DONNER SUMMIT
Public Utility District




Wastewater Facilities Plan

May 2010

Prepared for
Donner Summit
Public Utility District

3875 Atherton Road
Rocklin, CA 95765

916.773.8100 TEL
916.773.8448 FAX

www.ecologic-eng.com

Prepared by

ECO:LOGIC
ENGINEERS · CONSULTANTS

ECO:LOGIC
ENGINEERS · CONSULTANTS

Donner Summit Public Utility District

Wastewater Facilities Plan

REGISTERED PROFESSIONAL ENGINEER
JEFFREY R. HAUSER
EXP 12-31-10
No. 31744
CIVIL
STATE OF CALIFORNIA
5-7-10

May 2010

Prepared for
Donner Summit Public Utility District
53823 Sherritt Lane
Soda Springs, CA 95725

Prepared by
ECO:LOGIC

3875 Atherton Road
Rocklin, CA 95765

916.773.8100 TEL
916.773.8448 FAX

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Section 1

Introduction

The Donner Summit Public Utility District (DSPUD) owns and operates a wastewater treatment plant and associated disposal facilities that serve customers within their service boundary in the Soda Springs and Norden areas and users in the adjacent Sierra Lakes County Water District (SLCWD). During times of the year that land disposal of the effluent is not practical (generally, mid-autumn through spring), the plant effluent is discharged to the South Yuba River. When land disposal is practical (generally, summer to mid-autumn), the effluent is used to irrigate the Soda Springs Ski Area, which is desirable not only as a means of disposal, but also to grow grasses for erosion protection on the ski slopes.

Both methods of effluent disposal are governed by waste discharge requirements contained in a National Pollution Discharge Elimination System (NPDES) permit issued by the California Regional Water Quality Control Board, Central Valley Region (Regional Board). The wastewater treatment plant currently has a rated capacity of 0.52 million gallons per day (Mgal/d). Although current flows are generally below the rated capacity, the plant is not able to reliably meet requirements for discharge into the South Yuba River. In particular, requirements for ammonia and nitrate concentrations are not reliably met.

The purpose of this Facilities Plan is to investigate alternatives and to develop a recommended program for bringing the plant and associated disposal facilities into compliance with regulatory requirements, not only for the existing users, but also for expected new growth in both DSPUD and SLCWD.

Prior to authorizing preparation of this Facilities Plan, DSPUD conducted a reconnaissance-level study to investigate a wide range of alternative methods for handling its wastewater. The study included consideration of alternative methods and locations of treatment as well as alternative methods and locations of effluent disposal. The resulting report, entitled "Preliminary Investigation of Wastewater Management Options", dated June 10, 2009, is included herewith as Appendix C. That preliminary investigation served to narrow the list of potential alternative wastewater management schemes to be considered in this Facilities Plan.

Subsequent to this Facilities Plan, future steps required to implement a wastewater treatment and disposal improvement and expansion project include the following:

- Environmental analysis and completion of required environmental documentation
- Preliminary Design
- Detailed Design
- Financing
- Construction-Related Environmental Permitting
- Bidding and Construction
- Startup of New Facilities

Parallel to completing the tasks indicated above, various special studies, some of which are already in progress, must be conducted to investigate particular issues that are identified in the NPDES permit.

Section 2

Executive Summary

Section 2

Executive Summary

This executive summary includes a general overview of the analyses and key findings presented in Sections 3 through 17 of this Facilities Plan report. Each section is considered separately below.

2.1 CLIMATE *(SECTION 3)*

Cold winter temperatures and relatively high annual precipitation amounts (mostly falling as snow) at Donner Summit have major impacts on the design and operation of wastewater treatment and disposal facilities. Typical and extreme temperatures and precipitation amounts are presented on a month-by-month basis in Tables 2-1 and 2-2, respectively.

2.2 WASTEWATER FLOWS AND LOADS *(SECTION 4)*

Historical plant data from 2002 through 2007 and more recent special monitoring data were analyzed to establish appropriate design values for existing average and peak period influent flows and loads to the wastewater treatment plant. Incremental increases to the existing flows and loads were then calculated based on the number of new equivalent dwelling units (EDUs) to be served by the proposed project, as determined by the Donner Summit Public Utility District (DSPUD) and the Sierra Lakes County Water District (SLCWD). The numbers of future EDUs proposed in DSPUD and SLCWD are 332 and 80, respectively. Existing and future design flows and loads are summarized in Table 2-3.

2.3 EXISTING FACILITIES *(SECTION 5)*

The existing DSPUD wastewater treatment plant includes flow equalization, influent screening, integrated fixed film activated sludge (IFAS) biological treatment, filtration, and disinfection with chlorine gas. Effluent is discharged to the South Yuba River during the wet season and used to irrigate the Soda Springs Ski Area during the dry season. Waste activated sludge is stored during the wet season and processed on drying beds prior to landfill disposal in the summer. The overall plant layout and a flow diagram for existing facilities are shown in Figures 2-1 and 2-2, respectively. Design criteria and a hydraulic profile taken from the 1985 construction drawings are shown in Figure 2-3. Design criteria relating to the internals of the two treatment units have been modified in recent years in conjunction with the conversion to the current IFAS system. The reader is referred to Section 5.3 regarding design criteria for the current configuration of the reactor basins.

Section 5 includes discussions of the capacities and performance of each component of the wastewater treatment and disposal system. Of particular concern is the fact that the existing IFAS biological treatment system has not been able to consistently meet discharge requirements for ammonia and nitrate.

Table 2-1
Donner Summit Ambient Temperatures[a]

	Temperature for Indicated Month, °F											
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Daily Minimum Temperature												
Minimum of Monthly Averages	12.5	11.0	13.8	15.2	24.7	29.3	35.8	39.1	34.3	27.4	17.3	10.2
Maximum of Monthly Averages	24.8	27.3	30.5	30.2	38.6	44.6	49.6	50.5	42.9	36.6	30.6	25.7
Average of Monthly Averages	18.7	19.3	21.5	24.4	30.5	37.6	43.9	43.6	39.2	32.5	24.7	19.4
Daily Maximum Temperature												
Minimum of Monthly Averages	29.9	30.3	32.5	33.1	44.7	56.5	66.7	66.2	58.2	48.3	34.7	27.0
Maximum of Monthly Averages	44.7	49.1	52.9	54.8	69.4	74.6	83.8	82.5	76.4	68.3	56.1	47.6
Average of Monthly Averages	37.7	39.2	42.1	46.6	55.3	66.6	76.0	75.4	68.9	58.2	44.2	38.1
Daily Average Temperature												
Minimum of Monthly Averages	21.2	22.5	24.2	24.1	35.3	42.9	51.3	53.1	46.2	38.2	26.8	18.6
Maximum of Monthly Averages	34.4	37.0	39.8	41.8	54.0	58.3	66.5	65.4	59.5	52.2	43.3	36.1
Average of Monthly Averages	28.2	29.2	31.8	35.5	42.9	52.1	60.0	59.5	54.1	45.4	34.4	28.8

(a) Data from Central Sierra Snow Laboratory, Soda Springs, CA, July 1958 through May 2008. Monthly average temperatures are approximated as 30-day rolling average temperatures calculated on the last day of each month.

Table 2-2
Monthly Precipitation Totals[a]

Month	2-Yr RP Precip. (in)	100-Yr RP Precip. (in)	Average Precip. (in)
January	8.74	29.22	9.71
February	7.64	26.99	8.47
March	7.06	23.49	7.81
April	3.36	16.53	4.19
May	1.84	11.11	2.47
June	0.49	5.56	0.87
July	0.05	4.43	0.27
August	0.00	2.39	0.19
September	0.27	5.43	0.70
October	1.71	15.12	2.78
November	4.66	19.45	5.46
December	7.58	29.11	8.75
Total	---	---	51.67

(a) RP – Return Period. Statistical data, from 1871 to 2009, provided by Department of Water Resources, taken from the Soda Springs and Lake Van Norden climate stations.

Table 2-3
Design Flows and Loads Summary[a]

Parameter	Existing Conditions	Allowance for Growth	Future Condition
Design Flows, Mgal/d			
Average Annual Flow (AAF)	0.23	*0.05*	*0.28*
Average Day Maximum Monthly Flow (ADMMF)			
Typical	0.35	*0.07*	*0.42*
High	0.43	*0.09*	*0.52*
Average Day Maximum Weekly Flow (ADMWF)			
Typical	0.43	*0.09*	*0.52*
High	0.61	*0.13*	*0.74*
Peak Day Flow (PDF)	0.97	*0.21*	*1.18*
Peak Hour Flow (PHF)	*1.7*	*0.00*	*1.70*
BOD Load, lb/d			
Average Annual Load (AAL)	215	*70*	*285*
Average Day Maximum Monthly Load (ADMML)	520	*170*	*690*
Average Day Maximum Weekly Load (ADMWL)	780	*255*	*1035*
Peak Day Load (PDL)	900	*294*	*1194*
BOD Concentration, mg/L			
AAL combined with AAF	112	172	123
ADMML combined with Typical ADMMF	178	273	195
ADMML combined with High ADMMF	145	222	159
ADMWL combined with Typical ADMWF	218	334	238
ADMWL combined with High ADMWF	153	235	168
PDL combined with ADMWF	251	385	275
PDL combined with PDF	111		122
TSS Loads and Concentrations	1.0 x BOD	1.0 x BOD	1.0 x BOD
TKN Loads and Concentrations	0.3 x BOD	0.3 x BOD	0.3 x BOD

(a) Explanation of abbreviations and acronyms:
Mgal/d = million gallons per day
BOD = biochemical oxygen demand (5-day basis)
TSS = total suspended solids
TKN = total Kjeldahl nitrogen

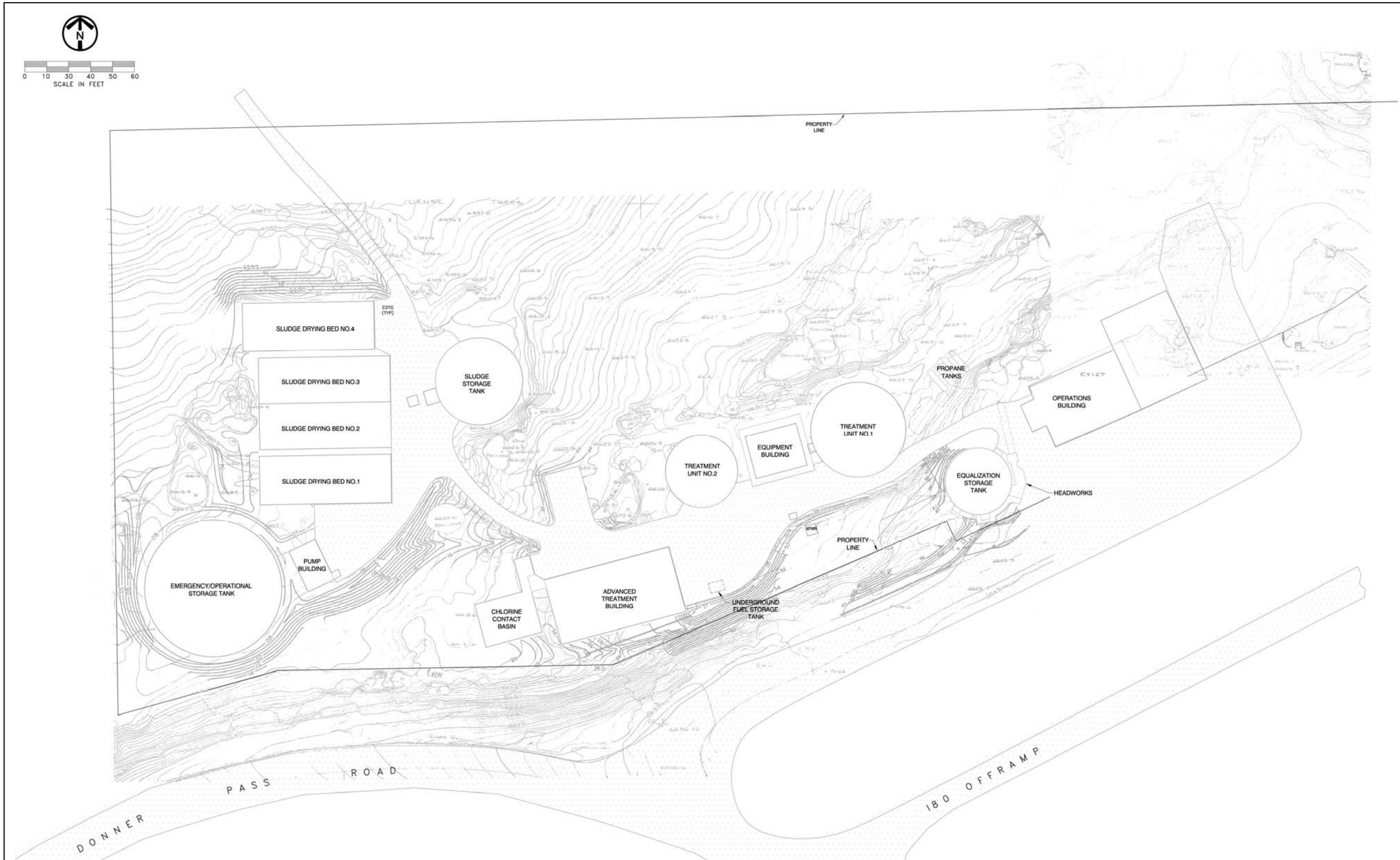


Figure 2-1
Existing Plant Layout

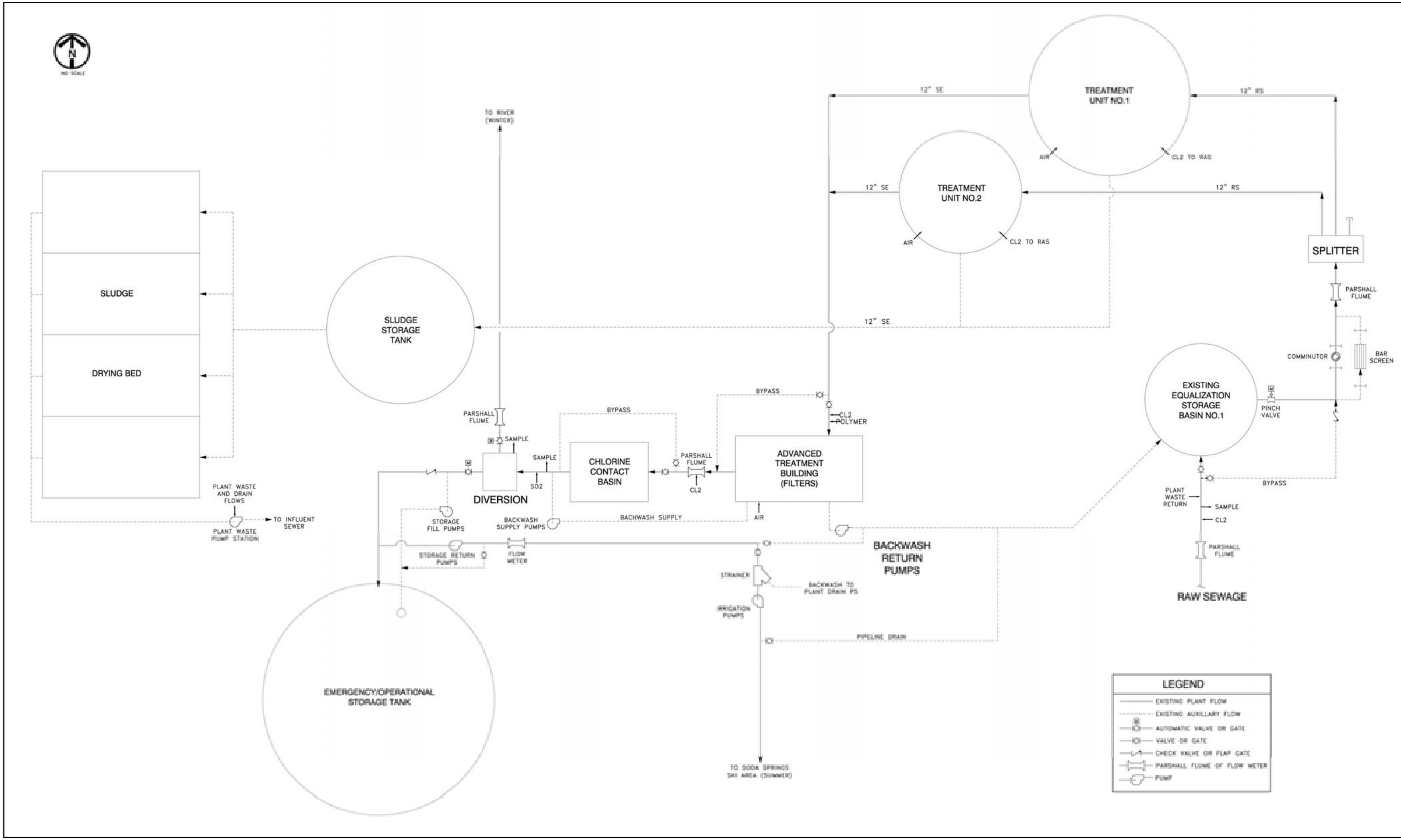


Figure 2-2
Existing Plant Flow Diagram

ITEM	EXIST(a)	PROPOSED	TOTAL
GENERAL DATA			
PEAK 3-DAY DESIGN FLOW(Q), MGD	0.3	0.22	0.52
PLANT HYDRAULIC CAPACITY, MGD	.98	.72	1.7(c)
INFLUENT BOD_5 AND SS, MG/L	--	--	150
INFLUENT BOD_5 AND SS, LOAD, LBS/DAY	375	275	650
MAXIMUM EFFLUENT BOD_5 AND SS, MG/L	--	--	10
EQUALIZATION STORAGE			
VOLUME, MG	0	0.20	0.20
AIR REQUIREMENT, CFM	--	250	250
HEADWORKS			
DESIGN FLOW, MGD	(b)	2.6	2.6
NUMBER COMMINUTORS	(b)	1	1
AERATION			
NUMBER OF BASINS	1	1	2
VOLUME OF BASIN, CF	10,800	7,800	18,600
BOD_5 LOADING, LBS/1000 CF·DAY	35	35	--
MIXED LIQUOR VOLATILE SUSPENDED SOLIDS, MG/L	2,000-3,500	2,000-3,500	--
SOLIDS RETENTION TIME, DAYS	6-12	6-12	--
HYDRAULIC DETENTION TIME @ Q, HRS	6.4	6.4	--
AIR SUPPLIED, CF/LB BOD_5 REMOVED	1,600	1,600	--
AIR REQUIRED, CFM	425	300	725
RETURN SLUDGE PUMPING			
NUMBER OF PUMPS	4	4	8
MAXIMUM RETURN RATE, MGD	.4	.22	.62
AIRLIFT PUMP AIR REQUIREMENT, CFM	140	160	300

ITEM	EXIST(a)	PROPOSED	TOTAL
CLARIFICATION			
NUMBER OF BASINS	1	1	2
NOMINAL DIAMETER, FT	37	32	--
OVERFLOW RATE @ Q, GPD/FT^2	280	275	--
SOLIDS LOADING RATE, $LB/DAY·FT^2$	25	25	--
AEROBIC DIGESTION			
NUMBER OF BASINS	1	1	2
VOLUME OF BASIN, CF	13,200	9,700	22,900
DESIGN SOLIDS LOAD, LBS/DAY	154	113	267
SOLIDS RETENTION TIME, DAYS	120	120	--
BLOWERS (EXCEPT FILTER BACKWASH)			
NUMBER	3	2	5
OUTPUT PRESSURE, PSI			
NO. 1	--	9	--
NO. 2	--	14.5	--
NOS. 3-5	7.0	--	--
HP EACH BLOWER			
NO. 1	--	15	--
NO. 2	--	30	--
NOS. 3-5	30	--	--
MAXIMUM OUTPUT PER BLOWER, INLET CFM			
NO. 1	--	220	--
NO. 2	--	200	--
NOS. 3-5	640	--	--
SLUDGE STORAGE RESERVOIR			
NUMBER	1	0	1
VOLUME, MG	0.6	0	0.6
AIR REQUIREMENT, CFM	100(b)	250	250
SLUDGE DEWATERING BEDS			
NUMBER BEDS	2	2	4
DRYING BED AREA, FT^2	5,600	4,800	10,400
LOADING RATE LBS SOLIDS/FT^2·SEASON	6.4	6.4	--
DESIGN DETENTION TIME, DAYS	12	12	--

ITEM	EXIST(a)	PROPOSED	TOTAL
FILTRATION			
NUMBER OF CELLS	0	3	3
FILTRATION RATE, 1 CELL @ Q, GPM/FT^2	--	5	--
SIZE OF EACH CELL, FT^2	--	72	--
WATER BACKWASH RATE, GPM/FT^2	--	12	--
WATER BACKWASH FLOW, GPM	--	900	900
NUMBER OF BACKWASH PUMPS (TOTAL FOR SUPPLY AND RETURN)	0	3	3
HP EACH PUMP	--	25	--
AIR SCOUR RATE, CFM/FT^2	--	4	--
AIR SUPPLY @ 5 PSI, CFM	--	290	290
NUMBER OF BLOWERS	0	2	2
HP EACH BLOWER	--	10	--
CHLORINATION			
NUMBER CHLORINATORS	1	2	3
CHLORINATOR CAPACITY, LB/D			
PRECHLORINATION	--	150	--
RETURN SLUDGE	--	30	--
FINAL DISINFECTION	70(b)	150	--
SULFONATION			
NUMBER SULFONATORS	0	2	2
CAPACITY OF EACH, LB/D	--	100	--
CHLORINE CONTACT			
NUMBER BASINS	1(b)	1	1
CONTACT TIME AT Q, MIN	--	60	60
AT 1.7 MGD, MIN	--	18	18
BASIN VOLUME, 1000 GAL	--	22	22
PLANT WASTE RETURN PUMPS			
NUMBER PUMPS	2(b)	2	2
CAPACITY EACH PUMP, GPM	150(b)	150	--
HP EACH PUMP	10(b)	20	--

ITEM	EXIST(a)	PROPOSED	TOTAL
EMERGENCY/OPERATIONAL STORAGE			
NUMBER OF TANKS	0	1	1
VOLUME OF TANK, MG	--	1.56	1.56
DAYS STORAGE @ Q	--	3	3
STORAGE FILL PUMPS:			
NUMBER	0	3	3
CAPACITY, EACH, GPM	--	900	--
HP EACH	--	10	--
STORAGE RETURN PUMPS:			
NUMBER	0	2	2
CAPACITY, EACH, GPM	--	600	--
HP EACH	--	15	--
IRRIGATION			
DESIGN FLOW, MGD	--	0.19	0.19
DESIGN APPLICATION, IN/MO	--	4.8	--
APPLICATION RATE, IN/HR	--	0.23	--
IRRIGATED AREA, ACRES	--	45	45
NUMBER IRRIGATION PUMPS	--	2	2
CAPACITY EACH PUMP, GPM	--	600	--
HP EACH PUMP	--	100	--
RECLAIMED WATER			
LOW PRESSURE:			
NUMBER PUMPS	2(b)	2	2
CAPACITY EACH PUMP, GPM	40(b)	70	--
HP EACH PUMP	2(b)	5	--
HIGH PRESSURE:			
NUMBER PUMPS	0	1	1
CAPACITY, GPM	--	100	--
HP	--	10	--

DESIGN DATA NOTES

(a) EXISTING PLANT DESIGN CRITERIA CORRECTED FOR DERATING TO 0.3 MGD

(b) EXISTING UNIT(S) WILL BE REMOVED FROM THIS SERVICE.

