the recommended loading on a free water surface wetland designed to achieve land application standards.

4.2.3 Disinfection System

Treated effluent is chlorinated prior to disposal using chlorine gas. When the WWMF is discharging to the Mad River, effluent is dechlorinated at the end of the chlorine contact basin using sulfur dioxide.

4.2.3.1 Chlorination

The WWMF is equipped with two chlorinators (one with a capacity to feed 200 ppd, and one with a capacity to feed 400 ppd) and one sulfonator, also with a 200 ppd capacity. Table 4-6 summarizes chlorine usage from 2010 data.

Table 4-6
Chlorine and Sulfur Dioxide Usage
MCSD Wastewater Management Facility

	Chlorine Usage								Sulfur Dioxide Usage			
	Dosage				Residual		Demand		Dosage			
	Ave	Max	Ave	Max	Ave	Max	Ave	Max	Ave	Max	Ave	Max
	ppd ¹	ppd	mg/L ²	mg/L	mg/L	mg/L	mg/L	mg/L	ppd	ppd	mg/L	mg/L
January	95	194	11	59	3	6	9	58	95	194	11	59
February	156	200	17	29	2	5	15	28	35	61	4	6
March	174	203	17	20	1	4	15	19	34	46	4	33
April	156	193	15	19	2	5	13	19	35	56	3	5
May	140	200	14	21	2	11	13	20	17	56	2	5
June	105	197	11	22	3	8	9	20	NA ³	NA	NA	NA
July	93	144	12	3	3	8	10	17	NA	NA	NA	NA
August	68	119	9	3	3	6	7	14	NA	NA	NA	NA
September	108	156	15	3	3	9	12	21	NA	NA	NA	NA
October	69	108	9	4	4	6	6	11	NA	NA	NA	NA
November	99	168	11	3	3	5	9	19	29	NA	4	NA
December	144	209	14	2	2	4	12	18	35	3	3	6
Average	117	---	13	---	3	---	11	---	40	---	4	---
Maximum	---	209	---	59	---	11	---	58	---	194	---	59

1. ppd: pounds per day
2. mg/L: milligrams per Liter
3. NA: Not Applicable

In 2010, daily usage averaged 117 ppd but exceeded 200 ppd several times.

Chlorine demand is the amount of chlorine required to maintain the required disinfection residual in the wastewater effluent. Chlorine demand at the MCSD facility ranges from 6 to 60 mg/L. Periods of high chlorine demand can be attributed to high levels of suspended solids and algae and build-up of hydrogen sulfide in the treatment wetlands. These periods of high demand result in an average demand that is more than twice that expected for secondary treatment using activated sludge (White, 1992).

4.2.3.2 Contact Basin

The serpentine plug flow configuration of the contact basin consists of 13 channels with an average length of 42 feet and a maximum depth of 9.9 feet providing a volume of approximately 197,000 gallons. Because some degree of short-circuiting is inherent in any plug flow regime, actual detention time will be less than the theoretical detention time based on total volume.

To measure actual detention time at the MCSD facility, a dye test was conducted in June 2000 (OLA, December 2000). The results of this test indicated that at high flows, the contact basin has an active volume of 106,500 gallons and could provide the required detention time of 30 minutes at a flow of 5.11 MGD (3,550 gallons per minute [gpm]).

4.2.3.3 Uniform Fire Code

At a minimum, Article 80 of the Uniform Fire Code requires facilities using chlorine gas and not equipped with scrubber systems to have the following controls:

- Approved containment vessels or containment systems
- Protected valve outlets
- Gas detection system
- Approved automatic-closing fail-safe valve

The gas chlorination at MCSD has been inspected by the Fire Marshal and determined to be compliance with the Uniform Fire Code. The gas cylinders are contained in a chlorine room equipped with gas monitors and the installation of automatic closing fail-safe valves has been budgeted for 2011.

4.2.4 Treatment System Performance

The facultative ponds are followed by treatment wetlands that were designed to treat secondary effluent. This section provides a detailed analysis of system performance in terms of the theoretical capacity, and a discussion of whether there are improvements and or operational changes that can improve system compliance and reliability.

4.2.4.1 BOD and NFR Removal

Figures 4-1 and 4-2 provide an overview of the monthly BOD and NFR removal achieved in the treatment system; respectively. Figures 4-3 and 4-4 show the 2010 monthly seasonal variation and annual average variation in BOD concentrations by system component, respectively. The seasonal variation in pond performance indicates declining performance in summer months. This deterioration in performance may be related to increased algae growth. Although algae are part of the treatment in any facultative system, large blooms can cause large variations in pH, and Dissolved Oxygen (DO), which can be detrimental to the process. The algae uses up carbon dioxide during the day and can cause increased pH. The result is usually an increase in NFR, and BOD may increase when algae sinks, contributing to eutrophic conditions at depth in the ponds.

Figure 4-1
BOD Removal

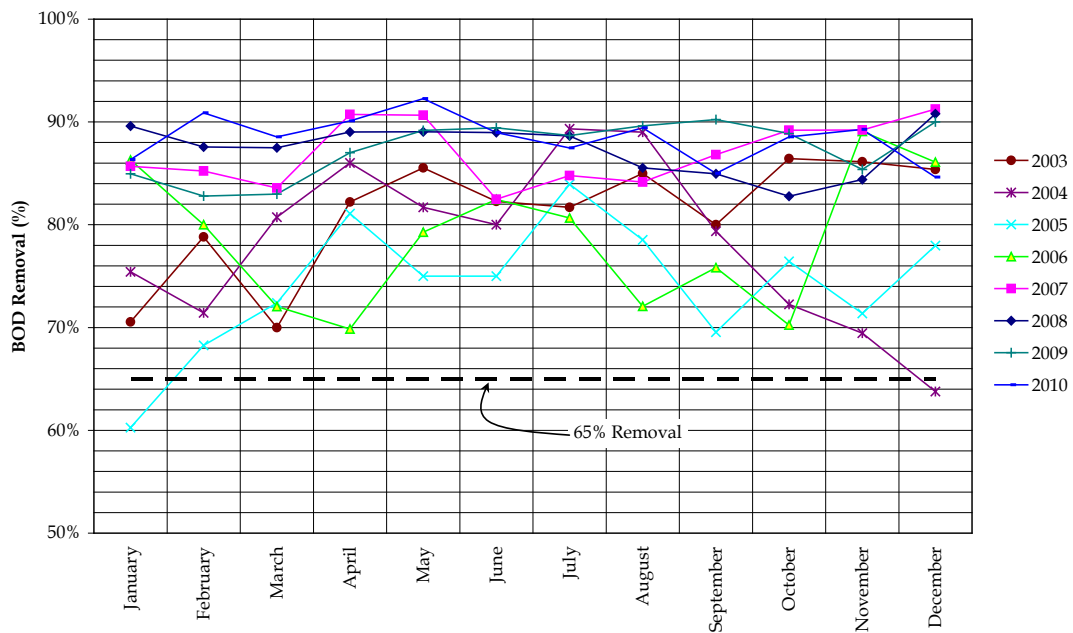


Figure 4-2
NFR Removal

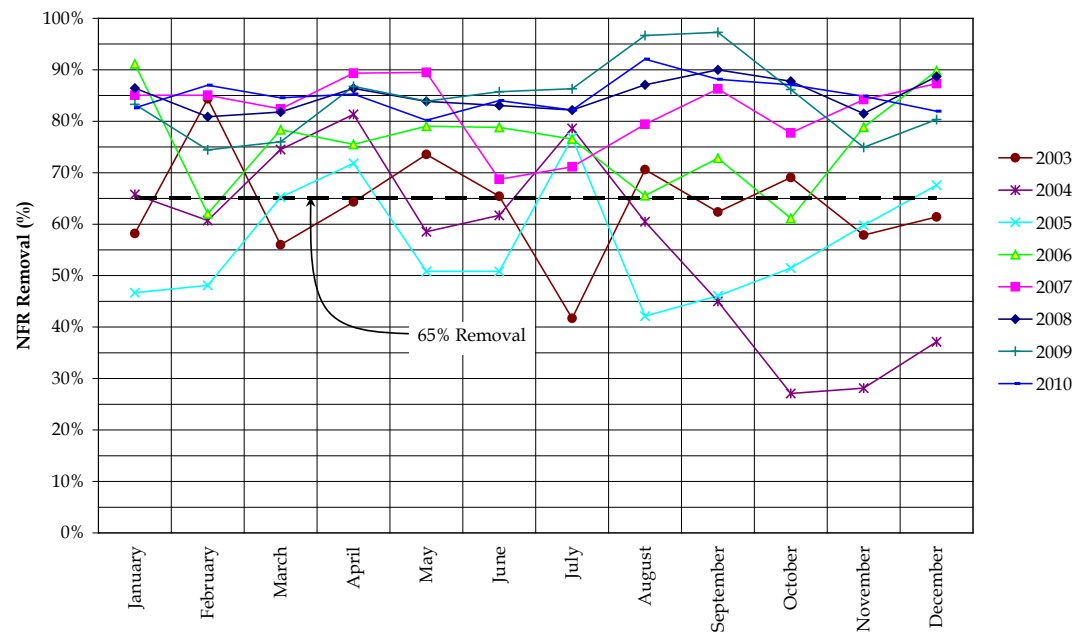


Figure 4-3
Facultative Pond Monthly Average BOD (2010)