(c) EXPANDABLE TO 2.6 MGD.

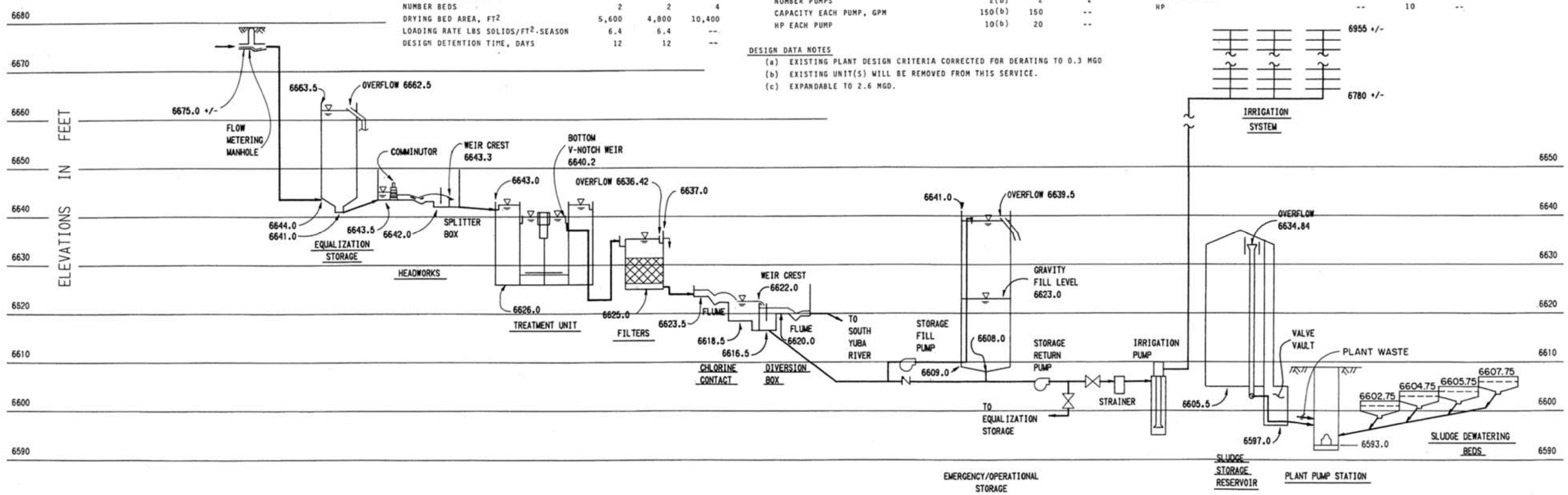


Figure 2-3
Design Data and Hydraulic Profile From 1985 Construction Drawings

2.4 WASTE DISCHARGE AND TREATMENT REQUIREMENTS *(SECTION 6)*

The DSPUD wastewater treatment plant effluent is discharged to the South Yuba River during the wet season and when discharges to land are not possible due to snow cover or wet soils. During the dry season, when possible, the effluent is used for irrigation of the Soda Springs Ski Area. Both of these methods of disposal are regulated under a National Pollution Discharge Elimination System (NPDES) permit and waste discharge requirements adopted by the California Regional Water Quality Control Board, Central Valley Region. The permit is updated approximately every five years. The current permit was adopted on April 24, 2009 (Order No. R5-2009-0034, NPDES No. CA0081621).

Key permit requirements, together with information on the performance of the existing plant and permit compliance strategies, are summarized in Table 2-4.

Some of the most problematic requirements of the permit are those for monthly average effluent ammonia-nitrogen and nitrate-nitrogen concentrations of 2.1 and 10 mg/L, respectively, for discharge to the South Yuba River. These are considered problematic because the existing plant frequently does not comply and substantial improvements will be required to attain compliance. Another troublesome permit requirement is that the discharge cannot cause water in the South Yuba River to contain biostimulatory substances that promote aquatic growths in concentrations that cause nuisance or adversely affect beneficial uses. This is troublesome because there were nuisance growths of algae in the river downstream from the DSPUD discharge in June 2008 and the discharge may have been a contributory factor. The current permit requires DSPUD to study this issue.

2.5 DEVELOPMENT AND SCREENING OF ALTERNATIVES *(SECTION 7)*

Prior to embarking on this Facilities Plan, DSPUD authorized a study to identify and screen various alternatives for wastewater treatment and disposal. The report from that study is included as Appendix C. The key conclusion from that investigation was that DSPUD should continue to operate its own wastewater treatment plant and continue current disposal methods, with the caveat that storage of effluent may be required in the spring (prior to beginning land disposal) to mitigate potential nuisance algae growths (biostimulation) in the South Yuba River.

After review of various options for biological treatment, DSPUD determined that the following options should be studied in this Facilities Plan:

- Upgrade the Existing IFAS System, 2-Stage
- Upgrade the Existing IFAS System, 4-Stage
- New IFAS, 4-Stage
- Submerged Attached Growth
- Membrane Bioreactor (MBR), 4-Stage

Table 2-4
Key NPDES Permit Requirements, Plant Performance and Compliance Strategy

Parameter	Units	Effluent Limits[a]	Existing Plant Performance[b]	Compliance Strategy
BOD	mg/L	10/15/30	Generally compliant.	Continue/improve biological treatment, coagulation and filtration.
pH	Units	6.5 to 8.0[c]	Generally compliant.	Automatic chemical addition for alkalinity and pH control.
TSS	mg/L	10/15/30	Generally compliant.	Continue/improve biological treatment, coagulation and filtration.
Aluminum	µg/L	71/--/143	Frequently noncompliant. (---, ---, 620, 1310, 38.4, 127)	Monitor acid soluble aluminum. Possible Water Effects Ratio (WER).
Ammonia-N	mg/L	2.1/--/5.6	Frequently noncompliant. (Frequent non-certified lab data over 25 mg/L)	Improved treatment required.
Copper	µg/L	1.5/--/3.1	Frequently noncompliant. (4, 4, 7.8, 4.2, 5.9, 6)	Increase effluent hardness by using lime instead of soda ash for required alkalinity addition. Consider increased potable water pH. Possible Water Effects Ratio (WER).
Cyanide	µg/L	4.3/--/8.5	Occasionally noncompliant. (23, <2, 33, <2, DNQ 4, <2)	Evaluate future monitoring results. Consider changing from chlorine to UV disinfection. Consider immediate on-site testing without sample preservation.
Aldrin	µg/L	ND(d)	Rare noncompliance. (<0.002, <0.002, <0.002, DNQ 0.005, <0.002, <0.0028)	Evaluate future monitoring. Public education, source control if needed.
Alpha BHC	µg/L	ND(d)	Rare noncompliance. (<0.005, <0.005, 0.044, <0.005, <0.005, <0.00034)	Evaluate future monitoring. Public education, source control if needed.
Dichlorobromomethane	µg/L	0.56/--/1.2	Uncertain (e). (<0.5, <0.5, <0.5, DNQ 0.3, 1.2, 0.2)	Violations of this chlorine disinfection byproduct will be more likely with complete nitrification. Consider dilution credit, chloramination, UV disinfection.
Nitrate-N	mg/L	10/--/--	Frequently noncompliant. (Frequent non-certified lab data over 15 mg/L. Would be worse with good nitrification.)	Improved treatment required.
Silver	µg/L	0.23[d]	Rare noncompliance. (<0.09, <0.08, 0.26, 0.18, < 0.1, <0.12)	Evaluate future monitoring. Public education, source control if needed.
Zinc	µg/L	15/--/30	Frequently noncompliant. (22, 33, 22, 23.6, 25.3, 30.8)	Increase effluent hardness by using lime instead of soda ash for required alkalinity addition. Consider increased potable water pH. Possible Water Effects Ratio (WER).

Parameter	Units	Effluent Limits[a]	Existing Plant Performance[b]	Compliance Strategy
Manganese	mg/L	50[f]	Possible noncompliance. (---, ---, 8.7, 8.3, 52.8, 88.4)	Evaluate future monitoring and consider manganese removal in treatment process evaluations.
Total Coliform	MPN/100 mL	2.2, 23, 240[g]	Generally compliant.	Continue/improve biological treatment, coagulation, filtration, and disinfection.
Turbidity	NTU	2, 5, 10[h]	Generally compliant.	Continue/improve biological treatment, coagulation and filtration.

[a] Unless indicated otherwise, limits are Average Monthly/Average Weekly/Maximum Daily.

[b] Where a series of six results are shown in parenthesis, they are from special California Toxics Rule and related grab samples taken in June 2001, April 2002, November 2003, February 2004, December 2005, and December 2006, respectively. "DNQ" indicates an estimated value that is below the method quantitation limit, which is indicated after "DNQ".

[c] Range is based on instantaneous minimum and instantaneous maximum.

[d] Instantaneous maximum.

[e] Dichlorobromomethane is a chlorine disinfection byproduct that is mitigated by the presence of ammonia. Ammonia concentrations at the time of historical sampling are unknown.

[f] Annual average.

[g] 2.2 weekly median, 23 once in 30 days, 240 at any time.

[h] 2 daily average, 5 more than 5% of time in 24 hours, 10 at any time.

2.6 INFLUENT FLOW EQUALIZATION AND PLANT HEADWORKS *(SECTION 8)*

Based on review of historical flow records, equalization storage volumes that would be needed to limit peak flows through the wastewater treatment plant were determined. Based on an approximate economic analysis of the costs of equalization storage and resultant cost savings from reduced peak flows through the treatment facilities, together with consideration of plant reliability benefits associated with equalization, it is recommended that DSPUD install a new 550,000 gallon equalization storage tank to supplement the existing 200,000 gallon tank. It is estimated that this would allow equalizing the plant flow over an entire peak week.

Two general alternatives for providing the new equalization storage capacity were considered, depending on whether the influent sewage is to be screened upstream or downstream of equalization storage. Alternative 1 involves downstream screening, which is the current practice. Upstream screening (Alternative 2) is preferred to keep nuisance large solids materials out of the equalization tanks, as well as providing other benefits, but is more expensive.

If a MBR biological treatment system is selected, a new headworks facility with new finer screens would be required. With this modification, Alternatives 1 and 2 become Alternatives 1-MBR and 2-MBR.

For Alternative 2 and Alternative 2-MBR, there are subalternatives, depending on whether the existing sludge storage tank is to be converted for equalization use (Alternatives 2-E and 2-MBR-E) or whether an entirely new equalization tank is to be built (Alternatives 2-N and 2-MBR-N).

Costs for all of the alternatives are shown in Table 2-5. Although using the existing solids holding tank for equalization reduces the cost for equalization storage as compared to building a new tank, the savings are outweighed by additional solids handling costs that would result if the tank were not used in its current function. Therefore, Alternatives 2-E and 2-MBR-E were eliminated from consideration.

Alternatives 2 and 2-MBR were eliminated because of the additional costs involved, as compared to Alternatives 1 and 1-MBR, and the fact that the existing facilities (similar to Alternative 1) have functioned adequately for over 20 years.

The choice between Alternative 1 and 1-MBR depends on the selection of a biological treatment alternative, which is discussed in Section 17.

2.7 BIOLOGICAL TREATMENT *(SECTION 9)*

Temperatures in biological reactor basins are critical because they have a major impact on biological reaction kinetics. Low temperatures are of particular concern for the microorganisms that accomplish the removal of ammonia in a process called nitrification. A minimum design temperature of 7 °C is recommended. Since the existing reactor basins frequently experience temperatures as low as 4 and 5 °C, the option of covering the equalization and biological reactor basins to conserve heat was extensively investigated. It was determined that adding heat when needed using a boiler and heat exchanger system would be more economical than providing basin covers.

Table 2-5
Equalization and Headworks Alternative Costs

	Cost for Indicated Alternative (a), $					
Alternative Designation:	**1**	**1-MBR**	**2-N**	**2-E**	**2-MBR-N**	**2-MBR-E**
Biological Treatment Alt.:	**Not MBR**	**MBR**	**Not MBR**	**Not MBR**	**MBR**	**MBR**
EST2 New or Existing:	**New**	**New**	**New**	**Exist**	**New**	**Exist**
Capital Cost						
New Mixing and Aeration in EST1	180,000	180,000	180,000	180,000	180,000	180,000
Construct New EST2	300,000	300,000	300,000		300,000	
Mixing and Aeration in EST2	250,000	250,000	250,000	275,000	250,000	275,000
Blower and Elec Bldg at EST2	120,000	120,000	120,000	120,000	120,000	120,000
Pump Station at EST2	100,000	100,000	200,000	200,000	200,000	200,000
Back-Up Pump Sump at EST2			25,000	25,000	25,000	25,000
New Headworks / Fine Screens		600,000			600,000	600,000
Sitework	50,000	80,000	50,000	40,000	80,000	70,000
Equalization Site Piping	50,000	60,000	120,000	120,000	130,000	130,000
Electrical and Instrumentation	200,000	380,000	230,000	230,000	410,000	410,000
Subtotal 1	1,250,000	2,070,000	1,475,000	1,190,000	2,295,000	2,010,000
General Conditions, OH&P, 20%	250,000	410,000	300,000	240,000	460,000	400,000
Subtotal 2	1,500,000	2,480,000	1,775,000	1,430,000	2,755,000	2,410,000
Contingency, 20%	300,000	500,000	360,000	290,000	550,000	480,000
Total Construction Cost	1,800,000	2,980,000	2,135,000	1,720,000	3,305,000	2,890,000
Engineering, Admin., Environ. 25%	450,000	750,000	530,000	430,000	830,000	720,000
Total Capital Cost	2,250,000	3,730,000	2,665,000	2,150,000	4,135,000	3,610,000
Annual Cost						
Labor	25,000	26,000	27,000	27,000	28,000	28,000
Power	22,000	22,000	24,000	24,000	24,000	24,000
Total	47,000	48,000	51,000	51,000	52,000	52,000
Present Worth Cost						
Present Worth of Annual Costs (b)	699,000	714,000	759,000	759,000	774,000	774,000
Total Present Worth	2,949,000	4,444,000	3,424,000	2,909,000	4,909,000	4,384,000

(a) First quarter 2010 cost level, ENR 20-Cities CCI = 8700.

(b) 20 years at inflation-adjusted discount rate of 3 percent, Present Worth Factor = 14.88.

A biological treatment alternative analysis was completed to investigate the alternatives previously identified (see Section 2.5 above).

Two methods (two-stage and four-stage) for upgrading and expanding the existing IFAS system, which is based on web-type biological growth support media, were originally planned for study. However, after analysis and consultation with the manufacturer of the existing web-type media, it was determined that: 1) web-type media is no longer offered by the manufacturer and 2) the four-stage option is definitely preferred over the two-stage option. Therefore, only one option for upgrading and expanding the existing IFAS system was developed in detail. This involves use of new structured sheet media (offered by the same manufacturer) in a four-stage (anoxic-aerobic-anoxic-aerobic) arrangement.

It was determined that, for all alternatives, except the submerged attached growth alternative, the existing Plant 1 and Plant 2 basins would be fully utilized as the new process reactor basins. This includes converting the existing clarifiers to new reactor basins. For the IFAS alternatives, new secondary clarifiers would be built as separate structures. For the MBR alternative, the clarifiers would be replaced with new membrane filtration systems. The submerged attached growth alternative would require the construction of all new basins for a high-rate chemically-assisted primary clarification process and for the actual submerged attached growth reactors and ancillary facilities.

A cost analysis for the biological treatment alternatives is shown in Table 2-6. Final selection between the biological treatment alternatives depends on other areas of the plant that would be impacted by this choice. Therefore, selection between these alternatives is considered in Section 17, which is summarized in Section 2.15.

Table 2-6
Biological Treatment Alternative Cost Analysis

Item	Upgrade Existing IFAS	New IFAS	MBR	Submerged Attached Growth
Capital Costs				
Demolition and Modification Inside Plant 1 and Plant 2 Structures	150,000	85,000	100,000	0
New Chemically-Enhanced Primary Clarification Structures	0	0	0	570,000
New Process Basins for Submerged Attached Growth	0	0	0	160,000
Main Process Flow Pump Stations in Treatment System	0	0	0	400,000
New Secondary Clarifiers and Splitter Box	930,000	930,000	0	0
New RAS Pump Station	300,000	300,000	0	0
Membrane Basins (b)	0	0	330,000	0
Main Vendor Equipment Package, Installed	450,000	1,300,000	1,900,000	3,100,000
Anoxic Mixers Installed	90,000	Included	90,000	0
Aeration Facilities Not in Main Equipment Package	250,000	50,000	250,000	0
Other Ancillary Facilities and Equipment	100,000	50,000	100,000	50,000
Internal Process Piping	50,000	50,000	300,000	300,000
Building Enclosures	150,000	150,000	950,000	2,000,000
Subtotal 1	2,470,000	2,915,000	4,020,000	6,580,000
Electrical and Instrumentation @ 25% of Subtotal 1	620,000	730,000	1,010,000	1,650,000
Site Piping @ 10% of Subtotal 1	250,000	290,000	400,000	660,000
Sitework @ 5% of Subtotal 1	120,000	150,000	200,000	330,000
Subtotal 2	3,460,000	4,085,000	5,630,000	9,220,000
Contingencies @ 20% of Subtotal 2	690,000	820,000	1,130,000	1,840,000
Subtotal 3	4,150,000	4,905,000	6,760,000	11,060,000
General Conditions, Overhead and Profit @ 20% of Subtotal 3	830,000	980,000	1,350,000	2,210,000
Total Construction Cost	4,980,000	5,885,000	8,110,000	13,270,000
Engineering, Administration and Environmental @ 25%	1,250,000	1,470,000	2,030,000	3,320,000
Total Capital Cost	6,230,000	7,355,000	10,140,000	16,590,000
Annual O&M Costs				
Labor	140,000	140,000	140,000	160,000
Power	35,000	35,000	40,000	20,000
Ammonia	20,000	20,000	20,000	16,000
Lime	12,000	12,000	12,000	11,000
Methanol (c)	17,000	17,000	14,000	17,000
Ferric Chloride	0	0	0	18,000
Other Chemicals	0	0	2,000	5,000
Maintenance Materials, Not Including Membranes	9,000	14,000	23,000	32,000
Membrane Replacement	0	0	0	14,000
Total Annual Cost	233,000	238,000	251,000	293,000
Present Worth Costs				
Present Worth of Annual Costs (d)	3,467,040	3,541,440	3,734,880	4,359,840
Total Present Worth	9,697,040	10,896,440	13,874,880	20,949,840

(a) First quarter 2010 cost level, ENR 20-Cities CCI = 8700.
(b) Depending on manufacturer, membrane basins may be prefabricated and part of equipment package.
(c) Methanol is assumed herein, but other carbon sources can be used and should be investigated during design.
(d) 20 years at inflation-adjusted discount rate of 3 percent, Present Worth Factor = 14.88.

Regardless of which biological treatment alternative is selected, new or modified chemical feed systems for ammonia, methanol (or an alternative carbon source), and alkalinity are needed. Costs for these improvements are summarized in Section 2.15.

2.8 TERTIARY FILTRATION *(SECTION 10)*

The existing tertiary filtration system has adequate capacity for the future design flows and would continue to be used in conjunction with the IFAS and submerged attached growth biological treatment options. This filtration system would no longer be needed with the MBR alternative.

For continued use of the existing filtration system, it is recommended that a new backwash supply tank and ancillary facilities be constructed so that filter backwash water would not have to be taken from the chlorine contact basin as it is currently. Under the current configuration, the chlorine contact basin is drawn down during filter backwash cycles, which interrupts the plant effluent flow and causes problems in the control of sulfur dioxide feed rates, which are flow paced. Costs for these improvements are summarized in Section 2.15.

2.9 EFFLUENT DISINFECTION *(SECTION 11)*

Three alternatives for effluent disinfection were analyzed:

- Continued use of chlorine gas (and sulfur dioxide gas for dechlorination)
- Ultraviolet (UV) radiation
- Ozonation

For continued use of chlorine and sulfur dioxide gases, compliance with Uniform Fire Code requirements for storage and use of these hazardous materials is recommended. For this Facilities Plan, it is presumed that automatic shutoff valves would be provided on all supply cylinders for these gases (this was assumed for the ammonia feed system also). However, the final improvements required should be confirmed after consultation with the governing fire authorities.

With UV radiation, differing design criteria resulting in lower costs are used following MBRs, as compared to the other biological treatment alternatives. Additionally, a closed vessel UV system can be considered for use with MBRs. Similarly, ozonation alone is considered to be an adequate disinfection process only after MBR biological treatment. For the other biological treatment alternatives, ozonation would be supplemented with UV radiation, but at a lower UV dose than would be required with UV alone.

A disinfection alternative cost analysis is shown in Table 2-7. Although ozonation provides the benefit of substantial removals of emerging contaminants of concern, such as pharmaceuticals and personal care products, these contaminants are not currently regulated, so ozonation was eliminated from further consideration. Both chlorination and UV disinfection are carried forward for further consideration in the overall project alternative analysis summarized in Section 2.15.

Table 2-7
Disinfection Alternative Cost Analysis

Item	Cost for Indicated Alternative (a), $					
	Chlorine	UV-Filt	UV-MBR-OC	UV-MBR-CV	Ozone/UV-Filt	Ozone-MBR
Capital Cost						
Modify Existing Gas Feed Systems	12,000	0	0	0	0	0
Automatic Emergency Shutoff Valves and Controls	200,000	0	0	0	0	0
Expand Chlorine Contact Basin	60,000	0	0	0	0	0
Install River Diffuser	150,000	0	0	0	0	0
Install River Gaging Station	100,000	0	0	0	0	0
New Basins / System Piping / Ancillary Mechanical	0	170,000	150,000	50,000	310,000	260,000
Building Enclosures		290,000	240,000	110,000	400,000	320,000
UV Equipment, Installed	0	590,000	460,000	540,000	350,000	0
Ozone Equipment, Installed	0	0	0	0	800,000	800,000
Subtotal 1	522,000	1,050,000	850,000	700,000	1,860,000	1,380,000
Elect/Instrum, 25% of Subtotal 1, Unless Noted Otherwise (b)	50,000	263,000	213,000	175,000	465,000	345,000
Sitework, 5% of Subtotal 1 Unless Noted Otherwise	Included	42,000	34,000	28,000	74,000	55,000
Site Piping, 10% of Subtotal 1, Unless Noted Otherwise	Included	105,000	85,000	70,000	186,000	138,000
Subtotal 2	572,000	1,460,000	1,182,000	973,000	2,585,000	1,918,000
Contingencies, 20%	114,000	292,000	236,000	195,000	517,000	384,000
Subtotal 3	686,000	1,752,000	1,418,000	1,168,000	3,102,000	2,302,000
General Conditions, Overhead and Profit, 20%	137,000	350,000	284,000	234,000	620,000	460,000
Total Construction Cost	823,000	2,102,000	1,702,000	1,402,000	3,722,000	2,762,000
Engineering and Administration, 25%	206,000	526,000	426,000	351,000	931,000	691,000
Special Studies, Permitting (c)	170,000	0	0	0	0	0
Total Capital Cost	1,199,000	2,628,000	2,128,000	1,753,000	4,653,000	3,453,000
Annual Costs						
Labor	8,400	9,740	8,940	8,140	18,720	12,480
Power	1,000	17,000	17,000	19,000	20,000	10,000
Chemicals	8,000	0	0	0	4,000	4,000
Maintenance Materials	3,000	9,000	9,000	10,000	13,000	8,000
Total Annual Cost	20,400	35,740	34,940	37,140	55,720	34,480
Present Worth Costs						
Present Worth of Annual Costs (d)	304,000	532,000	520,000	553,000	829,000	513,000
Total Present Worth Cost	1,503,000	3,160,000	2,648,000	2,306,000	5,482,000	3,966,000

(a) First quarter 2010 cost level, ENR 20-Cities CCI = 8700.
(b) Chlorine alternative electrical and instrumentation cost not based on 25% of Subtotal 1.
(c) For dilution credits, need mixing zone study, anti-degredation analysis and NPDES permit revision.
(d) 20 years at inflation-adjusted discount rate of 3%, Present Worth Factor = 14.88.

2.10 EMERGENCY STORAGE AND IRRIGATION STORAGE *(SECTION 12)*

The existing 1.5 Mgal tank that is used for emergency storage and irrigation operational storage is adequate for continued use in the proposed project. No expansion of this facility is needed.

2.11 EFFLUENT STORAGE TO MITIGATE BIOSTIMULATION IN THE SOUTH YUBA RIVER *(SECTION 13)*

In June 2008, there were nuisance growths of algae in the South Yuba River beginning immediately downstream from the DSPUD wastewater effluent discharge point. As a result, the Regional Water Quality Control Board has required DSPUD to investigate the potential impact of its effluent on biostimulation in the river. Although those studies are inconclusive at this time, storage of the DSPUD effluent during such times in the spring that river flows and other conditions are conducive to algae growths may be beneficial. This “biostimulation storage” would continue until the time that land disposal could be initiated each year.

It is believed that the onset of nuisance algae growths can only occur after the spring snowmelt has been essentially completed and river flows have substantially decreased each year. Based on review of historical flow patterns in the South Yuba River combined with information on when land disposal was commenced each year from 2002 through 2009, it is suggested that the timing of biostimulation storage can be correlated to river flows. Storage would be started when the 7-day average flow of the South Yuba River at Cisco Grove decreases from the peak snowmelt-related values to about 300 or 400 cfs. Storage would be ended and land disposal would be started when or shortly after the 7-day average flow falls below 20 cfs. River flows below 20 cfs are believed to occur after the snow is gone and the land is dry enough to support land disposal of effluent.

After analyzing historical plant flows in conjunction with river flows, it was determined that a reasonable amount of biostimulation storage for future conditions could be in the range of about 7 to 11 Mgal.

Six potential sites for earthen reservoirs and three potential sites for steel or concrete storage tanks were investigated, as shown in Figure 2-4. It was determined that lined earthen reservoirs would be most cost effective and that the best site for such a reservoir would probably be Site No. 3.

The estimated capital cost for a 12 Mgal reservoir and related pumping and piping systems is about $3.9 million (the cost at 11 Mgal would be essentially the same).

Specific operational procedures will be needed to allow precipitation falling on the reservoir during the fall and winter to be drained to the river and to make sure that adequate reservoir volume, free from ice and snow, is available when needed each spring. Appropriate plans will have to be reviewed and accepted by the Regional Water Quality Control Board.

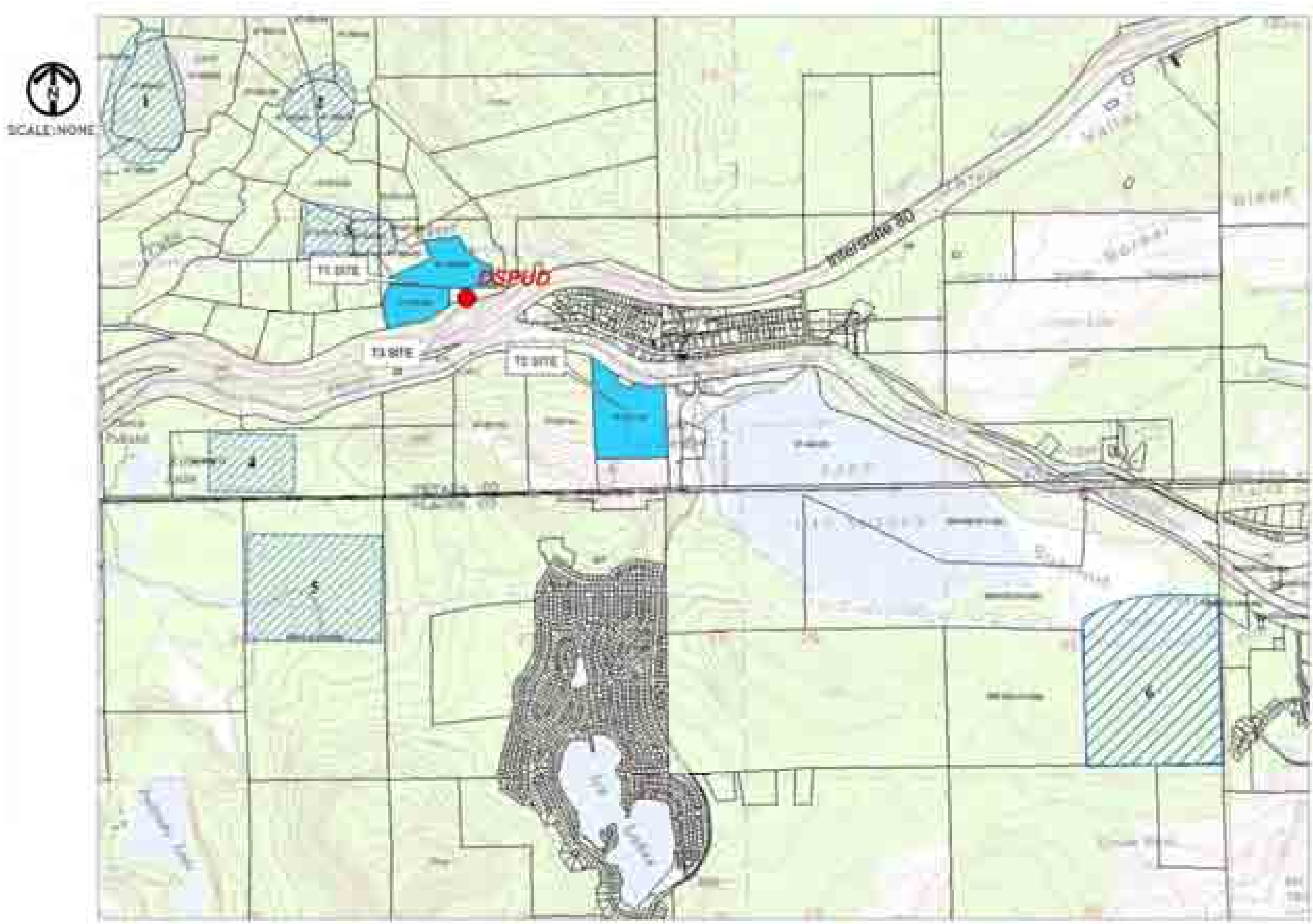


Figure 2-4
Potential Tank and Reservoir Sites

2.12 EFFLUENT IRRIGATION DISPOSAL *(SECTION 14)*

DSPUD currently operates a 45 acre irrigation disposal system at the Soda Springs Ski Area. However, portions of the area typically have moist soils even without irrigation and are not used. The net effective existing irrigation area is estimated to be about 34 acres; however, more detailed site analyses during preliminary design are warranted to confirm this area.

Based on calculations with 100-year return frequency precipitation in the required land disposal months of August and September, the irrigation areas needed to support existing and future flows, not including the effects of potential biostimulation storage, were determined to be about 28 and 32 acres, respectively. These area requirements are very sensitive to certain key assumptions upon which the analyses are based and should be confirmed after more detailed information is developed during preliminary design. Based on these calculations, it is currently anticipated that, without biostimulation storage, no expansion of the existing irrigation disposal system would be required.

When up to 11 Mgal of biostimulation storage is included in the analysis, the required irrigation area for future conditions could be around 53 acres, necessitating a 19 acre expansion of the existing system. Again, these areas are subject to verification during preliminary design.

Six potential sites for expanding the irrigation disposal system were investigated as shown in Figure 2-5. Because of its proximity to the existing irrigation disposal system and the fact that DSPUD has already secured the right to expand in this area, Site No. 4 is preferred, followed by Site No. 3 and Site No. 5. Detailed soils investigations during preliminary design are required to confirm the site selection.

If Site No. 4 can be used, the estimated capital cost for the 19 additional acres of irrigation area, including land preparation as well as irrigation and runoff recovery system improvements, is about $700,000. The existing irrigation supply pumping system and conveyance pipeline from the wastewater treatment plant are adequate and do not need to be improved.

2.13 RESIDUAL SOLIDS PROCESSING AND DISPOSAL *(SECTION 15)*

Four alternatives for handling residual solids produced during wastewater treatment were analyzed for each of the biological treatment alternatives. The four alternatives are:

1. Continued use of the existing solids storage tank and sludge drying beds.
2. Construction of a new aerobic digester and mechanical dewatering using a belt press.
3. Construction of a new aerobic digester and mechanical dewatering using a centrifuge.
4. Construction of a new aerobic digester and mechanical dewatering using a screw press.

After calculating residual solids quantities associated with each of the biological treatment alternatives, capital and annual cost analyses were completed for each of the residual solids handling alternatives, as shown in Table 2-8. Regardless of which biological treatment alternative is selected, the most cost-effective residual solids handling alternative is to continue using the existing solids storage tank and sludge drying beds. This is because the potential cost savings that could be realized by converting the existing solids storage tank for use as an equalization storage tank are far outweighed by the costs of mechanical dewatering.

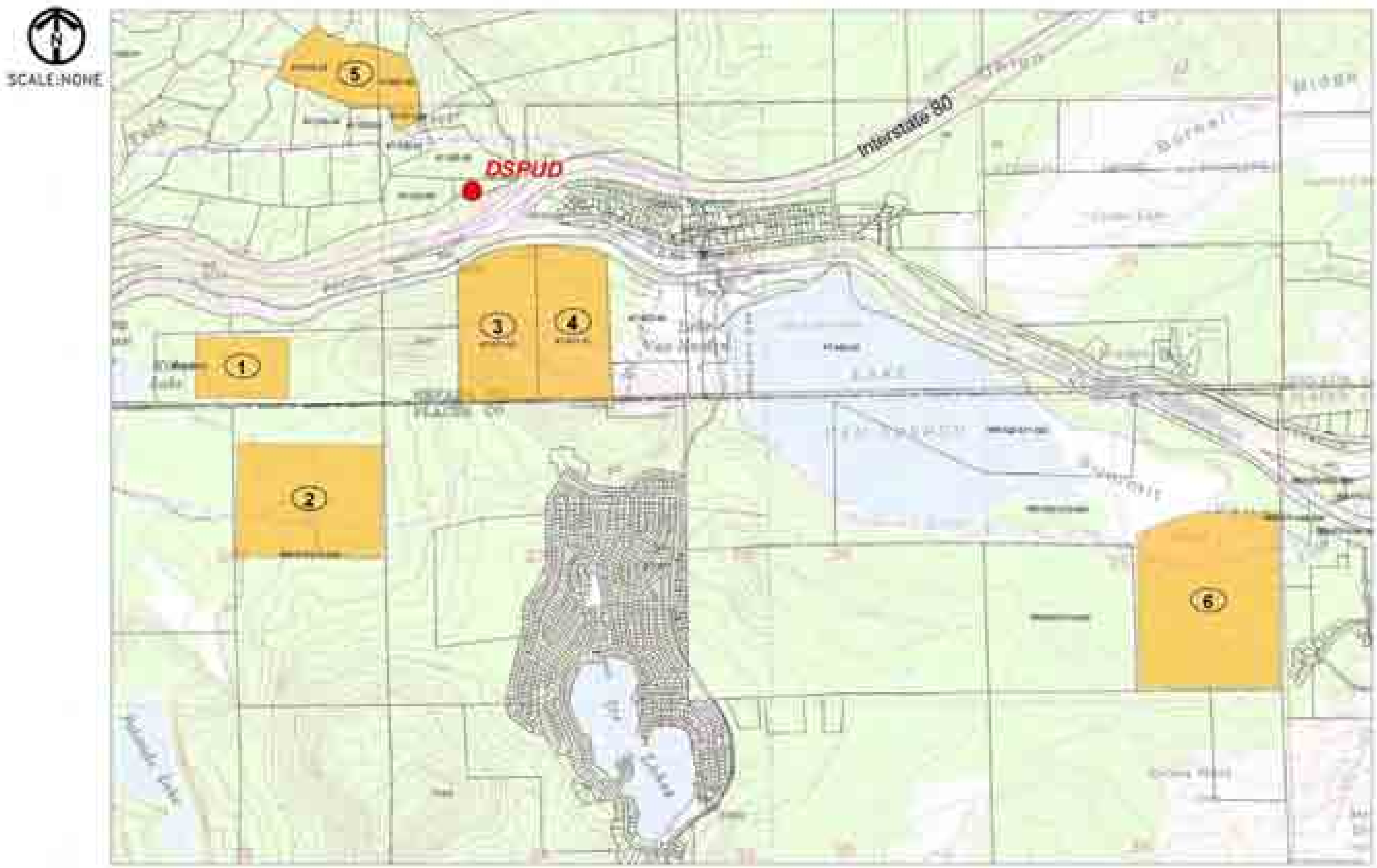


Figure 2-5
Potential Irrigation Disposal Sites

2.14 PRELIMINARY ENVIRONMENTAL ANALYSIS *(SECTION 16)*

A preliminary desktop environmental review was conducted of each potential treated effluent storage reservoir site, tank site, and effluent irrigation disposal site investigated in Sections 13 and 14 and summarized above. The purpose of the environmental reviews was to identify possible environmental issues or environmental "fatal flaws" that could threaten the viability of the proposed improvements. Examples of environmental "fatal flaws" are the presence of endangered species and the existence of wetlands that would require extensive mitigation. The evaluations in Section 16 were incorporated in the selection of sites in Sections 13 and 14. Most of the sites investigated, including the preferred sites identified in Sections 13 and 14, are unlikely to contain environmental "fatal flaws."

Of all the sites investigated, only one contains a clear "fatal flaw": the Sugar Bowl parcel investigated as Reservoir Site No. 6 and Irrigation Disposal Site No. 6. This site contains the greatest potential to cause both impacts to the environment (wetlands, special-status species, etc.) and create public scrutiny of the project, especially if any infrastructure connecting the site with DSPUD existing facilities impact Lake Van Norden and its associated highly valued wetlands (mountain meadow).

2.15 SELECTION AND DESCRIPTION OF THE APPARENT BEST PROJECT *(SECTION 17)*

An overall alternative cost analysis including the combined costs of equalization storage and headworks, biological treatment, filtration, disinfection, residual solids handling, and shop/office space is shown in Table 2-9. In the analysis, each biological treatment alternative was coupled with either chlorine or UV disinfection, resulting in a total of eight alternatives. As indicated in the table, the least cost alternative is to upgrade the existing IFAS system, coupled with chlorine disinfection. The next least cost alternative is a new IFAS system, coupled with chlorine disinfection.

In addition to capital and annual costs, the alternatives were compared with respect to various noneconomic factors in a weighted rating analysis, as shown in Table 2-10. This table was developed with the input and review of DSPUD staff and the Joint Wastewater Facilities Committee formed by DSPUD and SLCWD in an effort to assure that the criteria included in the table and the relative weighting factors appropriately reflect the interests and concerns of DSPUD and SLCWD. As indicated in Table 2-10, when noneconomic factors are included, the combined project alternative with the highest overall score is the MBR biological treatment alternative, coupled with UV disinfection. The second ranked alternative is MBR coupled with chlorine disinfection.

The main reasons for the high overall ratings for MBR biological treatment include a high level of confidence in this technology that has been extensively proven in probably thousands of plants throughout the world and for which standard and nonproprietary biological process design methods can be employed. Additionally, the MBR is considered to be the most robust and reliable treatment system evaluated and the MBR provides a higher level of treatment than any other system, which would be helpful in meeting both existing and anticipated future discharge requirements. Finally, the MBR has the smallest footprint and is the easiest to expand.

Some of the reasons why chlorine disinfection was rated lower than UV disinfection could be eliminated if chloramination could be tested and proven effective as a method for mitigation of disinfection byproducts, without the need for dilution credits. This, coupled with the issue that ozonation may be desired in the future to remove emerging contaminants of concern, resulted in a determination by the Joint Wastewater Facilities Committee that both chloramination and UV disinfection should be considered further after completion of this Facilities Plan.

Another matter considered by the Joint Wastewater Facilities Committee was whether biostimulation storage and the spray irrigation system expansion that would be triggered by such storage should be included in the recommended project. Since the causes and contributing factors that produced the algal bloom in the South Yuba River in June 2008 are not known and since no such bloom occurred in 2009 nor is known to have occurred in years prior to 2008, the need for spending millions of dollars on a biostimulation storage reservoir and associated expansion of the spray irrigation disposal system cannot be firmly established at this time. Further studies of biostimulation in the South Yuba River are ongoing and planned. Accordingly, the committee determined that project costs with and without biostimulation storage and related facilities should be indicated in the Facilities Plan.

Based on the considerations above, cost estimates were developed for four alternative project combinations, as shown in Table 2-11. The four alternatives cover both chloramination and UV disinfection, with and without biostimulation storage and irrigation facilities. As indicated in the table, the additional cost of UV over chloramination is about $700,000. However, this cost difference would be reduced by the cost of testing and proving the effectiveness of chloramination. The cost difference resulting from the addition of biostimulation storage and an irrigation system expansion is about $4.9 million. These cost differences are based on the escalated costs indicated in Table 2-11.

It is anticipated that DSPUD, working together with SLCWD, will decide whether to pursue chloramination or UV disinfection and whether or not to include biostimulation storage in the proposed project after review of this document and consideration of other factors that are relevant to the two Districts.

A flow diagram and a conceptual site plan for the recommended improvements (with alternatives) are presented in Figures 2-6 and 2-7, respectively.

Table 2-8
Residual Solids Handling Alternative Cost Analysis

	Cost for Indicated Alternative (a), $											
Biological Treatment Alt.	**MBR**				**IFAS (Upgrade or New)**				**Submerged Attached Growth**			
Digester Alt.	**Exist**	**New**	**New**	**New**	**Exist**	**New**	**New**	**New**	**Exist**	**New**	**New**	**New**
Dewatering Alt.	**Beds**	**Belt**	**Cent.**	**Screw**	**Beds**	**Belt**	**Cent.**	**Screw**	**Beds**	**Belt**	**Cent.**	**Screw**
Capital Costs												
Modify Existing Solids Tank	232,000				232,000				232,000			
New Digester and Ancillary		285,000	285,000	285,000		285,000	285,000	285,000		315,000	315,000	315,000
Sludge Dewatering and Related Equipment,		425,000	542,500	391,000		425,000	542,500	391,000		425,000	542,500	391,000
Sludge Dewatering Building		230,000	200,000	200,000		230,000	200,000	200,000		230,000	200,000	200,000
Subtotal 1	232,000	940,000	1,027,500	876,000	232,000	940,000	1,027,500	876,000	232,000	970,000	1,057,500	906,000
Sitework @ 5% of Subtotal 1	NA	47,000	51,000	44,000	NA	47,000	51,000	44,000	NA	49,000	53,000	45,000
Site Piping @ 10% of Subtotal 1	NA	94,000	103,000	88,000	NA	94,000	103,000	88,000	NA	97,000	106,000	91,000
Electrical/Instrum. @ 25% of Subtotal 1	58,000	235,000	257,000	219,000	58,000	235,000	257,000	219,000	58,000	243,000	264,000	227,000
Subtotal 2	290,000	1,316,000	1,438,500	1,227,000	290,000	1,316,000	1,438,500	1,227,000	290,000	1,359,000	1,480,500	1,269,000
General Conditions, Overhead and Profit, 20%	58,000	263,000	288,000	245,000	58,000	263,000	288,000	245,000	58,000	272,000	296,000	254,000
Subtotal 3	348,000	1,579,000	1,726,500	1,472,000	348,000	1,579,000	1,726,500	1,472,000	348,000	1,631,000	1,776,500	1,523,000
Contingency, 20%	70,000	316,000	345,000	294,000	70,000	316,000	345,000	294,000	70,000	326,000	355,000	305,000
Total Construction Cost	418,000	1,895,000	2,071,500	1,766,000	418,000	1,895,000	2,071,500	1,766,000	418,000	1,957,000	2,131,500	1,828,000
Engineering, Admin, Environmental, 25%	105,000	474,000	518,000	442,000	105,000	474,000	518,000	442,000	105,000	489,000	533,000	457,000
Total Capital Cost	523,000	2,369,000	2,589,500	2,208,000	523,000	2,369,000	2,589,500	2,208,000	523,000	2,446,000	2,664,500	2,285,000
Annual Costs												
Labor	24,600	14,500	14,700	13,000	24,100	14,400	14,600	13,000	32,700	15,600	16,000	13,400
Power	14,300	5,700	6,300	5,900	13,800	5,500	6,000	5,600	18,400	5,500	6,300	5,800
Polymer	1,300	1,800	3,000	2,200	1,200	1,700	2,900	2,200	2,100	2,600	4,300	3,200
Hauling and Disposal	2,100	9,700	8,800	10,300	2,000	9,400	8,500	10,000	5,100	14,200	12,800	15,000
Maintenance	14,500	65,800	71,900	61,400	14,500	65,800	71,900	61,400	14,500	68,000	74,000	63,500
Total Annual Cost	56,800	97,500	104,700	92,800	55,600	96,800	103,900	92,200	72,800	105,900	113,400	100,900
Present Worth of Annual Costs (b)	845,000	1,451,000	1,558,000	1,381,000	827,000	1,440,000	1,546,000	1,372,000	1,083,000	1,576,000	1,687,000	1,501,000
Total Present Worth Cost	1,368,000	3,820,000	4,147,500	3,589,000	1,350,000	3,809,000	4,135,500	3,580,000	1,606,000	4,022,000	4,351,500	3,786,000

(a) In first-quarter 2010 dollars, ENR 20-Cities CCI = 8700.