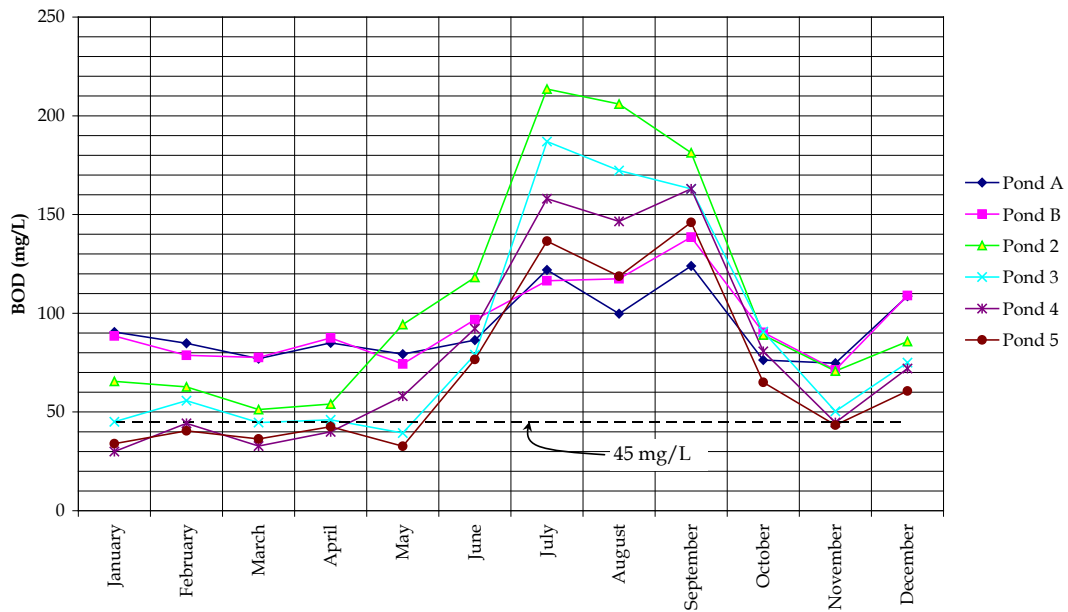
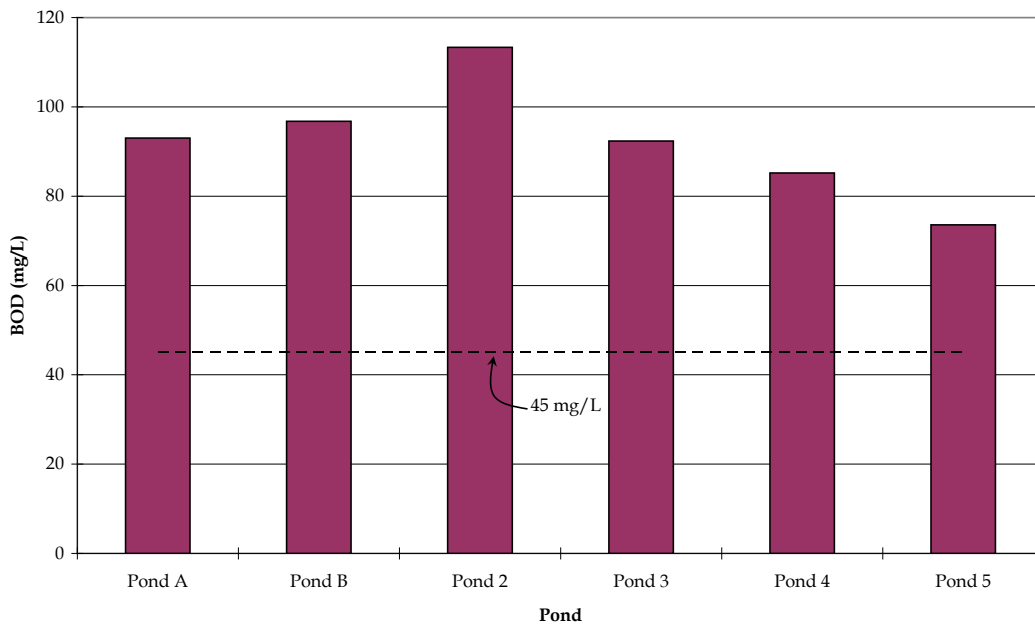


Figure 4-4
Facultative Pond Annual Average BOD (2010)

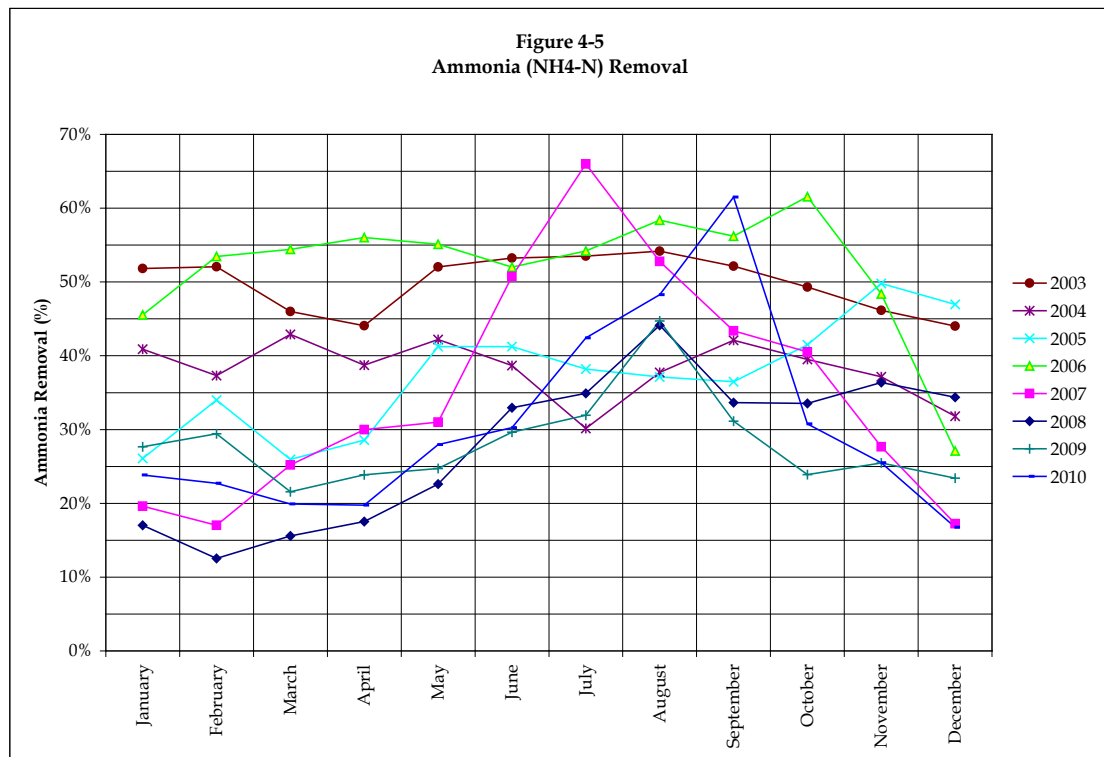


4.2.4.2 Ammonia Removal

Nitrogen removal in facultative ponds is positively correlated with increased temperature, pH, and detention time. It occurs principally through the following processes:

- Gaseous ammonia stripping to atmosphere
- Ammonia assimilation in algal biomass
- Nitrate assimilation in algae
- Biological nitrification-denitrification

Figure 4-5 provides an overview of the monthly ammonia removal achieved in the treatment system from 2003 through 2010. Figure 4-6 shows the monthly range in TKN removal for 2010.



4.2.4.3 Nitrogen Removal in Facultative Ponds

During the winter and spring, ammonium-nitrogen (NH₄-N) removal in the facultative pond system is reduced by colder temperatures and shorter detention times. Effluent NH₄-N levels have been a concern because of the potential toxicity of unionized ammonia when the MCSD WWMF is discharging to the Mad River. SHN analyzed ammonia and total nitrogen removal in the secondary pond system during the 2010 wet weather discharge period, to determine if the ammonia removal process was meeting theoretical expectations based on temperature, pH, and detention time. Table 4-7 summarizes the results.