(b) 20 years at inflation-adjusted discount rate of 3 percent, Present Worth Factor = 14.88.

Table 2-9
Overall Alternative Cost Analysis

Biological Treatment Alternative: Disinfection Alternative:	Cost for Indicated Combination of Alternatives (a), $							
	Upgrade Existing IFAS		New IFAS		MBR		Submerged Attached Growth	
	Chlorine	UV	Chlorine	UV	Chlorine	UV	Chlorine	UV
Capital Cost								
Equalization Storage / Headworks (b)	2,250,000	2,250,000	2,250,000	2,250,000	3,730,000	3,730,000	2,250,000	2,250,000
Biological Treatment	6,230,000	6,230,000	7,355,000	7,355,000	10,140,000	10,140,000	16,590,000	16,590,000
Filtration (c)	201,000	201,000	201,000	201,000	0	0	700,000	700,000
Disinfection (d)	1,199,000	2,628,000	1,199,000	2,628,000	1,199,000	1,753,000	1,199,000	2,628,000
Solids Handling (e)	523,000	523,000	523,000	523,000	523,000	523,000	523,000	523,000
Reconfigure Existing Space for Shop/Offic	0	25,000	0	25,000	50,000	75,000	0	25,000
New Shop/Office Space	475,000	385,000	475,000	385,000	195,000	105,000	475,000	385,000
Total	10,878,000	12,242,000	12,003,000	13,367,000	15,837,000	16,326,000	21,737,000	23,101,000
Annual Cost								
Equalization Storage / Headworks (b)	47,000	47,000	47,000	47,000	48,000	48,000	47,000	47,000
Biological Treatment	227,000	227,000	233,000	233,000	251,000	251,000	293,000	293,000
Filtration (c)	11,950	11,950	11,950	11,950	0	0	14,340	14,340
Disinfection (d)	20,400	35,740	20,400	35,740	20,400	37,140	20,400	35,740
Solids Handling (e)	43,400	43,400	43,400	43,400	44,600	44,600	60,600	60,600
Total	349,750	365,090	355,750	371,090	364,000	380,740	435,340	450,680
Present Worth Cost								
Present Worth of Annual Costs (f)	5,204,000	5,433,000	5,294,000	5,522,000	5,416,000	5,665,000	6,478,000	6,706,000
Total Present Worth	16,082,000	17,675,000	17,297,000	18,889,000	21,253,000	21,991,000	28,215,000	29,807,000

(a) First quarter 2010 cost level, ENR 20-Cities CCI = 8700.
(b) Based on Equalization Concept 1.
(c) New coagulation and flocculation assumed to be required ahead of the filters for the submerged attached growth option.
(d) Chlorine cost based on free chlorine, not chloramination. Costs include studies and facilities needed to obtain dilution credits for disinfection byproducts.
UV disinfection for MBR based on closed vessel system.
(e) Based on continued use of existing solids storage tank and sludge drying beds.
(f) 20 years at inflation-adjusted discount rate of 3 percent. Present Worth Factor = 14.88.

Table 2-10
Alternative Ratings and Ranking

Criterion	Weighting Factor %	Ratings For Indicated Alternative Combination (a)							
		Upgrade Existing IFAS		New IFAS		MBR		Submerged Attached Growth	
		Chlorine	UV	Chlorine	UV	Chlorine	UV	Chlorine	UV
Capital Cost	25	10.0	8.9	9.1	8.1	6.8	6.6	5.0	4.7
Annual Cost	10	10.0	9.6	9.8	9.4	9.6	9.2	8.0	7.8
Confidence In Design and Technology	25	4	4	8	8	10	10	7	7
Robustness and Reliability	5	8	8	8	8	10	10	8	8
Misc. Compliance Improvements, Exi	5	6	7	6	7	9	10	6	7
Adaptability to Future Permits	5	6	8	6	8	10	8	6	8
Ease of Future Expansion	5	9	9	9	9	10	10	9	9
Plant Footprint	5	8	8	8	8	10	10	8	8
Construction Impacts in River (d)	3	5	10	5	10	5	10	5	10
Power Use	3	9	8	9	8	8	7	10	9
Chemical Use	3	9	10	9	10	9	10	8	9
Residuals Produced	3	10	10	10	10	10	10	8	8
Hazardous Gas Exposure Risk	3	3	10	3	10	3	10	3	10
Overall Weighted Score (b)	100	7.43	7.63	8.19	8.41	8.66	8.88	6.67	7.09
Rank (c)		6	5	4	3	2	1	8	7

(a) The highest rated alternative is assigned a score of 10. Other alternatives are scored lower, according to the relative concern compared to the highest rated alternative.
(b) Summation of individual ratings multiplied by the corresponding weighting factors.
(c) The alternative with the highest overall weighted score is ranked "1". Other alternatives are ranked "2" through "8", according to overall score.
(d) Construction in the river would be associated with continuing chlorine disinfection, based on installing a diffuser to obtain dilution credits for disinfection byproducts.

Table 2-11
Alternative Project Cost Estimates

Item	Cost (a), $			
	With Biostimulation Storage and Irrigation		Without Biostimulation Storage and Irrigation	
	MBR, UV	MBR, Chloram.	MBR, UV	MBR, Chloram.
Wastewater Treatment Plant				
Existing Equalization Facilities Modifications	180,000	180,000	180,000	180,000
New Equalization Storage Tank and Ancillary Facilities	770,000	770,000	770,000	770,000
New Headworks / Fine Screens	600,000	600,000	600,000	600,000
Modify Plants 1 and 2 Basins	100,000	100,000	100,000	100,000
New Membrane Basins	330,000	330,000	330,000	330,000
New MBR System Equipment, Installed	1,900,000	1,900,000	1,900,000	1,900,000
Building for MBR and Related Equipment	950,000	950,000	950,000	950,000
Secondary Process Equipment Not Included in MBR Pkg.	440,000	440,000	440,000	440,000
MBR Internal Process Piping	300,000	300,000	300,000	300,000
Secondary Process Supplemental Heat System	739,000	739,000	739,000	739,000
Ammonia Feed System Modifications	175,000	175,000	175,000	175,000
Methanol Storage and Feed System	225,000	225,000	225,000	225,000
Soda Ash Feed System Modifications	20,000	20,000	20,000	20,000
Chlorine and Sulfur Dioxide System Modifications, Chloram.		312,000		312,000
Expand Chlorine Contact Basin		60,000		60,000
UV Disinfection Structures	160,000		160,000	
UV Disinfection Equipment, Installed	540,000		540,000	
Modify Existing Sludge Storage Tank	232,000	232,000	232,000	232,000
Shop/Office Space	75,000	140,000	75,000	140,000
New Standby Power System in Building	300,000	300,000	300,000	300,000
Subtotal 1, Wastewater Treatment Plant	8,036,000	7,773,000	8,036,000	7,773,000
Electrical and Instrumentation at 25% of Subtotal 1	2,010,000	1,940,000	2,010,000	1,940,000
Sitework @ 5% of Subtotal 1	400,000	390,000	400,000	390,000
Site Piping @ 10% of Subtotal 1	800,000	780,000	800,000	780,000
Subtotal 2, Wastewater Treatment Plant	11,246,000	10,883,000	11,246,000	10,883,000
Remote Facilities				
Biostimulation Storage and Ancillary Facilities	2,626,000	2,626,000		
Expand Spray Irrigation Disposal System	475,000	475,000		
Subtotal 3, Remote Facilities	3,101,000	3,101,000	0	0
Subtotal 4, Wastewater Treatment Plant and Remote Facilities	14,347,000	13,984,000	11,246,000	10,883,000
General Conditions, Overhead and Profit @ 20% of Subtotal 4	2,250,000	2,180,000	2,250,000	2,180,000
Subtotal 5	16,597,000	16,164,000	13,496,000	13,063,000
Contingencies @ 20% of Subtotal 5	3,320,000	3,230,000	2,700,000	2,610,000
Total Construction Cost	19,917,000	19,394,000	16,196,000	15,673,000
Engineering, Administration and Environmental @ 25%	4,980,000	4,850,000	4,050,000	3,920,000
Total Project Cost	24,897,000	24,244,000	20,246,000	19,593,000
Escalated Total Project Cost (b)	26,420,000	25,730,000	21,490,000	20,790,000

(a) First-quarter 2010 cost level, ENR 20-Cities CCI = 8700, except as noted below.
(b) Escalated construction cost based on assumed inflation rate of 2% per year for three years to the estimated mid-point of construction, ENR 20-Cities CCI = 9233.

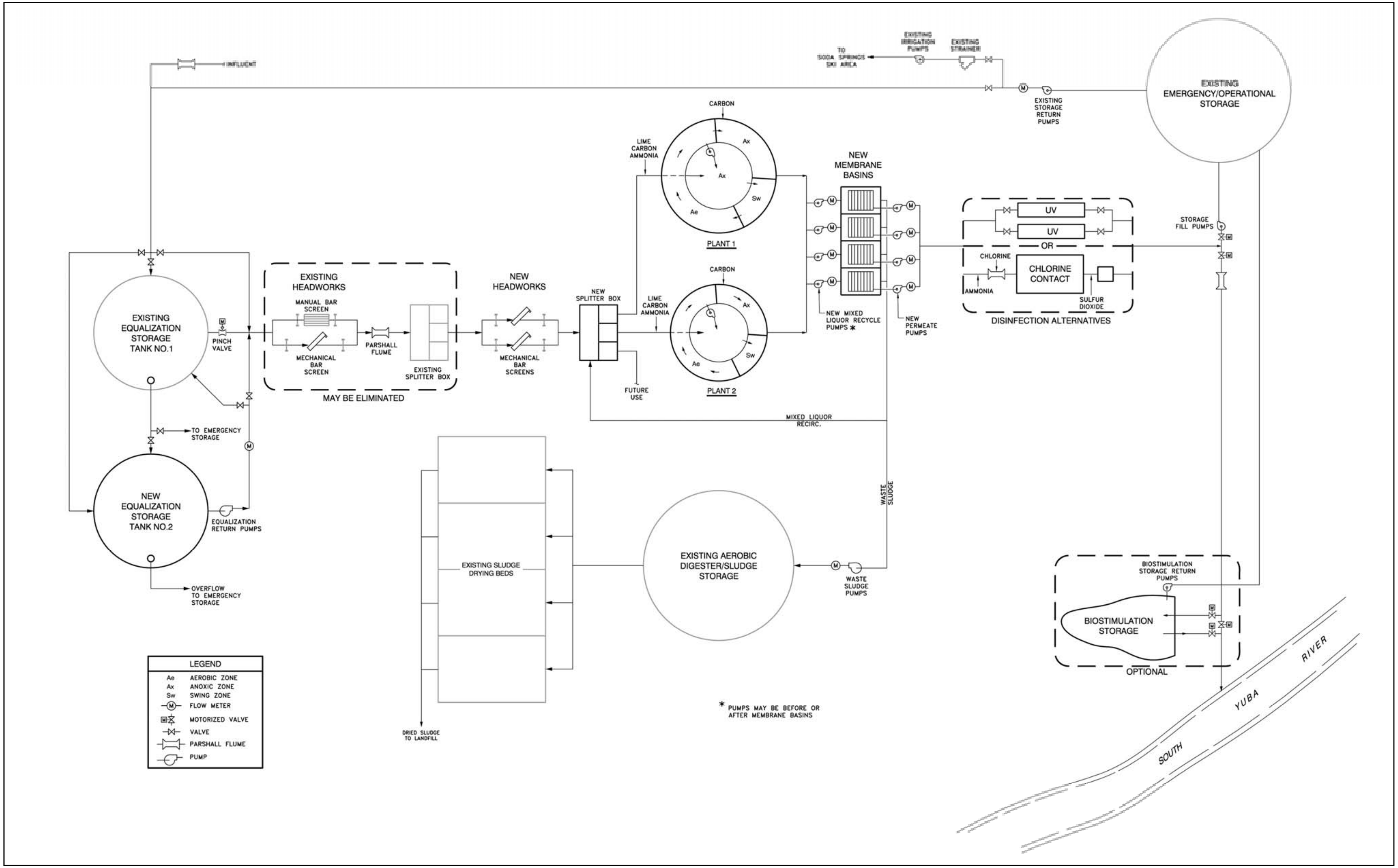


Figure 2-6
Flow Diagram

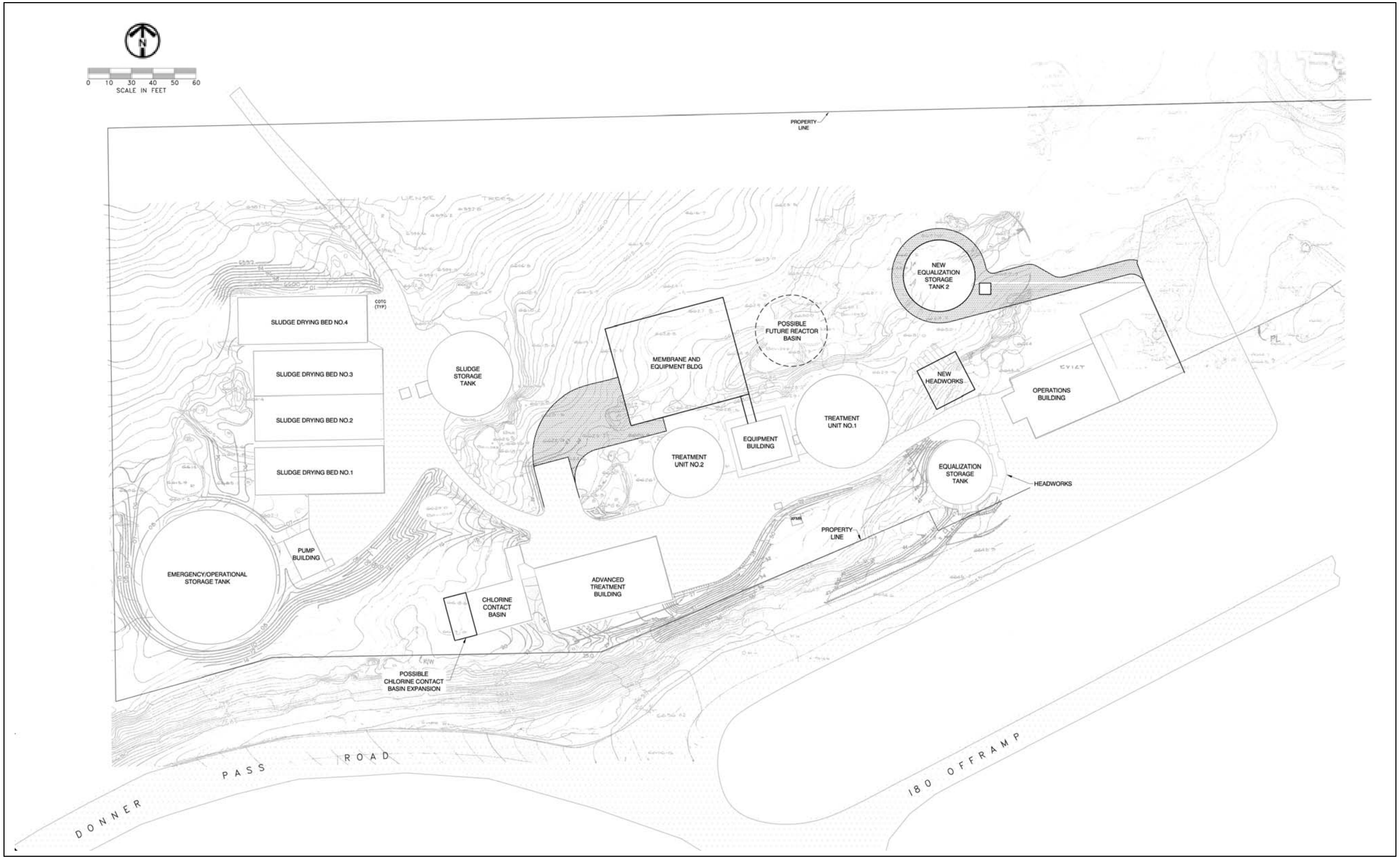


Figure 2-7
Conceptual Site Plan

Section 3

Climate

Section 3

Climate

The climate of the Donner Summit area has major impacts on wastewater management practices. For example, the combination of low temperatures and high precipitation amounts in the late fall and winter preclude land disposal of wastewater effluent during those times of the year. Land disposal also is not practical for most of the spring because, depending on temperatures, snow can remain on the ground well into the spring, with wet soil conditions persisting even longer. Low ambient temperatures during the winter result in low temperatures in the wastewater treatment process basins and slow rates of treatment. Low temperatures and snow conditions also necessitate special considerations in plant design to assure continued performance and operator access to processes and equipment throughout the winter.

Because of the importance of temperatures and precipitation amounts on wastewater treatment and disposal practices, typical and extreme values for these parameters are considered on a month-by-month basis below.

3.1 AMBIENT TEMPERATURES

Ambient temperatures in the Donner Summit area are indicated in Table 3-1. As indicated in the table, the coldest month of the year is typically January, with 50-year average daily minimum and maximum temperatures of about 19 °F and 38 °F, respectively. The warmest month of the year is typically July, with 50-year average daily minimum and maximum temperatures of about 44 °F and 76 °F, respectively.

3.2 PRECIPITATION

Average, 2-year return period, and 100-year return period monthly precipitation totals for the Donner Summit area are shown in Table 3-2. The average annual precipitation in the area is about 52 inches per year, most of which falls as snow. The average annual snowfall reported at the nearby Central Sierra Snow Laboratory is nearly 34 feet.

As snow falls in the colder months, it typically accumulates until temperatures begin to warm in the spring. Warmer temperatures lead to melting and runoff of the accumulated snow, which directly affects the amount of flow in the South Yuba River. Snow frequently remains on the slopes until mid-June. Flow patterns in the South Yuba River resulting from the spring snowmelt are presented and discussed in Section 13.

The precipitation data presented in Table 3-2 were compiled from the old Lake Van Norden weather station and more recent historical data from the Central Sierra Snow Laboratory. It represents the time period from 1871 through 2009. During that period, the maximum reported precipitation for May was 10.27 inches, for September was 6.45 inches and for August, the driest month on average at Donner Summit, the maximum was reported to be 2.28 inches. Just as significant rainfall events can occur during typically dry months, the Sierra Nevada can experience extremely dry periods, with the lowest precipitation reported on an annual basis (19.20 inches) in 1924. This condition is reflected in the 2-year return period precipitation reported in Table 3-2.

Table 3-1
Donner Summit Ambient Temperatures (a)

	Temperature for Indicated Month, °F											
	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Daily Minimum Temperature												
Minimum of Monthly Averages	12.5	11.0	13.8	15.2	24.7	29.3	35.8	39.1	34.3	27.4	17.3	10.2
Maximum of Monthly Averages	24.8	27.3	30.5	30.2	38.6	44.6	49.6	50.5	42.9	36.6	30.6	25.7
Average of Monthly Averages	18.7	19.3	21.5	24.4	30.5	37.6	43.9	43.6	39.2	32.5	24.7	19.4
Daily Maximum Temperature												
Minimum of Monthly Averages	29.9	30.3	32.5	33.1	44.7	56.5	66.7	66.2	58.2	48.3	34.7	27.0
Maximum of Monthly Averages	44.7	49.1	52.9	54.8	69.4	74.6	83.8	82.5	76.4	68.3	56.1	47.6
Average of Monthly Averages	37.7	39.2	42.1	46.6	55.3	66.6	76.0	75.4	68.9	58.2	44.2	38.1
Daily Average Temperature												
Minimum of Monthly Averages	21.2	22.5	24.2	24.1	35.3	42.9	51.3	53.1	46.2	38.2	26.8	18.6
Maximum of Monthly Averages	34.4	37.0	39.8	41.8	54.0	58.3	66.5	65.4	59.5	52.2	43.3	36.1
Average of Monthly Averages	28.2	29.2	31.8	35.5	42.9	52.1	60.0	59.5	54.1	45.4	34.4	28.8

(a) Data from Central Sierra Snow Laboratory, Soda Springs, CA, July 1958 through May 2008. Monthly average temperatures are approximated as 30-day rolling average temperatures calculated on the last day of each month.

Table 3-2
Monthly Precipitation Totals (a)

Month	2-Yr RP Precip. (in)	100-Yr RP Precip. (in)	Average Precip. (in)
January	8.74	29.22	9.71
February	7.64	26.99	8.47
March	7.06	23.49	7.81
April	3.36	16.53	4.19
May	1.84	11.11	2.47
June	0.49	5.56	0.87
July	0.05	4.43	0.27
August	0.00	2.39	0.19
September	0.27	5.43	0.70
October	1.71	15.12	2.78
November	4.66	19.45	5.46
December	7.58	29.11	8.75
Total	---	---	51.67

(a) RP – Return Period. Statistical data, from 1871 to 2009, provided by Department of Water Resources, taken from the Soda Springs and Lake Van Norden climate stations.

Section 4

Wastewater Flows and Loads

Section 4

Wastewater Flows and Loads

Existing wastewater flows and loads were developed in Technical Memorandum No. 1, which was first issued as a partial draft in February 2008, based on evaluation of historical plant data from January 2002 through September 2007. The memorandum was later updated to include special plant monitoring data developed in January and February, 2008. Projections of future flows and loads were added to the memorandum in September 2009, after both DSPUD and SLCWD established the number of future dwelling units to be served by the proposed project. These projections were then revised, resulting in the final version of the memorandum, dated November 11, 2009, which is included herewith as Appendix A.

A summary of design flows and loads from TM1 is shown in Table 4-1. The reader is referred to Appendix A for a complete discussion of how these flows and loads were developed.

Subsequent to the preparation of TM1, analyses were completed to assess "typical" flows and loads on a month-by-month basis throughout the year. These typical values are based on analysis of historical data from January, 2002 through December, 2007. The average monthly flow for each month in that period was divided by the annual average flow for that calendar year, resulting in the normalized flow variations shown in Figure 4-1. The monthly ratio values shown in Figure 4-1 were then averaged and multiplied by the projected future annual average design flow o f 0.28 Mgal/d to develop projected future design flows on a month-by-month basis as shown in Table 4-2.