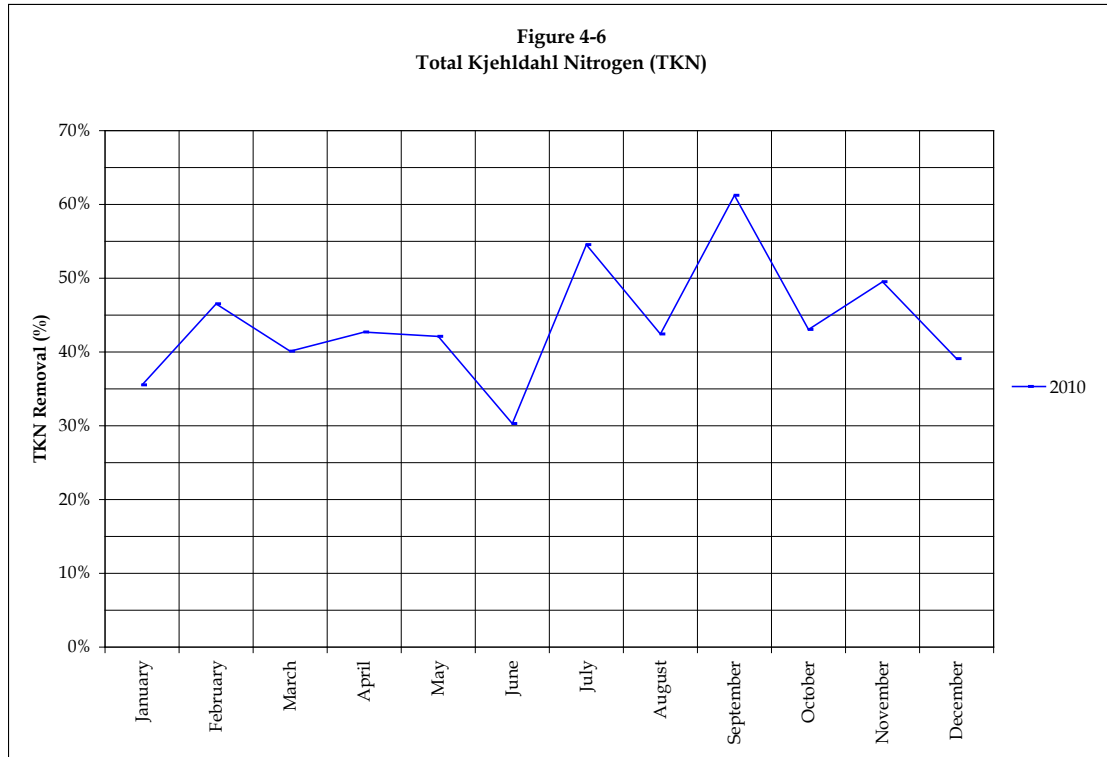


Table 4-7 Nitrogen Removal, 2010 MCSD Wastewater Management Facility									
	NH ₄ -N ⁽¹⁾				TKN ²				
	Influent	Effluent	Removal		Influent	Effluent	Removal		
	mg/L ³	mg/L	Observed	Predicted ¹	mg/L	mg/L	Observed	Predicted	Predicted
January	36	28	25%	39%	64	41	47%	30%	25%
February	40	30	23%	40%	72	39	40%	32%	28%
March	36	29	19%	33%	53	32	43%	28%	25%
April	34	27	19%	32%	58	33	42%	28%	24%
1. NH ₄ -N: Ammonium-Nitrogen 2. TKN: Total Kjeldahl Nitrogen 3. mg/L: milligrams per Liter									

It is important to note that the observed ammonia removal rates noted in Table 4-7 underestimate total ammonia removal in the ponds because the contribution from influent organic nitrogen is not included in the calculation. An annual average of 20 mg/L of organic nitrogen is removed in the facultative pond system; this organic nitrogen is first converted to NH₄-N. If the ammonia removal rates in Table 4-7 are recalculated to include the contribution from influent organic nitrogen, ammonia removal averages 47%. In general, total nitrogen removal in the ponds exceeds theoretical expectations as indicated by the values for TKN removal.

4.2.4.4 Nitrogen Removal in Wetlands

The treatment wetlands came on-line in 2006. Although some increase in overall performance has been noted, the expected increase in nutrient removal has not been achieved and effluent ammonia levels remain high.

The treatment wetlands, which receive secondary effluent from Ponds 2 and 3, are shallow and planted with emergent vegetation, which is best at removing nitrogen in the form of nitrates either by denitrification (which converts nitrates to nitrogen gas [N₂] thereby removing it from the system) or plant uptake. However, the nitrogen from the secondary ponds is almost entirely in the form of ammonia.

The wetlands system lacks a method of reliable nitrification. Wetland Pond 4 was designed with deeper cells for nitrification, but the facility has not been successful in growing the submerged plants best suited for conversion of ammonia. The District is conducting a pilot test in Pond 3 using Submerged Aquatic Vegetation (SAV) species to treat effluent from Ponds 1 and 2.

4.2.5 Permit Compliance

According to the 2010 annual report for the MCSD WWMF (MCSD, 2011) the treatment system was in compliance with BOD, NFR, settleable solids, chlorine residual, and nitrate as nitrogen limitations during 2010. Coliform monthly median and daily maximum concentrations were also in compliance with the exception of the values recorded in May 2010.

Priority pollutant testing results for copper, lead, alpha-BHC, 4,4'-DDT, bis(2-ethylhexyl)phthalate and 2,3,7,8-TCDD equivalents were in compliance with applicable limitations with the exception of the 4,4'-DDT concentrations recorded in February and March 2010. With regard to the 4,4'-DDT excursions, the District noted that the February and March results for 4,4'-DDT indicated an intermittent appearance of that constituent in treated effluent. During the sanitary survey conducted in 2009, no generator of that constituent was identified (FES, 2009). All forms of DDT are currently illegal to purchase or sell and it has been difficult to identify the source of the pollutant given that the appearance of the pollutant is intermittent. The District is continuing to investigate and will (hopefully) eliminate the source as the solution to the intermittent excursions.

Acute and chronic toxicity monitoring was conducted in 2010 and acute toxicity testing results were in compliance with designated limitations during 2010. The chronic toxicity monitoring indicated that the test results for 2010 exceeded the chronic toxicity trigger of 1 toxicity unit (TUC). Similar results were recorded during the chronic toxicity testing completed in 2009. In response to the 2009 chronic toxicity monitoring results, the District completed a Toxicity Reduction Evaluation (TRE) that concluded that the TUC exceedances were due to ammonia toxicity in the effluent (SHN, 2010).

The main area of concern noted in the 2010 annual report was the presence of high ammonia concentrations in the effluent. Although the current permit does not directly limit ammonia in effluent discharges, the District anticipates limits will be established in the next permit cycle.

4.3 Disposal System

4.3.1 Mad River Discharge

During the discharge period, October 1 through May 14, and when the flow in the river is greater than 200 cubic feet per second (cfs), treated wastewater effluent is discharged to the Mad River. The existing outfall pipe for the Mad River Discharge is located at the Hammond Trail Bridge crossing on the Mad River.

4.3.2 Percolation Ponds

During the discharge prohibition period, May 15 through September 30, effluent is discharged to the percolation ponds and/or to land for reclamation. The percolation ponds include two separate basins that are alternated in use. The use of the percolation ponds for effluent disposal is allowed under the current permit; however, the RWQCB has indicated that future discharge permits may limit this use. The District is currently in the process of studying other disposal alternatives to the percolation pond discharge to comply with the Bay and Estuaries Policies and the Basin Plan discharge prohibitions for summer discharge of treated wastewater effluent.

4.4 Land Reclamation System

4.4.1 Existing Reclamation System

MCSD reclaims wastewater effluent at the Lower Fisher Ranch, Upper Fisher Ranch, the Hiller Parcel, and the Pialorsi Ranch. Land reclamation locations are shown on Figure 1-2. A reserve reclamation area is also shown Figure 1-2 for the Pialorsi Ranch property located east of Fisher Road. Although this area is not currently used for reclamation, recent discussions (2011) with the owner have indicated future use of this area for reclamation may be considered.

The Fisher and Pialorsi Ranches are located south of School Road and west of Fisher Road in McKinleyville. Wastewater effluent is also reclaimed for irrigation of storm water wetlands and a forested area at Hiller Park in McKinleyville during the dry months of the year. For the purpose of this facilities plan, the analysis of wastewater loading on the MCSD wastewater effluent reclamation areas was limited to the Fisher Ranch and Pialorsi Ranch irrigation areas, where the majority of reclamation occurs. Reference to the "Upper" and "Lower" Fisher Ranch is used to differentiate the upper terrace, where both flood irrigation and spray irrigation operations occur, from the lower floodplain at the toe of the hill slope, where spray irrigation is applied. The Upper Fisher Ranch consists of approximately 33 acres, 28 of which are used for reclamation. Wastewater effluent is applied to approximately 19 acres through flood irrigation and to 9 acres by spray irrigation. The Lower Fisher Ranch consists of approximately 45 acres and the Pialorsi Ranch has approximately 35 acres available for irrigation.

Based on data collected from 2008 through 2010, the Upper Fisher Ranch received 25 to 29% of the annual effluent discharge, whereas the Lower Fisher and Pialorsi Ranches received zero to 2% and 4 to 7% of the annual discharge, respectively.