Historical monthly BOD load data were analyzed similar to the flows, resulting in the normalized load variations shown in Figure 4-2. Only the years 2005 through 2007 were included in Figure 4-2 because of significant missing data in previous years. The monthly ratios shown in Figure 4-2 were then averaged and applied to the future design average annual BOD load of 285 lb/d to obtain the month-by-month future design BOD loads shown in Table 4-2. Total suspended solids (TSS) and Total Kjeldahl nitrogen (TKN) loads are estimated to be 1.0 and 0.3 times the BOD loads, respectively.

Table 4-1
Design Flows and Loads Summary[a]

Parameter	Existing Conditions	Allowance for Growth	Future Condition
Design Flows, Mgal/d			
Average Annual Flow (AAF)	0.23	*0.05*	*0.28*
Average Day Maximum Monthly Flow (ADMMF)			
Typical	0.35	*0.07*	*0.42*
High	0.43	*0.09*	*0.52*
Average Day Maximum Weekly Flow (ADMWF)			
Typical	0.43	*0.09*	*0.52*
High	0.61	*0.13*	*0.74*
Peak Day Flow (PDF)	0.97	*0.21*	*1.18*
Peak Hour Flow (PHF)	*1.7*	*0.00*	*1.70*
BOD Load, lb/d			
Average Annual Load (AAL)	215	*70*	*285*
Average Day Maximum Monthly Load (ADMML)	520	*170*	*690*
Average Day Maximum Weekly Load (ADMWL)	780	*255*	*1035*
Peak Day Load (PDL)	900	*294*	*1194*
BOD Concentration, mg/L			
AAL combined with AAF	112	172	123
ADMML combined with Typical ADMMF	178	273	195
ADMML combined with High ADMMF	145	222	159
ADMWL combined with Typical ADMWF	218	334	238
ADMWL combined with High ADMWF	153	235	168
PDL combined with ADMWF	251	385	275
PDL combined with PDF	111		122
TSS Loads and Concentrations	1.0 x BOD	1.0 x BOD	1.0 x BOD
TKN Loads and Concentrations	0.3 x BOD	0.3 x BOD	0.3 x BOD

(a) Explanation of abbreviations and acronyms:
Mgal/d = million gallons per day
BOD = biochemical oxygen demand (5-day basis)
TSS = total suspended solids
TKN = total Kjeldahl nitrogen

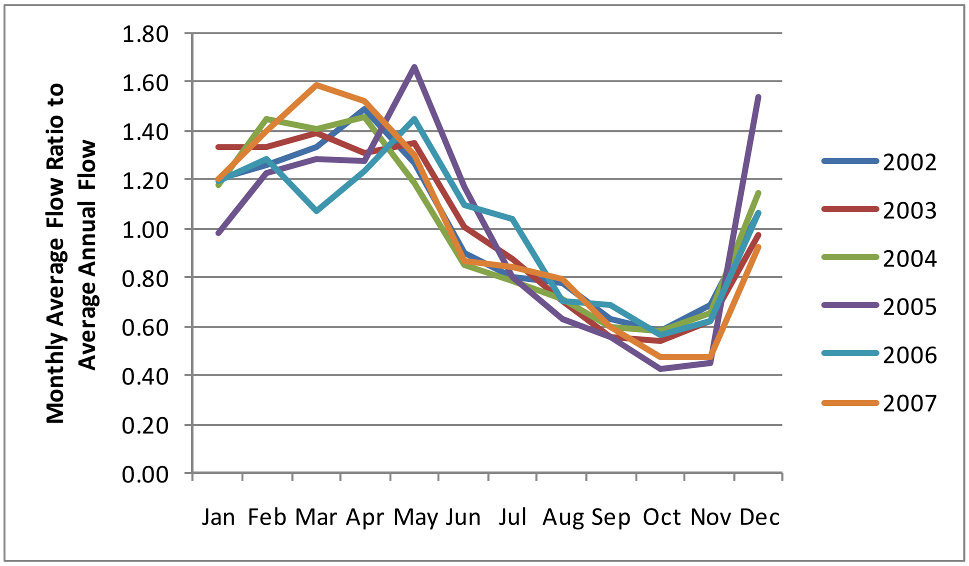


Figure 4-1
Monthly Average Flow Compared to Average Annual Flow 2002-2007

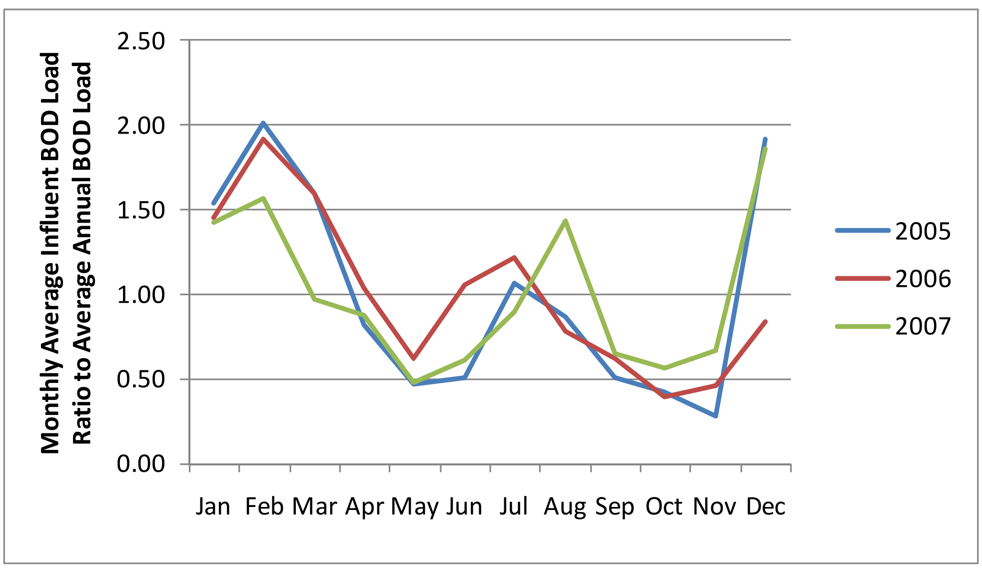


Figure 4-2
Monthly Average BOD Load Compared to Average Annual BOD Load, 2005-2007

Table 4-2
Design Average Flows and Loads on a Month-by-Month Basis

Month	Monthly Avgerage Flow		Monthly Average BOD Load	
	Ratio to Ann. Avg. Flow 2002-2007	Future Design Flow, Mgal/d	Ratio to Ann. Avg. BOD Load 2005-2007	Future Design BOD Load, lb/d
Jan	1.18	0.33	1.47	420
Feb	1.32	0.37	1.85	526
Mar	1.34	0.38	1.41	401
Apr	1.38	0.39	0.91	260
May	1.37	0.38	0.52	149
Jun	0.98	0.27	0.73	208
Jul	0.86	0.24	1.07	304
Aug	0.72	0.20	1.01	288
Sep	0.61	0.17	0.59	169
Oct	0.53	0.15	0.45	129
Nov	0.59	0.16	0.46	132
Dec	1.12	0.31	1.52	434
Average	1.00	0.28	1.00	285

Section 5

Existing Facilities

Section 5

Existing Facilities

The existing DSPUD wastewater treatment plant includes flow equalization, influent screening, integrated fixed film activated sludge (IFAS) biological treatment, filtration, and disinfection with chlorine gas. Effluent is discharged to the South Yuba River during the wet season and used to irrigate the Soda Springs Ski Area during the dry season. Waste activated sludge is stored during the wet season and processed on drying beds prior to landfill disposal in the summer. The overall plant layout and a flow diagram for existing facilities are shown in Figures 5-1 and 5-2, respectively. Design criteria and a hydraulic profile taken from the 1985 construction drawings are shown in Figure 5-3. Design criteria relating to the internals of the two treatment units have been modified in recent years in conjunction with the conversion to the current IFAS system. Design criteria for the current configuration of the reactor basins are discussed in Section 5.3.

In the remainder of this section, the various treatment and disposal facilities are discussed to establish the basis for consideration of improvements in other sections. The discussion includes the following major subsections:

- Influent Flow Measurement, Sampling, and Equalization Storage
- Headworks
- Biological Treatment
- Effluent Filtration
- Effluent Disinfection
- Emergency Storage and Pumping System
- Chemical Feed Systems
- Outfall to South Yuba River
- Effluent Irrigation Facilities
- Solids Handling Facilities

5.1 INFLUENT FLOW MEASUREMENT, SAMPLING AND EQUALIZATION STORAGE

The main gravity sewer collector pipeline from the combined DSPUD and SLCWD service areas is located in Donner Pass Road on the east side of Interstate 80 and crosses the freeway from east to west suspended from the Caltrans bridge. The raw sewage influent flow meter for the wastewater treatment plant is a 9-inch Parshall flume in a prefabricated fiberglass manhole just east of the freeway. The maximum flow capacity of the Parshall flume is 5.7 Mgal/d. After the flow measurement manhole, the 21-inch gravity sewer transitions to 15-inch and then 14-inch pressure sewer pipes for the remaining run to the wastewater treatment plant.

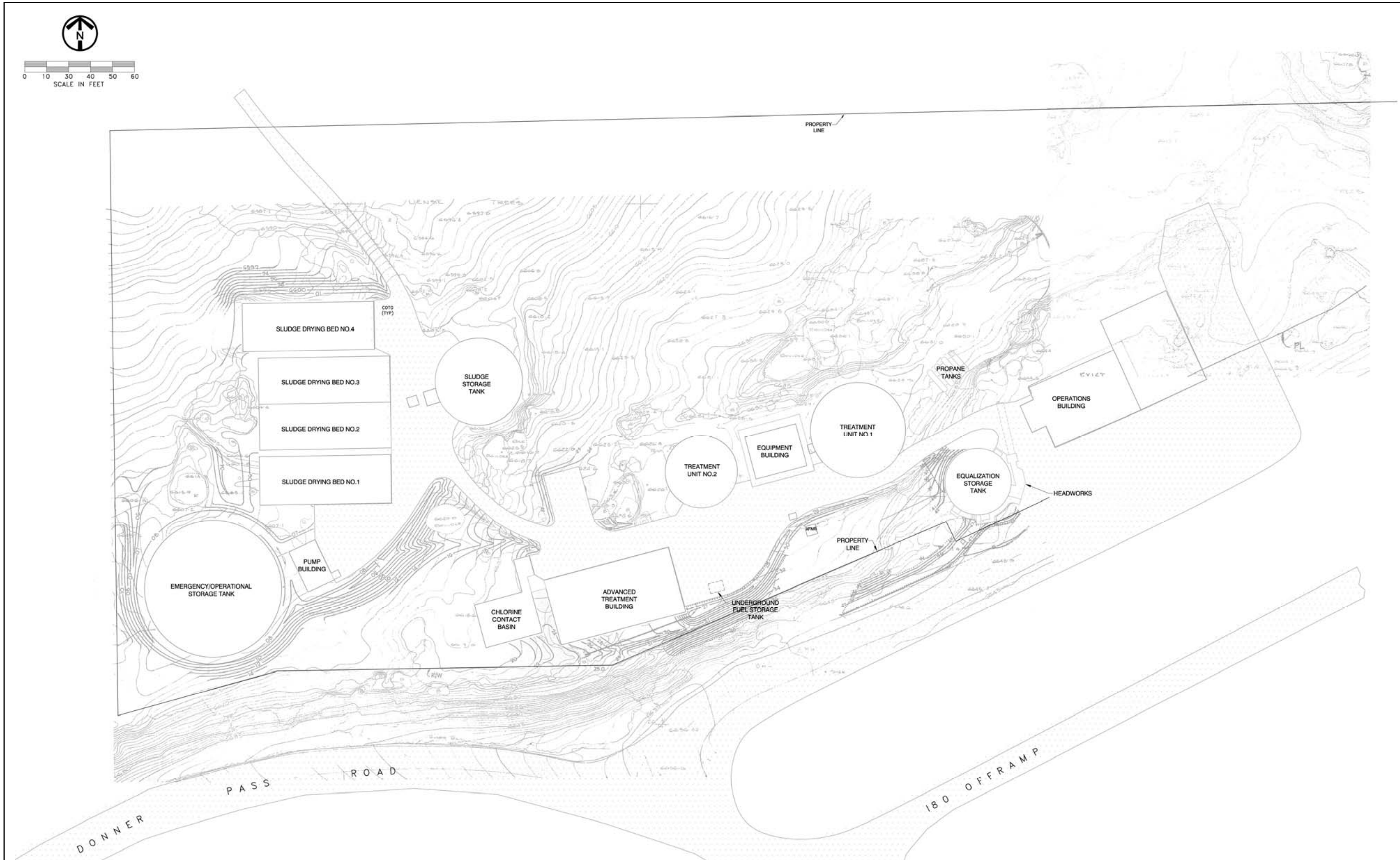


Figure 5-1
Existing Plant Layout

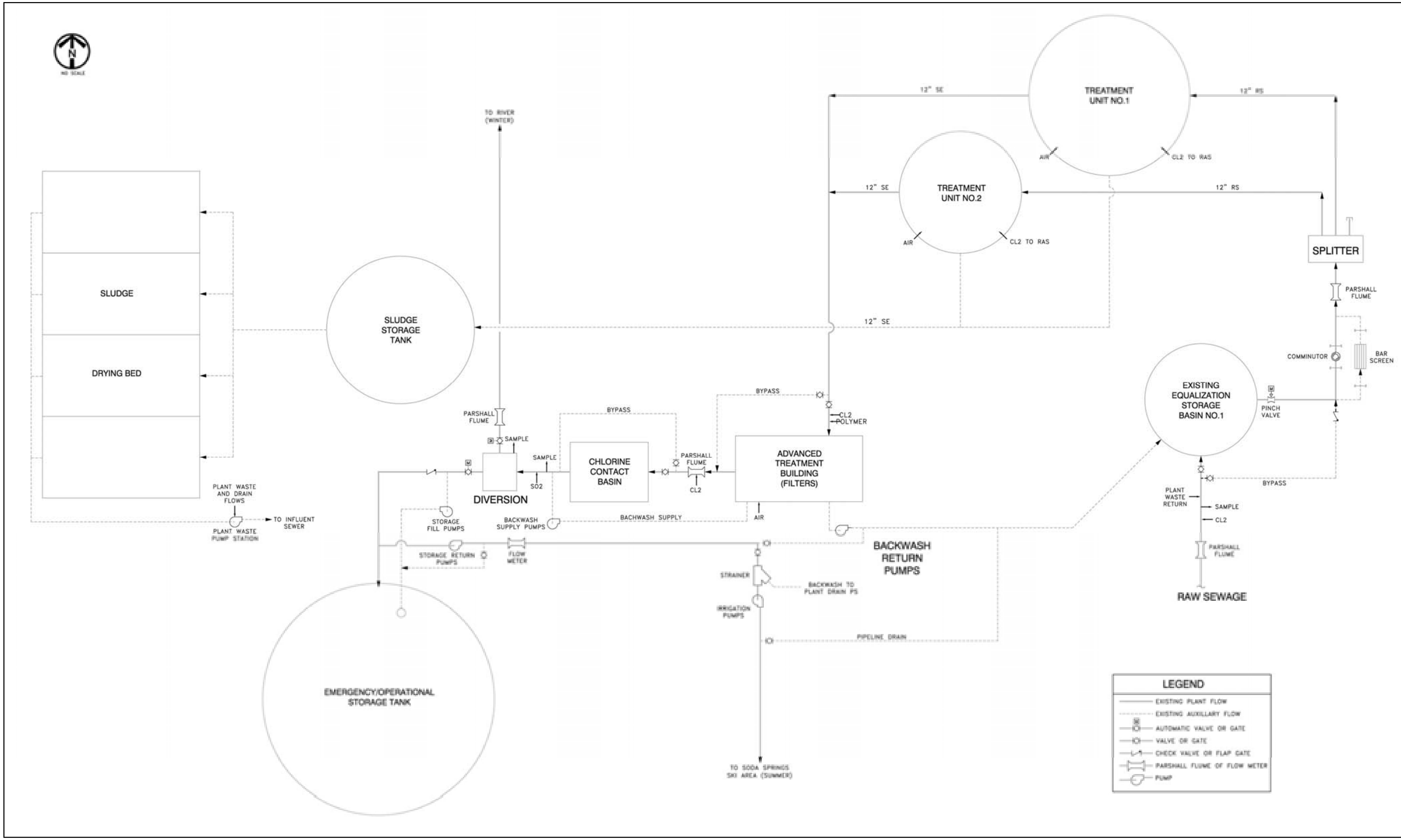


Figure 5-2
Existing Plant Flow Diagram

ITEM	EXIST(a)	PROPOSED	TOTAL
GENERAL DATA			
PEAK 3-DAY DESIGN FLOW(Q), MGD	0.3	0.22	0.52
PLANT HYDRAULIC CAPACITY, MGD	.98	.72	1.7(c)
INFLUENT BOD_5 AND SS, MG/L	--	--	150
INFLUENT BOD_5 AND SS, LOAD, LBS/DAY	375	275	650
MAXIMUM EFFLUENT BOD_5 AND SS, MG/L	--	--	10
EQUALIZATION STORAGE			
VOLUME, MG	0	0.20	0.20
AIR REQUIREMENT, CFM	--	250	250
HEADWORKS			
DESIGN FLOW, MGD	(b)	2.6	2.6
NUMBER COMMINUTORS	(b)	1	1
AERATION			
NUMBER OF BASINS	1	1	2
VOLUME OF BASIN, CF	10,800	7,800	18,600
BOD_5 LOADING, LBS/1000 CF-DAY	35	35	--
MIXED LIQUOR VOLATILE SUSPENDED SOLIDS, MG/L	2,000-3,500	2,000-3,500	--
SOLIDS RETENTION TIME, DAYS	6-12	6-12	--
HYDRAULIC DETENTION TIME @ Q, HRS	6.4	6.4	--
AIR SUPPLIED, CF/LB BOD_5 REMOVED	1,600	1,600	--
AIR REQUIRED, CFM	425	300	725
RETURN SLUDGE PUMPING			
NUMBER OF PUMPS	4	4	8
MAXIMUM RETURN RATE, MGD	.4	.22	.62
AIRLIFT PUMP AIR REQUIREMENT, CFM	140	160	300

ITEM	EXIST(a)	PROPOSED	TOTAL
CLARIFICATION			
NUMBER OF BASINS	1	1	2
NOMINAL DIAMETER, FT	37	32	--
OVERFLOW RATE @ Q, GPD/FT^2	280	275	--
SOLIDS LOADING RATE, $LB/DAY \cdot FT^2$	25	25	--
AEROBIC DIGESTION			
NUMBER OF BASINS	1	1	2
VOLUME OF BASIN, CF	13,200	9,700	22,900
DESIGN SOLIDS LOAD, LBS/DAY	154	113	267
SOLIDS RETENTION TIME, DAYS	120	120	--
BLOWERS (EXCEPT FILTER BACKWASH)			
NUMBER	3	2	5
OUTPUT PRESSURE, PSI			
NO. 1	--	9	--
NO. 2	--	14.5	--
NOS. 3-5	7.0	--	--
HP EACH BLOWER			
NO. 1	--	15	--
NO. 2	--	30	--
NOS. 3-5	30	--	--
MAXIMUM OUTPUT PER BLOWER, INLET CFM			
NO. 1	--	220	--
NO. 2	--	200	--
NOS. 3-5	640	--	--
SLUDGE STORAGE RESERVOIR			
NUMBER	1	0	1
VOLUME, MG	0.6	0	0.6
AIR REQUIREMENT, CFM	100(b)	250	250
SLUDGE DEWATERING BEDS			
NUMBER BEDS	2	2	4
DRYING BED AREA, FT^2	5,600	4,800	10,400
LOADING RATE LBS SOLIDS/FT^2-SEASON	6.4	6.4	--
DESIGN DETENTION TIME, DAYS	12	12	--

ITEM	EXIST(a)	PROPOSED	TOTAL
FILTRATION			
NUMBER OF CELLS	0	3	3
FILTRATION RATE, 1 CELL @ Q, GPM/FT^2	--	5	--
SIZE OF EACH CELL, FT^2	--	72	--
WATER BACKWASH RATE, GPM/FT^2	--	12	--
WATER BACKWASH FLOW, GPM	--	900	900
NUMBER OF BACKWASH PUMPS (TOTAL FOR SUPPLY AND RETURN)	0	3	3
HP EACH PUMP	--	25	--
AIR SCOUR RATE, CFM/FT^2	--	4	--
AIR SUPPLY @ 5 PSI, CFM	--	290	290
NUMBER OF BLOWERS	0	2	2
HP EACH BLOWER	--	10	--
CHLORINATION			
NUMBER CHLORINATORS	1	2	3
CHLORINATOR CAPACITY, LB/D			
PRECHLORINATION	--	150	--
RETURN SLUDGE	--	30	--
FINAL DISINFECTION	70(b)	150	--
SULFONATION			
NUMBER SULFONATORS	0	2	2
CAPACITY OF EACH, LB/D	--	100	--
CHLORINE CONTACT			
NUMBER BASINS	1(b)	1	1
CONTACT TIME AT Q, MIN	--	60	60
AT 1.7 MGD, MIN	--	18	18
BASIN VOLUME, 1000 GAL	--	22	22
PLANT WASTE RETURN PUMPS			
NUMBER PUMPS	2(b)	2	2
CAPACITY EACH PUMP, GPM	150(b)	150	--
HP EACH PUMP	10(b)	20	--

ITEM	EXIST(a)	PROPOSED	TOTAL
EMERGENCY/OPERATIONAL STORAGE			
NUMBER OF TANKS	0	1	1
VOLUME OF TANK, MG	--	1.56	1.56
DAYS STORAGE @ Q	--	3	3
STORAGE FILL PUMPS:			
NUMBER	0	3	3
CAPACITY, EACH, GPM	--	900	--
HP EACH	--	10	--
STORAGE RETURN PUMPS:			
NUMBER	0	2	2
CAPACITY, EACH, GPM	--	600	--
HP EACH	--	15	--
IRRIGATION			
DESIGN FLOW, MGD	--	0.19	0.19
DESIGN APPLICATION, IN/MO	--	4.8	--
APPLICATION RATE, IN/HR	--	0.23	--
IRRIGATED AREA, ACRES	--	45	45
NUMBER IRRIGATION PUMPS	--	2	2
CAPACITY EACH PUMP, GPM	--	600	--
HP EACH PUMP	--	100	--
RECLAIMED WATER			
LOW PRESSURE:			
NUMBER PUMPS	2(b)	2	2
CAPACITY EACH PUMP, GPM	40(b)	70	--
HP EACH PUMP	2(b)	5	--
HIGH PRESSURE:			
NUMBER PUMPS	0	1	1
CAPACITY, GPM	--	100	--
HP	--	10	--

DESIGN DATA NOTES

(a) EXISTING PLANT DESIGN CRITERIA CORRECTED FOR DERATING TO 0.3 MGD

(b) EXISTING UNIT(S) WILL BE REMOVED FROM THIS SERVICE.