4.4.2 Capacity

To evaluate the capacity of the reclamation area based on the irrigation pattern currently used by MCSD to reclaim wastewater effluent, the following were addressed:

- Hydrologic Properties of Soil
- Climate Data
- Irrigation Season
- Water Balance

4.4.2.1 Hydrologic Soil Properties

The Upper Fisher Ranch soils have been defined as coarse loam with an observed rooting depth of 36 to 45 inches (NRCS, 2010). The Lower Fisher Ranch soils have been defined as the fine silt Arlynda soil series, which was observed with rooting depths of 22 to 41 inches (NRCS, 2010). These soil qualities are used to estimate soil porosity, field capacity, and the Available Water Holding Capacity (AWHC), which are summarized in Table 4-8. AWHC is a measure of the total amount of water stored in the soil that is available to the plant, or the capacity of the soil. Field capacity is the upper limit of stored water in the soil once free drainage has occurred. When a soil is at field capacity, the soil reservoir is completely full. Hydrologic soil properties are used to develop irrigation schedules to maximize crop production by providing sufficient water supply and to reduce surplus runoff.

Table 4-8 Hydrologic Soil Properties for MCSD Reclamation Areas MCSD Wastewater Management Facility		
Soil Properties	Upper Fisher Ranch	Lower Fisher Ranch
Soil Class	Coarse Loam ¹	Fine Silt ¹
Porosity	46% ²	47% ²
Field Capacity	24% ²	28% ²
Wilting Point	10% ²	15% ²
Rooting Depth	36-45 inches ¹	22-41 inches ¹
AWHC ³ within the Root Zone	5-7 inches	4-8 inches
1. Source: NRCS soil series descriptions for Arcata and Arlynda soils 2. Source: Dunne and Leopold, 1978 3. AWHC: Available Water Holding Capacity		

4.4.2.2 Climate Data

Monthly average precipitation at the Eureka Woodley Island weather station for the period of record (1948 through 2010) has been summarized by the Western Regional Climate Center (WRCC, 2011). Over 90% of total rainfall occurs from October through April, which is considered the wet season.

Evapotranspiration (ET) is the loss of water to the atmosphere by the combined processes of evaporation (from soil and plant surfaces) and transpiration (from plant tissues). Reference ET (ET₀) is the ET rate of a reference crop, typically a standardized grass surface. Using the California

Irrigation Management Information System reference map of ET₀ zones in California, McKinleyville is within Zone 1 (coastal plains) characterized by dense fog (CIMIS, 1999). Crop evapotranspiration (ET_C) is a measure of the plant transpiration plus the soil surface evaporation. The ET_C rate is estimated as the product of a crop coefficient and ET₀. The crop coefficient for hay grass ranges from 0.6 near dormancy to 0.95 during the period of maximum growth, with an average of 0.82 during the growing season. The climate data used for the water balance is shown in Table 4-9.

Table 4-9 Climate Data for MCSD Reclamation Areas MCSD Wastewater Management Facility												
Parameter	Wet Season				Dry Season					Wet Season		
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Precipitation (inches) ¹	6.78	5.38	5.24	3.05	1.69	0.65	0.14	0.33	0.75	2.65	5.63	7.11
ET ₀ (inches) ²	0.93	1.40	2.48	3.30	4.03	4.50	4.65	4.03	3.30	2.48	1.20	0.62
K _C ³	0.60	0.60	0.73	0.91	0.95	0.95	0.87	0.80	0.80	0.73	0.65	0.60
ET _C (inches) ⁴	0.56	0.84	1.81	3.01	3.83	4.28	4.05	3.22	2.64	1.81	0.78	0.37
Potential storage	---	---	---	---	2.22	3.59	3.86	2.86	1.77	---	---	---
1. Eureka at Woodley Island for period of record 1949-2009 (WRCC, 2010) 2. ET ₀ : Reference evapotranspiration (CIMIS, 2010) 3. K _C : Crop coefficient (BLM, 2010) 4. ET _C : Crop evapotranspiration												

4.4.2.3 Irrigation Season

The irrigation season (dry season) is defined as the months where the monthly ET_C rate is greater than the monthly precipitation rate. As shown in Table 4-9, the irrigation season is from May through September. The difference between the precipitation rate and the ET_C rate during the irrigation season is a measure of the potential water loss or conversely, the potential storage available in the soil. The accumulated potential available storage in the soil during the irrigation season is 14.3 inches.

4.4.2.4 Irrigation Water Balance

The goal of a water balance calculation is to determine if the wastewater effluent irrigation is applied at reclamation rates to the application areas. The following assumptions were made:

1. Irrigation rates are based on average effluent distribution data from 2008 through 2010.
2. There are 165 week days (available irrigation days) from May 15–December 31.
3. Monthly effluent discharge volumes applied to irrigation areas were distributed equally to available irrigation days per month.
4. Daily effluent discharge volumes applied to irrigation areas were distributed equally over 18 hours per day.
5. The irrigation season begins with the top 24 inches of the root zone at field capacity.
6. The irrigation efficiency is 75%.
7. Permeability is controlled by the most restrictive soil horizons within the soil layer.

Management Allowable Depletion (MAD) is the percent of the AWHC that an irrigator will allow the crop to deplete before irrigating. Typically, depending on the crop and soil type, a MAD above 50% results in stress to the crop and yield reduction. The results of the water balance for each reclamation area are summarized in Table 4-10. Negative MAD values are representative of surplus water conditions (no soil moisture deficit) and may indicate periods when water ponds or drains from the soil layer toward the groundwater.

4.1.2.5 Agronomic Loading Rates

The agronomic rate is the amount of nutrients that can be applied to a specific crop within an appropriate period. Applying wastewater at an agronomic rate will supply a plant with the amount of nitrogen it demands while minimizing the amount of nitrogen that is released below the root zone and/or provide the appropriate amount of other plant nutrients to promote plant growth. Annual nitrogen loading per acre was estimated, assuming that average monthly wastewater effluent TKN concentrations will be used to grow hay grass on the Upper Fisher, Lower Fisher, and Pialorsi Ranches, which provide 28, 45, and 35 acres of land for reclamation, respectively. Data worksheets showing the nutrient loading calculations are included in Appendix E.

A comparison of the recommended nitrogen loading rates (approximately 170 pounds per acre, based on current crop cover) with the reported TKN loading rates from 2010 indicates that the Upper Fisher, Lower Fisher, and Pialorsi Ranches received nitrogen at approximately 683%, 0%, and 82% of agronomic rates, respectively. Based on nitrogen loading estimates from 2010, approximately 217 acres are required to balance effluent nitrogen loading with crop agronomic rates of uptake equally; the existing reclamation area supplies 50% of that target if the effluent distribution among irrigation areas were in proportion with available reclamation areas.

Plant Available Nitrogen (PAN) is the sum of available inorganic nitrogen and the percentage of organic nitrogen that mineralizes into ammonia. The mineralization of organic nitrogen typically is not a major concern for municipal wastewater land treatment systems, with the exception of systems receiving effluent containing significant concentrations of algae (EPA, 2006). Organic nitrogen concentrations calculated from the 2010 MCSD effluent data accounted for 21% of the total TKN annual loading; therefore, it is included in the nitrogen loading budget. Realistically, PAN would be sourced from the proportion of ammonia that was not volatilized, the proportion of organic nitrogen that was mineralized, and nitrate.

Treated wastewater contains many essential nutrients; however, if ratios are inadequate, nutrient management should be employed. Optimum nutrient ratios to ensure proper nutrient use are generally 4 parts Nitrogen to 1 part Phosphorous to 2 parts Potassium (4N:1P:2K). The WWMF effluent is sampled for nitrogen; however, the phosphorus and potassium concentrations are not characterized as part of their monitoring program. To ensure that optimum nutrient uptake is occurring, effluent phosphorus and potassium are recommended to be included as part of the on-going monitoring program for WWMF effluent discharges.

Table 4-10
Water Balance for Existing Irrigation Practices at Reclamation Areas¹
MCSD Wastewater Management Facility

Month	Monthly Average Precipitation (in/day)	Monthly Average ET _c ² (in/day)	Assumed Irrigation Days per Month (days)	Assumed Irrigation Duration (hours/day)	Upper Fisher Ranch		Lower Fisher Ranch		Pialorsi Ranch	
					Monthly Average Application Rate (in/day)	Monthly Average MAD ² (%)	Monthly Average Application Rate (in/day)	Monthly Average MAD (%)	Monthly Average Application Rate (in/day)	Monthly Average MAD (%)
May	0.05	0.12	12	18	0.18	-9%	0.00	16%	0.02	14%
June	0.02	0.14	22	18	0.46	-150%	0.00	57%	0.09	31%
July	0.00	0.13	21	18	0.38	-307%	0.01	110%	0.10	43%
August	0.01	0.10	23	18	0.40	-458%	0.02	154%	0.10	47%
September	0.03	0.09	22	18	0.40	-622%	0.02	184%	0.11	40%
October	0.09	0.06	21	18	0.51	-858%	0.03	181%	0.06	13%
November	0.19	0.03	22	18	0.68	-1231%	0.02	127%	0.05	-55%
December	0.23	0.01	22	18	0.27	-1573%	0.00	37%	0.01	-154%

1. Based on average effluent distribution data from 2008 through 2010

2. ET_c: Crop evapotranspiration

3. MAD: Management Allowable Depletion