(c) EXPANDABLE TO 2.6 MGD.

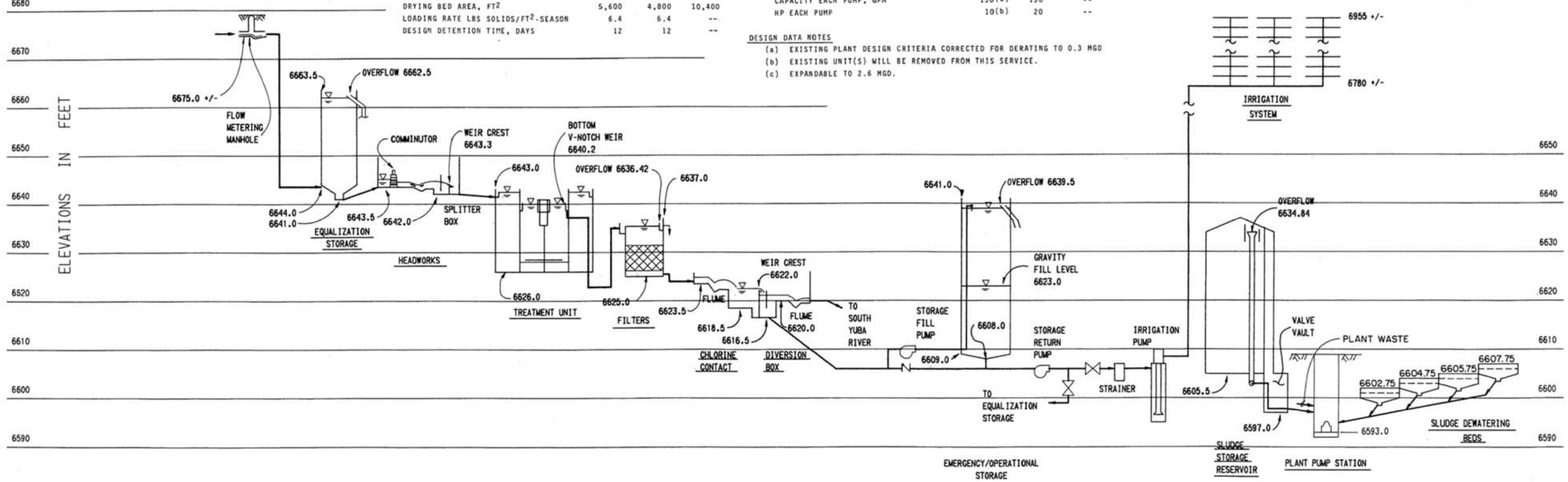


Figure 5-3
Design Data and Hydraulic Profile From 1985 Construction Drawings

The 14-inch influent pressure sewer pipe connects directly to the influent flow equalization tank, with a valved bypass to the plant headworks. The normal flow route is to the equalization tank. The 14-inch influent pipe enters the equalization tank near the bottom. As the wastewater level in the equalization tank rises, the 14-inch pipe becomes surcharged and flows as a full pressure pipe. This creates low flow velocities that result in some solids settling in the pipe. When the equalization tank is drained down, the influent sewer can flow partly full at velocities adequate to carry solids. Apparently, the operation is adequate to prevent problematic solids buildup, as there have been no reported issues with over 20 years in service.

A raw sewage influent composite sampler is located on an elevated platform on the side of the equalization tank, with sample intake tubing extended to the 14-inch pressure sewer. This sampling system apparently did not function as desired and was abandoned by the plant operators in favor of sampling in the headworks. Unfortunately, the headworks location is impacted by plant recycle flows, including effluent filter backwash water.

The existing equalization storage tank is an open-top steel tank with a concrete floor and a total volume of 200,000 gallons. The tank is equipped with floor-mounted aeration diffusers, supplied from a dedicated blower in the Equipment Building. The design air flow to the diffusers is 250 scfm, which keeps the sewage aerated to prevent odors and provides for partial mixing at the rate of 9.3 scfm per 1,000 cubic feet in the full tank (for complete mixing, 20 scfm per 1,000 cubic feet is generally preferred). The equalization tank is equipped with washdown monitors mounted at access platforms near the top of the tank. However, the monitors are never used. The tank is cleaned annually by maintenance staff going inside through a side access manway and using hoses.

Discharge from the equalization tank to the plant headworks is controlled by a motor-operated pinch valve. The valve position is automatically controlled to maintain a flow rate selected by the operator.

The equalization tank was intended to equalize the original design peak 3-day average influent flow of 0.52 Mgal/d.

5.2 HEADWORKS

The plant headworks consists of two concrete flow channels with a combined mechanical grinder/screening system in one channel and a bypass manual bar screen in the other. The mechanical unit has a perforated basket screen with ¼-inch openings and has a maximum flow capacity of 1.8 Mgal/d. Collected screenings are conveyed out of the basket screen with an auger and are washed and compacted prior to being deposited in a garbage can for subsequent collection and landfill disposal.

Downstream from the screens, the two screening channels converge into a single channel with a 9-inch Parshall flume (capacity 5.7 Mgal/d) that is used to measure the discharge flow to the downstream biological treatment facilities from the equalization storage tank or from the influent sewer if the equalization tank is out of service. The discharge from the flume goes into a splitter box that is used to split the flow to the two existing biological treatment units. 58 percent of the flow goes to Plant 1 and 42 percent goes to Plant 2, according to the original design capacities of the two plants. Cipolletti weirs of appropriate length are used to effect the flow split. The splitter box includes a third, currently unused outlet compartment for a future treatment unit.

5.3 BIOLOGICAL TREATMENT

The biological treatment system is provided in two circular steel package plants that were originally designed as activated sludge systems without provisions for ammonia removal (nitrification) or nitrate removal (denitrification), because there were no ammonia or nitrate limits at that time. Each plant originally included an activated sludge aeration basin and an aerobic digester in an annulus with a secondary clarifier in the center. During 2002 through 2006, the plants were upgraded from activated sludge to IFAS. In each plant, the aeration basin and digester was reconfigured into an anoxic basin, an aeration basin and a much smaller digester. In 2009, the digester basins were converted to additional anoxic volume. Webbing material supported on stainless steel frames was added in the anoxic and aeration basins to support attached biological growth in addition to suspended growth (previously, there was only suspended growth in the aeration basins). Mixed liquor recirculation pumps and anoxic basin mixers were also added. The IFAS system was designed and provided by Brentwood Industries and is called the AccuWeb system.

Approximate dimensions and volumes for the various portions of Plants 1 and 2 as they currently exist are shown in Table 5-1.

Table 5-1
Dimensions and Volumes for Plants 1 and 2 (a)

Item	Plant 1	Plant 2
Outer Tank Inside Diameter, ft	60.0	51.13
Clarifier Inside Diameter, ft	37.5	32.0
Reactor Basin Depth, ft	15.67	14.75
Clarifier Depth, ft	12.0	12.0
Anoxic Volume, Mgal	64,000	34,000
Aerobic Volume, Mgal	138,000	104,000
Total Reactor Volume, Mgal	202,000	138,000
Clarifier Volume, Mgal	99,000	72,000
Total Plant Volume, Mgal	301,000	210,000
Number AccuWeb Frames, Anoxic (b)	2	1
Number AccuWeb Frames, Aerobic (b)	15	12
Total AccuWeb Fabric Area, Anoxic, ft^2 (b)	17,377	11,666
Total AccuWeb Fabric Area, Aerobic, ft^2 (b)	130,331	69,995

(a) Data shown are approximate, as design drawings are not available for the current configuration.
(b) From Brentwood design criteria.

The AccuWeb system was designed for the purpose of providing nitrification and denitrification to meet monthly average effluent ammonia-N and nitrate-N concentrations of 5 and 10 mg/L, respectively. The first AccuWeb installation in a portion of Plant 2 (one of the steel package plants), constructed in 2002, was a demonstration project with a design capacity of 144,000 gpd. The District proceeded with the subsequent installations to complete the retrofits of Plants 1 and2 in 2005 and 2006, however, a firm capacity for these improvements has not been established.

The existing IFAS system frequently has not been able to meet the design objectives for effluent ammonia and nitrate. During the low load periods in the fall, the plant is frequently able to meet the original 5 mg/L ammonia-N limit, but nitrate-N concentrations are frequently above 20 mg/L. With the onset of peak ski season flows and low winter temperatures, effluent ammonia-N concentrations are frequently above 20 mg/L. During these times, nitrate-N concentrations may be below 10 mg/L, but this is mainly due to the lack of nitrification, not to successful denitrification.

It is believed that the high ammonia and nitrate concentrations may be due to several factors:

- High flow and load variability in general.
- Flows and loads are relatively low in the fall, but then they increase dramatically with the onset of peak skiing conditions around Christmas and sporadically around holiday weekends thereafter. It is believed that a sufficient population of nitrifiers cannot be developed during low loads to handle the sudden onset of high loads, particularly since the transition occurs with low mixed liquor temperatures (less than 8 °C).
- Potentially inadequately sized reactor basins and IFAS media.
- Low ratio of BOD/TKN and lack of adequate readily biodegradable substrate to support denitrification.

Another issue with the existing biological treatment system is that there can be excessive solids carryover from the secondary clarifier during high flow events with cold temperatures. Polymer is frequently added to the secondary clarifiers to help, but still loadings passed forward to the downstream filters can be excessive, causing calls for almost continuous backwashing.

5.4 EFFLUENT FILTRATION

The effluent filtration system includes three rectangular filter cells in one open-top, prefabricated steel structure, located in the Advanced Treatment Building. The filtration medium is a deep bed of coarse anthracite (4 feet of 1.5 mm anthracite). A combined air and water backwash system is used, with the anthracite media being retained using overlapping arcuous baffles on both sides of the backwash troughs.

The filtration system was designed so that only one cell would need to be in service to handle the original design equalized peak 3-day flow of 0.52 Mgal/d. A second cell could be in backwash and a third cell was provided for redundancy.

Backwash water is provided to the filtration system from the effluent of the chlorine contact basin through an inlet screen with 1 mm openings. There are three identical self-priming centrifugal pumps included in the backwash system, one for backwash supply, one to pump the spent backwash water to the equalization storage tank, and a third pump on standby.

When a filter backwash is initiated, the chlorine contact basin is drained down, causing a temporary interruption in plant effluent flows until the backwash is completed and the chlorine contact basin re-fills. These flow interruptions cause problems with the sulfur dioxide feed system (sulfur dioxide is added after the chlorine contact basin to remove residual chlorine before discharge).

Polymer or other filtration aid chemical can be injected into the filter influent pipeline at an in-line motorized flash mixer. There is no formal flocculation basin, but some flocculation can occur passively in the filter influent piping and in the water pool above the filter media.

5.5 EFFLUENT DISINFECTION

The effluent is disinfected using gaseous chlorine from 150 pound cylinders. Dechlorination is by sulfur dioxide, also from 150 pound cylinders. For each gas, there is one group of six cylinders manifolded together for active use and another groups of six cylinders manifolded together on standby and ready for use through an automatic switchover system. All of the chlorine and sulfur dioxide feed and control facilities were recently upgraded and, according to plant staff, function very well. However, there are no automatic shutoff valves on the cylinders and the storage and feed facilities are not protected with a containment and scrubber system in case of a leak. It is noted that a leak in the connection at any cylinder or in the piping that connects them could result in the escape of the contents of all six cylinders in a group; i.e., up to 900 pounds of chlorine or sulfur dioxide.

The existing chlorine contact tank includes three looped (up and back) channels, with each channel being 4 feet wide, about 4 to 4.5 feet deep (depending on flow and water level over the outlet weir) and 30 feet long on each leg of the loop (60 feet total). The loops are operated in series, for a total length of 180 feet and a length to width ratio of 45 to 1. Any one of the looped channels can be taken out of service for cleaning while retaining use of the other two via a connecting channel at the head of the basin. The design criteria for the basin indicated on the 1985 design drawings indicate a volume of 22,000 gallons and a contact time of 60 minutes at the design flow of 0.52 Mgal/d. Allowing for increased water depth due to head over the outlet weir and allowing for volume in inlet and outlet channels, the effective volume and contact time are somewhat higher. The basin was designed with knockout end walls and stubbed-out floor and sidewalls to allow easy expansion by extending the length of the channels.

According to plant staff, a chlorine dose of about 6 mg/L is used to attain a final residual of 2 mg/L at the end of the chlorine contact basin, and this is adequate to reliably meet the permitted total coliform limit of 2.2 MPN/1,000 ml. Sulfur dioxide is fed to produce an excess of 2 mg/L to assure reliable dechlorination.

5.6 EMERGENCY STORAGE AND PUMPING SYSTEM

There is a 1.56 Mgal open-top steel storage tank, which was originally designed to provide emergency storage for three days at the equalized design flow of 0.52 Mgal/d, in the event that the plant effluent did not meet discharge requirements. The plant effluent can be diverted to storage at the outlet of the chlorine contact basin. The emergency storage tank is also used as the operational storage tank for irrigation disposal in the summer months.

Although the emergency storage tank can be filled almost to half of its capacity by gravity flow, pumping is needed to fill the remainder. There are two duty and one standby pumps for filling the emergency storage tank, each rated for 900 gpm, for a reliable capacity of 2.6 Mgal/d. There is one duty and one standby storage return pump, each rated for 600 gpm, for a reliable return flow capacity of 0.86 Mgal/d. The return flow is routed to the influent flow equalization tank.

5.7 CHEMICAL FEED SYSTEMS

Besides the chlorine and sulfur dioxide feed systems, which were discussed as part of the disinfection system, there are existing permanent chemical feed systems for ammonia gas and for soda ash.

Ammonia gas is used to supplement influent ammonia concentrations during low load periods as needed to increase the population of bacteria that remove ammonia (nitrifiers) to levels that are high enough to handle peak influent ammonia loads. Currently, there are six 150 lb ammonia cylinders connected to feed Plant 1 and four for Plant 2. Assuming a feed capacity of 32 lb/d per cylinder, the feed capacities to Plants 1 and 2 are 192 and 128 lb/d, respectively.

The soda ash feed system includes a storage silo, dry feeders, slurry tank, slurry pumps and related facilities. Soda ash is used to supplement influent alkalinity, as needed for nitrification and disinfection. The existing silo can hold approximately 35 tons of soda ash. There is one duty and one standby soda ash slurry pump, each rated at 5 gpm. Assuming a maximum soda ash solution strength of 10 percent, the maximum soda ash feed rate per pump is about 6,000 lb/d.

5.8 OUTFALL TO THE SOUTH YUBA RIVER

The plant effluent pipeline is a combination of 10-inch and 8-inch piping, extending approximately 4,000 feet northwest from the wastewater treatment plant to an outfall along the South Yuba River. The outfall is a perforated pipe in a pile of rocks on the southern bank of the river. The plant effluent flows through the rocks into the river.

5.9 EFFLUENT IRRIGATION FACILITIES

During the summer months, plant effluent is stored in the existing Emergency/Operational Storage Tank until is pumped to an irrigation system on the slopes of the Soda Springs Ski Area. DSPUD has a lease allowing for the existence and operation of the irrigation and related facilities.

Pumping to the ski area irrigation system is accomplished in two stages, with one duty and one standby pump, each rated for 600 gpm, in each stage. The first-stage pumps are the storage return pumps, previously mentioned. From the first-stage pumps, the effluent is routed through an automatic self-cleaning strainer and then to the second-stage pumps. The pumping system responds to calls from irrigation controllers at the Soda Springs Ski Area.

The existing irrigation system covers approximately 45 acres and is divided into four pressure zones, extending up the slopes. However, the lowest pressure zone is not used, because the area stays too wet; therefore, the useful area is about 34 acres. Pumping pressure is as needed, depending on which pressure zone is in operation.

5.10 SOLIDS HANDLING FACILITIES

At the present time, all waste activated sludge from the two treatment units is discharged into an existing 600,000 gallon solids storage tank. The sludge is stored throughout the wet season and then is discharged to sludge drying beds in the dry season. The tank is decanted a few times during the year to thicken the sludge concentration. There are coarse bubble diffusers, supplied from an existing blower with a capacity of 250 scfm to aerate the tank.

There are four sludge drying beds, with a total area of 10,400 square feet. The beds have a top layer of sand over an underdrain system. Concrete runners are located in the bed to allow easy dried sludge removal using a front-end loader.

Currently, dried sludge is hauled to a landfill near Sparks, Nevada.

Section 6

Waste Discharge and Treatment Requirements

Section 6

Waste Discharge and Treatment Requirements

The DSPUD wastewater treatment plant effluent is discharged to the South Yuba River during the wet season and when discharges to land are not possible due to snow cover or wet soils. During the dry season, when possible, the effluent is used for irrigation of the Soda Springs Ski Area. Both of these methods of disposal are regulated under a National Pollution Discharge Elimination System (NPDES) permit and waste discharge requirements adopted by the California Regional Water Quality Control Board, Central Valley Region. The permit is updated approximately every five years. The current permit was adopted on April 24, 2009 (Order No. R5-2009-0034, NPDES No. CA0081621).

Key requirements of the permit were reviewed in the Donner Summit Public Utility District Preliminary Investigation of Wastewater Management Options, dated June 10, 2009, which is included herewith as Appendix B. Key permit requirements, together with information on the performance of the existing plant and permit compliance strategies, are summarized in Table 6-1, which is excerpted from the previous document. The reader is referred to Appendix B and the permit itself for more detailed information. The full permit is considered to be too voluminous to be bound with this document, but can be viewed on the DSPUD website or the website of the Central Valley Regional Water Quality Control Board. Alternatively, a hard copy of the permit can be viewed at the Regional Board office in Sacramento or at the DSPUD office in Soda Springs.

Some of the most problematic requirements of the permit are those for monthly average effluent ammonia-nitrogen and nitrate-nitrogen concentrations of 2.1 and 10 mg/L, respectively, for discharge to the South Yuba River. These are considered problematic because the existing plant does not comply and substantial improvements will be required to attain compliance. Another troublesome permit requirement is that the discharge cannot cause water in the South Yuba River to contain biostimulatory substances that promote aquatic growths in concentrations that cause nuisance or adversely affect beneficial uses. This is troublesome because there were nuisance growths of algae in the river downstream from the DSPUD discharge in June 2008 and the discharge may have been a contributory factor. The current permit requires DSPUD to study this issue.

Methods to obtain compliance with the current discharge permit are discussed in the remainder of this Facilities Plan.

Table 6-1
Key NPDES Permit Requirements, Plant Performance and Compliance Strategy

Parameter	Units	Effluent Limits[a]	Existing Plant Performance[b]	Compliance Strategy
BOD	mg/L	10/15/30	Generally compliant.	Continue/improve biological treatment, coagulation and filtration.
pH	Units	6.5 to 8.0[c]	Generally compliant.	Automatic chemical addition for alkalinity and pH control.
TSS	mg/L	10/15/30	Generally compliant.	Continue/improve biological treatment, coagulation and filtration.
Aluminum	µg/L	71/--/143	Frequently noncompliant. (---, ---, 620, 1310, 38.4, 127)	Monitor acid soluble aluminum. Possible Water Effects Ratio (WER).
Ammonia-N	mg/L	2.1/--/5.6	Frequently noncompliant. (Frequent non-certified lab data over 25 mg/L)	Improved treatment required.
Copper	µg/L	1.5/--/3.1	Frequently noncompliant. (4, 4, 7.8, 4.2, 5.9, 6)	Increase effluent hardness by using lime instead of soda ash for required alkalinity addition. Consider increased potable water pH. Possible Water Effects Ratio (WER).
Cyanide	µg/L	4.3/--/8.5	Occasionally noncompliant. (23, <2, 33, <2, DNQ 4, <2)	Evaluate future monitoring results. Consider changing from chlorine to UV disinfection. Consider immediate on-site testing without sample preservation.
Aldrin	µg/L	ND(d)	Rare noncompliance. (<0.002, <0.002, <0.002, DNQ 0.005, <0.002, <0.0028)	Evaluate future monitoring. Public education, source control if needed.
Alpha BHC	µg/L	ND(d)	Rare noncompliance. (<0.005, <0.005, 0.044, <0.005, <0.005, <0.00034)	Evaluate future monitoring. Public education, source control if needed.
Dichlorobromomethane	µg/L	0.56/--/1.2	Uncertain (e). (<0.5, <0.5, <0.5, DNQ 0.3, 1.2, 0.2)	Violations of this chlorine disinfection byproduct will be more likely with complete nitrification. Consider dilution credit, chloramination, UV disinfection.
Nitrate-N	mg/L	10/--/--	Frequently noncompliant. (Frequent non-certified lab data over 15 mg/L. Would be worse with good nitrification.)	Improved treatment required.
Silver	µg/L	0.23[d]	Rare noncompliance. (<0.09, <0.08, 0.26, 0.18, < 0.1, <0.12)	Evaluate future monitoring. Public education, source control if needed.
Zinc	µg/L	15/--/30	Frequently noncompliant. (22, 33, 22, 23.6, 25.3, 30.8)	Increase effluent hardness by using lime instead of soda ash for required alkalinity addition. Consider increased potable water pH. Possible Water Effects Ratio (WER).

Parameter	Units	Effluent Limits[a]	Existing Plant Performance[b]	Compliance Strategy
Manganese	mg/L	50[f]	Possible noncompliance. (---, ---, 8.7, 8.3, 52.8, 88.4)	Evaluate future monitoring and consider manganese removal in treatment process evaluations.
Total Coliform	MPN/100 mL	2.2, 23, 240[g]	Generally compliant.	Continue/improve biological treatment, coagulation, filtration, and disinfection.
Turbidity	NTU	2, 5, 10[h]	Generally compliant.	Continue/improve biological treatment, coagulation and filtration.

[a] Unless indicated otherwise, limits are Average Monthly/Average Weekly/Maximum Daily.

[b] Where a series of six results are shown in parenthesis, they are from special California Toxics Rule and related grab samples taken in June 2001, April 2002, November 2003, February 2004, December 2005, and December 2006, respectively. "DNQ" indicates an estimated value that is below the method quantitation limit, which is indicated after "DNQ".

[c] Range is based on instantaneous minimum and instantaneous maximum.

[d] Instantaneous maximum.

[e] Dichlorobromomethane is a chlorine disinfection byproduct that is mitigated by the presence of ammonia. Ammonia concentrations at the time of historical sampling are unknown.

[f] Annual average.

[g] 2.2 weekly median, 23 once in 30 days, 240 at any time.

[h] 2 daily average, 5 more than 5% of time in 24 hours, 10 at any time.

Section 7

Development and Screening Alternatives

Section 7

Development and Screening Alternatives

Prior to embarking on this Facilities Plan, DSPUD authorized the preparation of a separate study to identify and screen various alternatives for attaining compliance with the existing discharge permit and other applicable water quality regulations and objectives. That study resulted in a report entitled "Donner Summit Public Utility District Preliminary Investigation of Wastewater Management Options," dated June 10, 2009, which is included herewith as Appendix C.

Shown in Table 7-1 is a summary of all of the combined wastewater treatment and disposal options that were considered in the study mentioned above, including ratings applied to each option based on anticipated cost, reliability, ease of implementation, and environmental impacts.

Based on the previous study and the results summarized in Table 7-1, as well as further considerations by DSPUD and SLCWD, DSPUD determined that this Facilities Plan should investigate only the disposal option of wet season discharge to the South Yuba River, combined with seasonal storage (to mitigate biostimulation) and dry season irrigation. Furthermore, the following biological treatment alternatives were authorized for investigation:

- Upgrade the Existing Integrated Fixed Film Activated Sludge (IFAS) System, 2-Stage
- Upgrade the Existing IFAS System, 4-Stage
- New IFAS, 4-Stage
- Submerged Attached Growth
- Membrane Bioreactor (MBR), 4-Stage

These biological treatment options are considered in Section 9.

Table 7-1
Overall Wastewater Management Options

Disposal Option	Treatment Option	Cost	Reliability	Ease of Implementation	Environmental Impact	Further Consideration
Subsurface	Unknown	0	0	-	0	No
Wet Season Storage, Dry Season Irrigation	Secondary	-	+	-	-	No
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Upgrade Existing IFAS 2-Stage	+	-	+	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Upgrade Existing IFAS 4-Stage	+	-	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Upgrade Existing IFAS 2-Stage, Denitrification Filter	+	-	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	New IFAS 4-Stage	0	+	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	New IFAS 2-Stage, Denitrification Filter	0	+	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Submerged Attached Growth	0	+	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	MBR 4-Stage	0	+	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Non-Nitrifying Activated Sludge, Ion Exchange for Ammonia	0	-	-	-	No
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Non-Nitrifying Activated Sludge, Breakpoint Chlorination for Ammonia	+	-	0	-	No
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Upgrade Existing IFAS 2-Stage, Ion Exchange and Breakpoint Chlorination for Supplemental Ammonia Removal, Denitrification Filter for Supplemental Nitrate Removal	-	-	-	-	No
Wet Season Discharge to SYR, Dry Season Irrigation, No Seasonal Storage	Undetermined Enhanced Nutrient Removal System	-	-	-	0	No
Year-Round Discharge to SYR	Undetermined Extreme Treatment	-	-	-	-	No
Export Raw Sewage to TTSA	None	0	+	-	-	No
Export Treated Effluent to TTSA	Undetermined Enhanced Nutrient Removal System	-	-	-	-	No

Section 8

Influent Flow Equalization and Plant Headworks

Section 8
Influent Flow Equalization and Plant Headworks

Because of the large variability in flows at the DSPUD wastewater treatment plant, substantial influent flow equalization is recommended. Flow equalization reduces the maximum flow that must be treated in the plant (except for potential emergency hydraulic capacity considerations) and provides stability in the biological treatment process, which will make the plant more reliable in meeting discharge requirements. Equalization also provides for increased ease of operation. In this section, the sizing of equalization storage is considered first, followed by an investigation and analysis regarding site issues and the type of tank and ancillary facilities to be provided. Because the plant headworks and equalization facilities are interrelated, possible headworks modifications are also considered.

8.1 EQUALIZATION STORAGE VOLUME REQUIREMENTS AND EQUALIZATION CONTROLS

The sizes of equalization storage facilities that would have been required to trim peak flows through the plant to various levels during actual peak flow events from January 2001 through April 2008 were determined in Technical Memorandum 2 (TM2), which is included herewith as Appendix B. Key results from that analysis are summarized in Figure 8-1, which shows the storage volume required versus the limiting flow for the first, second, and third largest peak flow events in the period analyzed.

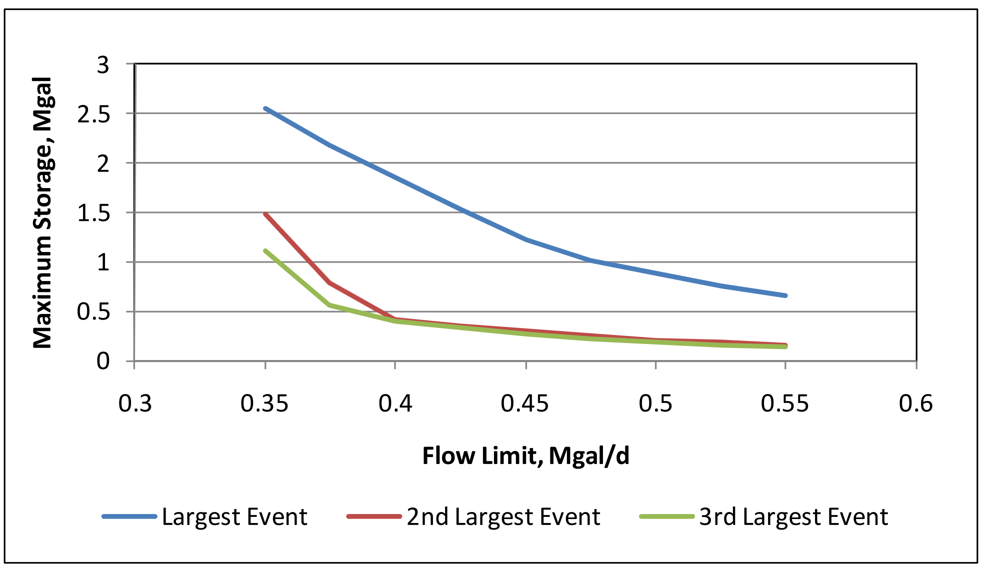


Figure 8-1
Storage Volume Versus Limiting Flow for Events in 2001 through April 2008

As discussed in TM2 (Appendix B), the most severe flow event in the period of analysis occurred near the end of December 2005 and the beginning of January 2006. That period included the highest weekly average flow (0.61 Mgal/d) and the highest daily average flow (0.97 Mgal/d) recorded at the plant. These flows were the result of a warm winter storm that dumped 14.4 inches of rain in 16 days and 4.25 inches in one day (measured in Truckee). These correspond to return frequencies of roughly 11 and 18 years, respectively. The high flows included large amounts of infiltration and inflow combined with high domestic flows for this peak occupancy period. The unusually high magnitude of this 2005/2006 peak flow event is evidenced by the difference in equalization storage requirements for that event (the largest) and the second and third largest events indicated in Figure 8-1. As suggested in TM2, design based on the largest event could be considered to include a reasonable safety factor. It is hoped that DSPUD and SLCWD can mitigate some of the infiltration and inflow sources that contributed to these peak flows, thus contributing to the safety factor.

Also noted in TM2 is the fact that the storage volumes determined in the theoretical analysis are "active" storage volumes and do not include water stored below a minimum depth required for mixing and/or aeration.

Further insight on the sizing and use of equalization storage can be gained by considering the daily flows occurring during the ten most severe weekly average flow events in the period of analysis, which are shown in Figure 8-2. As previously noted, the weekly average flow for the most severe event was 0.61 Mgal/d. From Figure 8-2, it can be seen that this flow rate was exceeded as a daily average flow in only three of the peak weeks analyzed, and two of these were part of the peak flows occurring near the end of December 2005 and the beginning of January 2006. The third event in which a daily average flow greater than 0.61 Mgal/d was recorded was just two months later, near the end of February 2006. From these data, it is clear that if adequate equalization storage were available at the time and were used only to trim flows in excess of the peak weekly average flow of 0.61 Mgal/d, the equalization storage tank would have been used to carry over peak flows from one day to the next on only three occasions in the entire eight years (approximate) analyzed. From Figure 8-1, it can be seen that the theoretical storage volume required for this limiting flow in the 2005/2006 event would have been about 500,000 gallons. Obviously, with so little usage, the valuable storage volume would be largely wasted.

From the above, it is clear that equalization storage sized based on a certain limiting flow for one or more peak flow events, must be used during other flow events with much lower limiting flows. In fact, to fully realize the benefits of equalization storage, the storage volume should be used, to the maximum extent possible, to provide uniform flow or minimal flow variations at all times and under all influent flow conditions. For example, the hypothetical 500,000 gallons of storage that would have been needed to limit flows to 0.61 Mgal/d in the severe 2005/2006 event considered above would have been adequate to allow limiting the peak flow to about 0.4 Mgal/d in the second and third most severe events analyzed for 2001 through April 2008 (Figure 8-1). For less severe events, even lower limiting flows would have been possible and desirable.

In actual practice, the benefits of the theoretical volume requirements shown in Figure 8-1 and discussed above cannot be fully realized. This topic is discussed in TM2, resulting in recommendations for a substantial safety factor on theoretical storage requirements and for emergency peak flow provisions (above the normal treatment capacity of the plant) and/or emergency storage of excess peak flows. Additionally, it is beneficial to provide an automatic control system to dynamically adjust plant flows based on current data. For example, in a particular event, the operator may establish a plant flow rate of 0.4 Mgal/d. The automatic control system would be programmed to gradually increase this flow as the storage volume increased above a certain level. Similarly, the automatic control system would be programmed to gradually decrease the plant flow as the storage volume decreased below a certain level.

As an example of emergency provisions for high flows, the DSPUD plant design in 1985 was based on an equalized peak 3-day flow of 0.52 Mgal/d. However, the plant was designed to hydraulically pass the projected peak hour flow rate of 1.7 Mgal/d, in case the equalization storage tank was prematurely filled. In the event of such high emergency flows, treatment performance could be severely impacted, including the need to partially or fully bypass the filters. Any noncompliant final effluent could be routed to the emergency storage tank (for storage and subsequent re-treatment) until that tank is filled, but then would have to go to the river discharge.

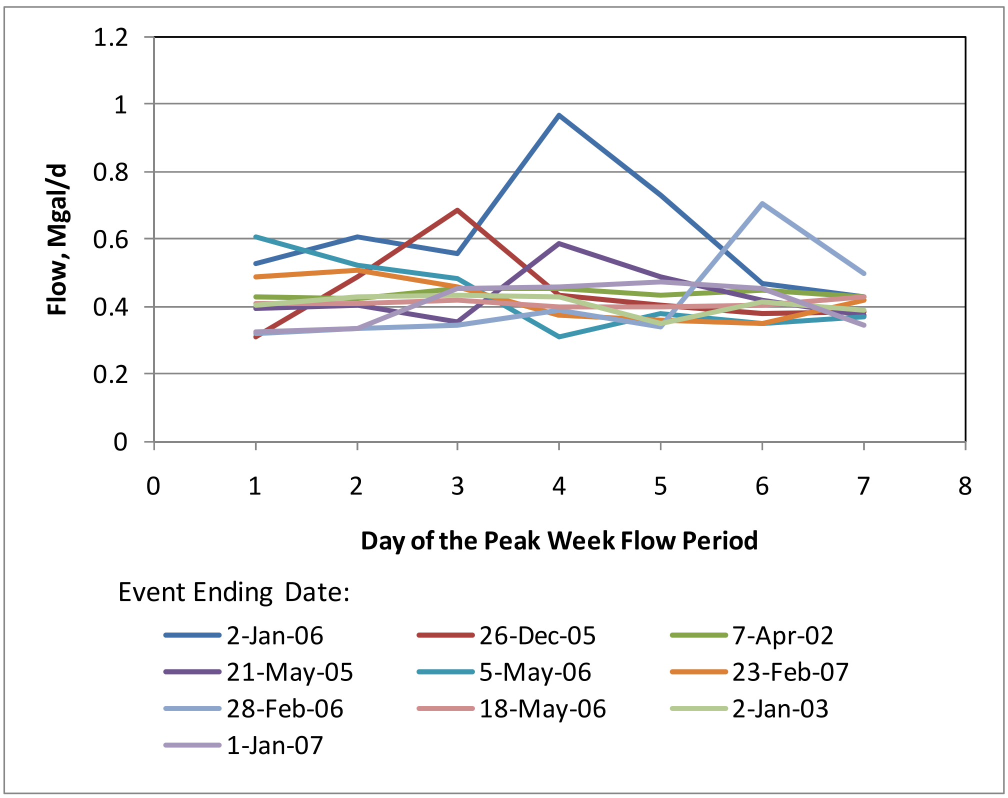


Figure 8-2
Daily Flows for the Ten Highest Weekly Average Flow Events

8.2 ECONOMIC BENEFITS OF EQUALIZATION STORAGE

Since equalization storage will allow a reduction in the “normal” (not emergency) peak flow rate through the plant, treatment cost savings can be realized to the extent that the reduced peak flows allow downsizing of treatment facilities. Facilities that potentially could be downsized based on equalization storage include secondary clarifiers, effluent filters, and disinfection facilities. If a membrane bioreactor were to be employed as a biological treatment system, there would be no clarifier or effluent filters, but the membrane filtration facilities could be downsized. Plant facilities that generally could not be downsized based on equalization storage include biological reactor basins and sludge handling facilities. Typically, biological reactor basins are sized based on average loading over about a month. Considering the high load variations at DSPUD, average loading over a week or two may be more appropriate; but this still would not allow a prudent reduction in sizing with reasonable equalization storage durations and volumes. Sludge handling facilities are sized based on average loading conditions over a month or more and cannot be downsized with equalization storage. Similarly, effluent storage and disposal facilities would not be impacted by equalization storage.

An approximate analysis of the incremental costs and benefits of equalization storage for various peak flow limitations is presented in Table 8-1. The base condition indicated in Table 8-1 is a limiting flow of 0.97 Mgal/d, which is the current peak day flow. Although no specific analysis of the diurnal flow pattern on the peak day was accomplished, it is estimated that the peak day flow could be equalized with a theoretical storage volume of 20 percent of that daily flow, or about 0.20 Mgal. The storage requirements for all other limiting flows were taken from an extended and more detailed analysis of the largest event in Figure 8-1, which is shown in Figure 8-3. For each step in capacity limitation and equalization volume, the incremental storage cost, incremental hydraulic capacity cost savings and incremental benefit:cost ratio are shown. With a benefit:cost ratio of 5.51, there is a large economic advantage for decreasing the flow limit from 0.97 to 0.70 Mgal/d. The benefit:cost ratio for each of the next two increments, taking the limiting flow to 0.65 and 0.60 Mgal/d, is 1.0, meaning both increments would cost as much as they would save. Subsequent increments in equalization storage would cost more than they would save.

As noted previously, equalization storage provides performance reliability and ease of operation, which cannot be easily translated directly into economic benefits. Therefore, it is certainly desirable to proceed with any increment with a benefit:cost ratio of 1.0 or greater, and it could be beneficial overall to proceed with additional increments, even when the benefit cost ratio is below 1. Furthermore, it is acknowledged that the analysis presented in Table 8-1 is approximate only and is based on the flow statistics for one peak flow event near the end of 2005 and beginning of 2006, which is presumed to represent a reasonable safety factor. Therefore, there is considerable room for subjective judgment that could result in different volumes from those suggested herein. Nevertheless, based on this analysis and on engineering judgment, it is suggested that equalization storage in this Facilities Plan should be based on a limiting flow of 0.60 Mgal/d, requiring an active storage volume of 0.50 Mgal, both based on current flows. Since future flows will be about 1.2 times current flows (from Table 4-1), both the limiting flow and the equalization storage volume should be adjusted accordingly, resulting in a limiting flow of 0.72 Mgal/d and an active equalization storage volume of 0.60 Mgal. It is noted that adjustments to future conditions for each of the limiting flows considered in Table 8-1 would require the same 1.2 factor to be applied to both the limiting flow and the equalization storage volume; therefore, there would be no change in the benefit:cost ratio.

Assuming the minimum water level to facilitate mixing and/or aeration in equalization storage tanks would represent 20 percent of the total tank volume, the total volume requirement would be 125 percent of the active volume requirement, or in this case, 0.75 Mgal. From this total, the current equalization volume of 0.2 Mgal could be subtracted, resulting in an additional volume requirement of 0.55 Mgal, if the existing equalization tank is retained as such.

Table 8-1
Economic Costs and Benefits of Equalization Storage

Flow Limit, Mgal/d	Maximum Equalization Storage Volume[a] Mgal	Incremental Equalization Storage cost[b], $M	Incremental Hydraulic Capacity Reduction, Mgal/d	Incremental Hydraulic Capacity Cost Savings[c], $M	Incremental Benefit:Cost Ratio
0.97	0.20	--	--	--	--
0.70	0.30	0.20	0.27	1.08	5.51
0.65	0.40	0.20	0.05	0.20	1.00
0.60	0.50	0.20	0.05	0.20	1.00
0.55	0.66	0.32	0.05	0.20	0.62
0.50	0.89	0.45	0.05	0.20	0.44

(a) Storage for 0.97 Mgal/d flow estimated at 0.2 Mgal. All other volumes based on Figure 8-3.
(b) Equalization storage cost based on $2 per gallon, active volume.
(c) Hydraulic capacity cost based on $4 per gpd for downsized components.

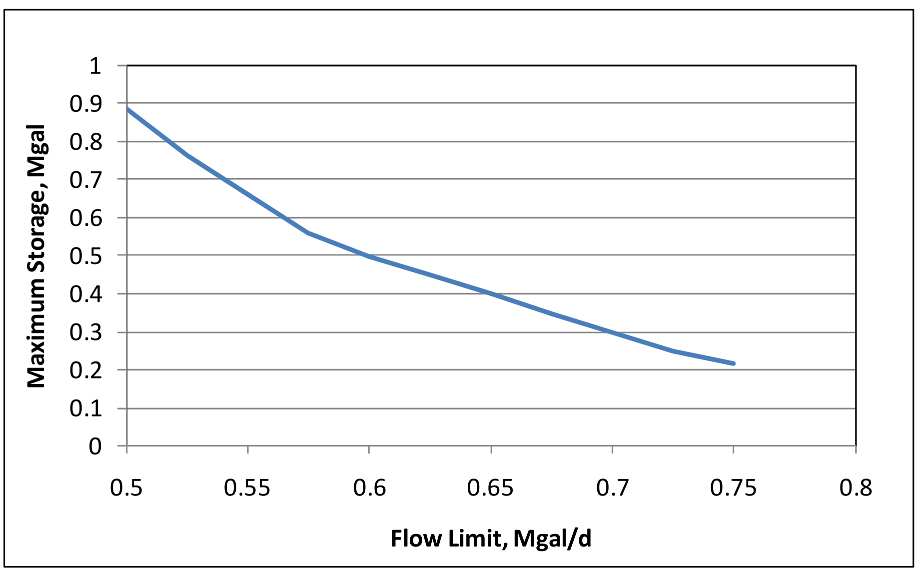


Figure 8-3
Enhanced Storage Volume Versus Limiting Flow for Largest Event

8.3 EQUALIZATION STORAGE SITE AND CONFIGURATION ANALYSIS

In this section, the location and configuration of the proposed equalization storage and ancillary facilities are discussed. Before considering the new facilities, however, it is helpful to discuss the existing system.

8.3.1 EXISTING EQUALIZATION STORAGE BASIN AND RELATED FACILITIES

The existing equalization storage tank has a total volume from floor to maximum normal water level of about 0.2 Mgal. The tank has a concrete floor and steel wall. The outside of the steel wall is encircled by concrete up to a height of 3 feet and then by an un-insulated stud wall with plywood covering to the top. Washdown monitors were provided along with access platforms at the top of the tank; however, plant operators prefer to enter the tank through a manway and manually hose down the tank bottom during annual cleanings. The tank is mixed and aerated using diffusers on the floor of the tank, which are supplied from a blower in the Equipment Building.

There are several features associated with the existing plant influent piping, equalization basin, and headworks that are less than ideal, but have seemed to function adequately over the years:

1. The influent sewer flows by gravity into the bottom of the equalization storage tank. As the equalization tank fills, the influent sewer becomes surcharged and flows full, beginning as far upstream as Donner Pass Road near the Interstate 80 crossing. This full-pipe condition results in low flow velocities that allow solids to settle in the pipe. The pipe is 14 inches in diameter and, when the pipe is full, a flow of 1.4 Mgal/d is needed to attain a velocity of 2 ft/s that is normally desired to keep solids in suspension. However, the pipe is allowed to flush when the equalization tank is drained or bypassed. It is not known how much solids have accumulated in the pipe and failed to flush out, because there are no access manholes in this pipe, which is designed for pressurized flow conditions. Nevertheless, the system has functioned without known problems for over 20 years.
2. The plant headworks is located downstream of the equalization storage basin. Ideally, the headworks would be upstream so that the influent screens could remove paper, hair, and stringy materials that otherwise can accumulate on piping and equipment in the tank, resulting in added cleaning and maintenance requirements. However, this arrangement is apparently acceptable, since the existing tank is cleaned only about once per year, with no notable issues.
3. There is currently no adequate influent sampling location. The original design of the influent sewer line and equalization basin included features for sampling from the influent pressure pipeline. However, the sampling system reportedly did not function as desired and was abandoned in favor of taking samples of the equalization tank effluent. Although this accurately represents the wastewater that is being treated in the downstream biological treatment system, there is no way to characterize the actual influent sewage without the impact of filter backwash water returned to the equalization tank.

Because the existing system has functioned adequately for more than 20 years, it would probably be acceptable to continue operating with the issues identified above. Nevertheless, it is appropriate to consider alternatives that would eliminate these issues.

8.3.2 NEW FACILITIES ALTERNATIVES

As mentioned previously, an additional 0.55 Mgal of equalization storage capacity is desired to supplement the existing 0.2 Mgal tank. The existing and new tanks would be designated as Equalization Storage Tank 1 and 2 (EST 1 and 2), respectively. Two alternative concepts for providing the additional storage volume and required ancillary facilities are as follows:

Concept 1: Retain the existing equalization tank and headworks in the current configuration. Normally, EST 2 would be filled via an overflow from EST 1, but piping and valves would be provided so that it would be possible to use either tank by itself and take the other out of service. EST 2 would be configured and located such that its maximum water surface elevation was lower than the overflow in EST 1. Because of its volume and because of site constraints, it is likely that the best configuration for EST 2 would involve a tank that is deeper than EST 1, with a lower bottom elevation than EST 1. An equalization return pump station would be needed to pump the contents of at least the lower portion of EST 2 back to EST 1 or directly to the existing headworks for subsequent treatment. This alternative would maintain the status quo with regard to influent sewer surcharging, no screening ahead of equalization, and lack of a good raw sewage sample location.

Concept 2: Bring the influent sewage in through the existing headworks first for flow measurement, sampling and screening and then directly into EST 2. Under this alternative, EST 2 would be located low on the plant site to facilitate gravity filling from the existing headworks. An equalization return pump station at EST 2 would be used to pump the screened sewage to the biological treatment system, but also to fill EST 1. EST 1 would drain by gravity back to EST 2 through a new motorized control valve. To facilitate taking EST 2 out of service, a small tank would be located near EST 2 to act as an alternative sump for the return pump station. This alternative would eliminate influent sewer surcharging, would provide a good location for raw sewage sampling, and would provide screening ahead of flow equalization. The drawbacks of this alternative, as compared to the previous alternative, are more plant piping and the additional power cost for pumping more flow at a higher head, instead of taking advantage of the head available by surcharging the sewer.

In a later section of this report, various biological treatment alternatives are considered, and one of these is a membrane bioreactor (MBR). If an MBR were to be implemented at DSPUD, a new headworks with finer screens would be required. This is because MBR systems require screens with openings of 1 to 3 mm, depending on the specific MBR system chosen, compared to 6 mm openings for the existing influent screen. Therefore, for the MBR treatment option, Alternatives 1 and 2 indicated above would have to be modified to Alternatives 1-MBR and 2-MBR as indicated below:

1-MBR. This alternative would be the same as Alternative 1, except that the existing headworks would be abandoned and a new headworks with fine screens would be installed between the EST1 outlet and the biological treatment system. The return pumps at EST2 would pump to EST1 or to the new headworks.

2-MBR. This alternative would be the same as Alternative 2, except a new headworks would be located between the existing headworks and EST2. With both the existing headworks screen and the new screens in service, two-stage screening would be provided. Although not essential, it is beneficial to have coarse screening ahead of fine screening to limit the load on the fine screens.

For Alternatives 1 and 1-MBR, a good location for the new 0.55 Mgal EST2 would be in the area behind the existing District Office and Fire Department building. For Alternatives 2 and 2-MBR, EST2 would be lower on the plant site, probably near the existing 600,000 gallon sludge storage tank or sludge drying beds. In this case, a new tank could be constructed or the existing sludge storage tank could be converted for use as EST2. Conversion of the existing sludge storage tank would be possible only if a new smaller digester and mechanical sludge dewatering facilities are included in the project. This topic is addressed further in Chapter 15.

8.3.3 PROPOSED EQUALIZATION FEATURES

The existing equalization basin includes an aeration system that is used for both mixing and to provide oxygen to keep the wastewater from going septic. The air flow requirements for mixing far exceed the requirements for oxygen, resulting in elevated dissolved oxygen concentrations in the tank. These high dissolved oxygen concentrations are detrimental to the downstream biological process for nitrogen removal. Therefore, it is proposed that EST 1 be refitted and that EST 2 be provided with features for mixing without over-aerating the tanks. If the existing sludge storage tank is converted for use as EST2, it would be refitted with a new mixing and aeration system, also. A jet aeration system is suggested. In the jet aeration system, tank contents would be recirculated through a manifold with discharge nozzles to mix the tank. Air would be introduced only as needed and mixed with the water exiting the nozzles. The amount of air added could be automatically controlled based on the oxidation reduction potential (ORP) in the tank. Just enough air would be added, when needed, to prevent septic conditions and odors.

Regardless of which equalization alternative is chosen, the controls for releasing or pumping equalization tank contents to the downstream biological treatment process would be programmed into the plant supervisory control and data acquisition (SCADA) system. Although the desired setpoint for flows to the biological treatment system would be input regularly by the plant operators, the control system would monitor levels in the equalization basins and dynamically adjust the setpoint flow if needed based on undesirable high or low levels, as previously discussed. As needed, high and low level alarms and alarms regarding equipment malfunctions would be triggered to notify the operators of potential problems. Both equalization storage tanks would be provided with overflow outlets to either the downstream biological treatment system (as allowed, depending on the biological treatment alternative selected) or to the emergency storage tank.

Section 9 of this report includes an investigation of heat transfers between the wastewater and the environment and the resultant impact on wastewater temperatures, which is a critical concern for biological treatment. As developed in that analysis, covering the tanks in the wastewater treatment plant is beneficial for conserving heat. However, the cost of covers cannot be justified by the reduced heat loss, as compared to providing supplemental heat when needed. Therefore it is not proposed to retrofit a cover to EST1 or to provide a cover on EST2, if a new basin is built.

However, in the case of a new tank being built for EST2, the incremental cost of incorporating a cover in the original construction of that tank should be verified during preliminary design.

8.3.4 EQUALIZATION AND HEADWORKS ALTERNATIVE COST ANALYSIS

Cost estimates for six different equalization alternatives are shown in Table 8-2. These include Alternatives are 1, 2, 1-MBR, and 2-MBR as previously described. For Alternatives 2 and 2-MBR sub-alternatives are shown, depending on whether a new tank is built for EST2 (designated by “N” or the existing 600,000 gallon sludge storage tank is converted for this use (designated by “E”).

In Section 15, an analysis of solids handling alternatives is presented. In that analysis, which includes evaluation of the costs in Table 8-2, it is concluded that converting the existing 600,000 gallon sludge storage tank for equalization storage use would not be cost-effective. Therefore, Alternatives 2-E and 2-MBR-E are eliminated from further consideration.

After elimination of the alternatives indicated above, four alternatives remain, representing Equalization Concepts “1” and “2” under the MBR and non-MBR biological treatment alternatives. As previously discussed, the benefits of Concept 2 are elimination of influent sewer surcharging, screening ahead of equalization storage, and provision of a convenient location to monitor the influent raw sewage without the impact of plant recycle streams. Based on the costs given in Table 8-2, these benefits would come at a cost of approximately $500,000 (total present worth). Since the existing facilities have worked adequately for over 20 years, it appears that there is not a compelling reason to incur this cost. Therefore, only Concept 1 is considered further.

Of course, selection between Alternatives 1 and 1-MBR depends on the analysis of the biological treatment alternatives and other plant components that would be impacted by them. This topic is discussed in Section 17.

Table 8-2
Equalization and Headworks Alternative Costs

	Cost for Indicated Alternative (a), $					
Alternative Designation:	**1**	**1-MBR**	**2-N**	**2-E**	**2-MBR-N**	**2-MBR-E**
Biological Treatment Alt.:	**Not MBR**	**MBR**	**Not MBR**	**Not MBR**	**MBR**	**MBR**
EST2 New or Existing:	**New**	**New**	**New**	**Exist**	**New**	**Exist**
Capital Cost						
New Mixing and Aeration in EST1	180,000	180,000	180,000	180,000	180,000	180,000
Construct New EST2	300,000	300,000	300,000		300,000	
Mixing and Aeration in EST2	250,000	250,000	250,000	275,000	250,000	275,000
Blower and Elec Bldg at EST2	120,000	120,000	120,000	120,000	120,000	120,000
Pump Station at EST2	100,000	100,000	200,000	200,000	200,000	200,000
Back-Up Pump Sump at EST2			25,000	25,000	25,000	25,000
New Headworks / Fine Screens		600,000			600,000	600,000
Sitework	50,000	80,000	50,000	40,000	80,000	70,000
Equalization Site Piping	50,000	60,000	120,000	120,000	130,000	130,000
Electrical and Instrumentation	200,000	380,000	230,000	230,000	410,000	410,000
Subtotal 1	1,250,000	2,070,000	1,475,000	1,190,000	2,295,000	2,010,000
General Conditions, OH&P, 20%	250,000	410,000	300,000	240,000	460,000	400,000
Subtotal 2	1,500,000	2,480,000	1,775,000	1,430,000	2,755,000	2,410,000
Contingency, 20%	300,000	500,000	360,000	290,000	550,000	480,000
Total Construction Cost	1,800,000	2,980,000	2,135,000	1,720,000	3,305,000	2,890,000
Engineering, Admin., Environ. 25%	450,000	750,000	530,000	430,000	830,000	720,000
Total Capital Cost	2,250,000	3,730,000	2,665,000	2,150,000	4,135,000	3,610,000
Annual Cost						
Labor	25,000	26,000	27,000	27,000	28,000	28,000
Power	22,000	22,000	24,000	24,000	24,000	24,000
Total	47,000	48,000	51,000	51,000	52,000	52,000
Present Worth Cost						
Present Worth of Annual Costs (b)	699,000	714,000	759,000	759,000	774,000	774,000
Total Present Worth	2,949,000	4,444,000	3,424,000	2,909,000	4,909,000	4,384,000

(a) First quarter 2010 cost level, ENR 20-Cities CCI = 8700.

(b) 20 years at inflation-adjusted discount rate of 3 percent, Present Worth Factor = 14.88.

Section 9

Biological Treatment

Section 9

Biological Treatment

The main method for removing pollutants from wastewater is biological treatment in which various types of microorganisms are grown and used to accomplish the desired treatment objectives. The microorganisms are grown in the wastewater where they consume or utilize the pollutants for their growth and/or respiration. Once the desired level of treatment is accomplished, the microorganisms are removed from the wastewater by physical means such as settling or filtering, resulting in treated effluent. For treatment to be successful, an adequate mass of microorganisms must be developed and held in the plant, corresponding to the volume of wastewater and mass of pollutants to be treated. Careful attention must be paid in plant design and operation to provide adequate tank volumes or growth surfaces as well as suitable environmental conditions for the microorganisms, including such things as temperature, pH, and removal or prevention of toxic or inhibitory compounds.

In this section, various means for accomplishing biological treatment, specifically tailored to the conditions at the DSPUD wastewater treatment plant are investigated. The alternatives considered herein are those selected for detailed analysis by DSPUD after completion and review of a preliminary study in which a larger array of alternatives was investigated and screened, as discussed in Section 7. For all alternatives considered in this section, heat and wastewater temperature management are key issues. Therefore, this topic is considered first below, followed by analysis of the various treatment options. Finally, the chemical storage and feed systems needed to support biological treatment are considered.

9.1 HEAT TRANSFER AND TEMPERATURE MANAGEMENT

The microorganisms that accomplish wastewater treatment are very sensitive to temperature. As a typical rule of thumb, the growth rate of microorganisms doubles with a 10 °C increase in temperature. Therefore, although there are factors other than growth rate involved, reactor basins in a plant operating at 10 °C would need to be much larger, perhaps double or more, those for a plant operating at 20 °C.

The microorganisms that accomplish the removal of ammonia are particularly slow growing and sensitive to temperature. Like all microorganisms in wastewater treatment processes, the ammonia oxidizing bacteria (AOB) undergo both growth and decay, and it is the net growth rate (growth minus decay) that is important. Of particular concern is that, with decreasing temperatures, the growth rate is slowed much more than the decay rate, so the net growth rate is impacted even more than the growth rate. Furthermore, when denitrification must be accomplished (as required at DSPUD) in a suspended growth process, the AOB, which can only grow under aerobic conditions, must spend part of their time in reactor areas without oxygen where they continue to decay but do not grow. In this case, the impacts of low temperature are further exacerbated. As temperatures get further and further below 10 °C, ammonia removal becomes more and more difficult and less reliable. Although it is possible to have nitrification even at temperatures as low as 5 °C, it is undesirable to operate at such low temperatures, particularly when denitrification is also required. For this project, a minimum sustained temperature of 7 °C was selected as an objective.

Shown in Figure 9-1 are actual recorded temperatures of the existing plant effluent from January 2002 through May 2008. As indicated in the figure, effluent temperatures can be at or below 5 °C for about five months of the year (December through early May). Temperatures in the reactor basins have been shown to be about the same as the plant effluent temperatures. Plant influent temperatures are not monitored, however, the surface water supply temperature for DSPUD is frequently as low as 4 °C in the winter. SLCWD also has a relatively cold surface water supply. Because of the historical low process temperatures and the desire to maintain higher temperatures, an analysis of heat transfers from the wastewater to the environment during treatment was completed as discussed below. In the analysis the potential benefits of adding covers on the process basins are evaluated. Subsequently, the option of adding a boiler and heat exchanger system to increase the wastewater temperature is considered.

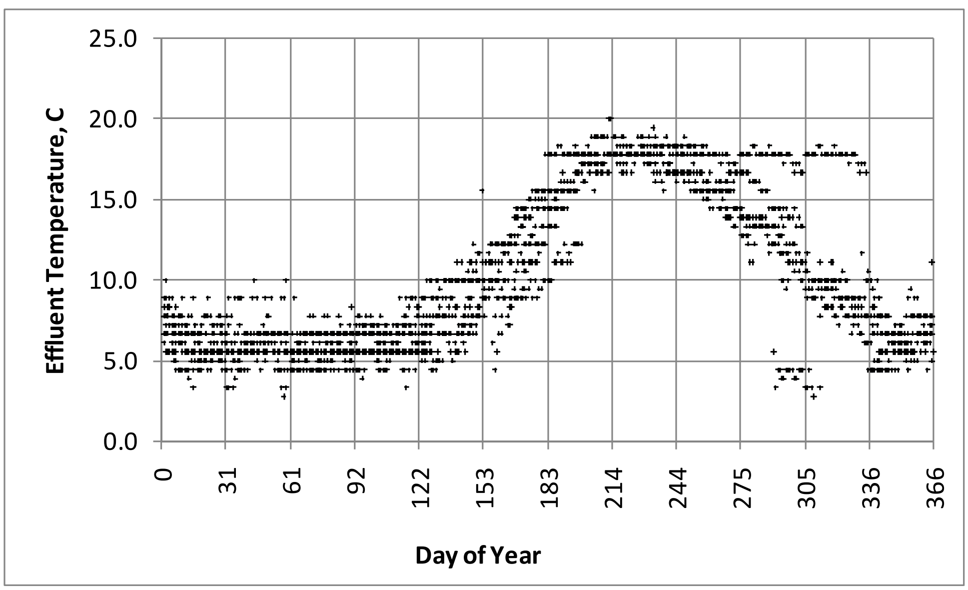


Figure 9-1
Effluent Temperatures Jan. 2002 - May 2008

9.1.1 HEAT TRANSFER AND TEMPERATURES WITH AND WITHOUT BASIN COVERS

An investigation was completed to estimate the magnitude of heat losses and gains by various mechanisms throughout the wastewater treatment plant with and without covers on the treatment basins. Calculations were completed generally in accordance with Talati 1990, supplemented by various other references as listed at the end of this subsection. Additionally, adjustments were made for the low air density at elevation 6,600 ft.

The following heat losses and heat gains were considered in the calculations:

Heat Losses: Atmospheric Radiation
Surface Evaporation
Surface Convection
Subsurface Aeration
Tank Wall and Floor Convection/Conduction

Heat Gains: Solar Radiation
Power Inputs by Aeration, Mixing, and Pumping
Heat Produced by Biological Reactions

Although heat transfers associated with snow or rainfall falling into the basins at the wastewater treatment were not included in the detailed calculations, adjustments for these effects are discussed later in this section.

In this investigation, heat transfers from the existing and proposed new equalization storage basins and the existing reactor and clarifier basins (Plants 1 and 2) were calculated. Therefore, the calculations would approximately represent any treatment option that makes use of the existing reactor and clarifier basins, such as the MBR and IFAS options considered later in this Section 9. The power inputs related to aeration, mixing and pumping were generally set up based on the IFAS option, but differences in these factors for the MBR option would not produce significantly different results and conclusions. For the IFAS option with added external clarifiers, the heat transfers from the proposed external clarifiers were not included in the calculations. Although these clarifiers would be downstream of the reactor basins, cooling in the clarifiers would somewhat impact temperatures in the reactor basins due to recirculation of return activated sludge from the clarifiers to the reactor basins. Thus, including the external clarifiers would provide additional opportunity for further cooling the biological reactor basins. For the submerged attached growth option considered later in this section, the heat losses and gains occurring in the equalization basins would be the same as calculated in this investigation, but the heat transfers in the associated primary clarifiers and biological reactor vessels would be different. Nevertheless, it is believed that conclusions from these calculations can be applied to the submerged attached growth alternative. It is suggested that heat balance calculations similar to those considered herein should be refined in the preliminary design phase for the final project to be implemented.

Wastewater heat loss to the environment is dependent on ambient air temperature. Based on 50 years of data from Snow Lab, the average of monthly average temperatures for the months of December through March were -1.5, -2.1, -1.6, and -0.1 °C, respectively. The minimum monthly average temperatures recorded for those same months were -7.5, -6.0, -5.3, and -4.0 °C, respectively. For the temperature modeling accomplished for this study, temperatures of 0 and -5 °C were considered.

Wastewater heat loss to the environment in uncovered process basins is highly dependent on wind speed. There is no wind speed monitoring at the DSPUD wastewater treatment plant, so it is somewhat uncertain what typical and high sustained wind speeds might be at the plant.

However, wind speeds are monitored at Blue Canyon and at the Central Sierra Snow Lab in Norden. Selected winter wind speed data from these locations are shown in Figure 9-2. Although the Norden location is closer to the DSPUD plant than Blue Canyon, Snow Lab personnel indicated that wind speeds at the DSPUD plant might tend to be higher than at the Snow Lab because of the funneling effect of adjacent Interstate 80. However, wind speeds at a particular location are dependent on wind protection afforded by the specific topography, buildings and trees in the immediate vicinity. Therefore, the existing influent equalization basin, which is on relatively high ground and without any significant wind screening features, might experience more wind than Plants 1 and 2, which are lower and more protected. On the other hand, when the water level is substantially below the top of the tank, the actual water surface in the equalization basin would be afforded more protection from the wind. For this study, it is estimated that long-term average winter wind speeds for the months of December through March at the DSPUD wastewater treatment plant might be in the range of about 3 to 4 mph (slightly higher than the 2.5 to 2.7 mph range at Snow Lab). It is suggested that a reasonable design worst-case wind speed is one that might be sustained as an average for several days or a week. Since the maximum weekly and maximum daily average wind speeds at Snow Lab for the months in question (based on data for the years of 2002 through 2009) were typically around 4 to 6 mph, a reasonable design worst-case wind speed for the DSPUD wastewater treatment plant might be around 5 to 7 mph. Of course, winds occurring for shorter durations, such as several hours, could be much higher, but these short-term events would not have a significant impact on wastewater process temperatures.

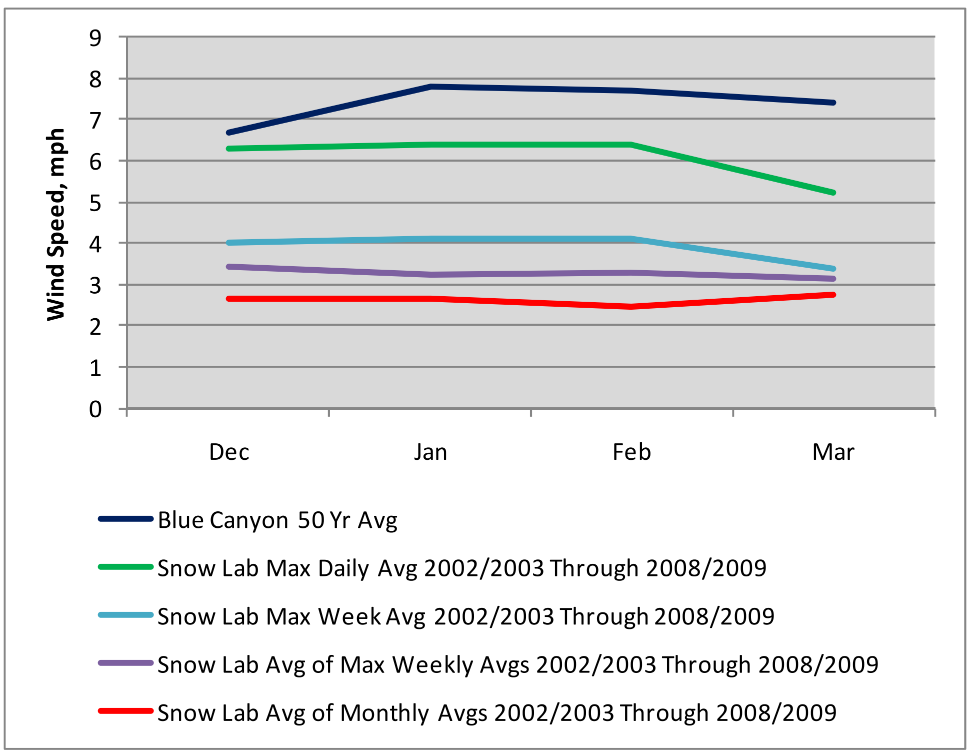


Figure 9-2
Wind Speeds During December Through March

Based on the above discussion and other relevant factors, the following input data variations were considered in the temperature modeling completed for this study:

Wastewater Flow: 0.1, 0.35 and 0.74 Mgal/d
Wastewater COD: 200 and 500 mg/L
Influent Temperature: 5, 8, and 10 °C
Air Temperature: 0 and -5 °C
Wind Speed: 0, 3, 6, and 9 miles per hour

For simplicity, all calculations were done based on solar radiation occurring on January 1, which would fairly accurately represent the key peak flow and load conditions occurring around that time of the year. Additionally, the calculations were based on an assumed typical cloud cover of 50 percent.

As noted previously, the main focus of the investigation was to determine the approximate impact that could be expected by covering the basins versus leaving them open to the atmosphere. Based on the above variations in input parameters, a total of 144 scenarios were modeled with and without basin covers (288 scenarios total).

By adding covers to the basins, it was estimated that the following heat transfers would be essentially eliminated: solar radiation, atmospheric radiation, surface convection, and evaporative heat loss (snow or rain falling into the basins would also be eliminated, as discussed later in this section). It is realized that these calculations are not exact, but give a reasonable estimate of the heat loss impacts. For example, although direct solar radiation on the wastewater in the basins would be eliminated by the covers, limited heating of the wastewater due to solar radiation would continue to occur due to heat transfers through the covers and the air below the covers. Atmospheric radiation from the wastewater should be effectively eliminated by the covers. Surface convection is proportional to the temperature difference between the water and the air and is also proportional to wind speed. With covers, and in the absence of major ventilation, the air above the water would tend to be the same temperature as the water and there would be no wind, eliminating surface convection as a heat transfer mechanism. Evaporative heat loss is impacted by the temperature difference between the water and the air and the relative humidity of the air and is directly proportional to wind speed. With covers, it was presumed that the air and water would be nearly the same temperature, the air would become saturated with water vapor, and there would be no wind, effectively eliminating evaporative heat loss. However, to the extent that ventilation air might be passed under the cover, there could still be some evaporative heat loss.

Input data and results for several selected scenarios are shown graphically and tabulated in Figures 9-3 through 9-7 and are discussed briefly below. In each figure, results are shown with and without basin covers.

The scenario represented in Figure 9-3 is that of low wastewater flows and low strength such as might occur in the late fall. The influent wastewater temperature was assumed to be cold at 5 °C, the ambient air temperature was 0 °C and no wind was assumed. In this case, covering the basins is estimated to result in an increase in the biological reactor basin temperature from 5.6 to 6.7 °C. This relatively low impact of about 1 °C is mainly due to the assumption of no wind. This is clearly evident from Figure 9-4, which represents an identical scenario, except for a wind speed of 6 mph. In this case, covering the basins would result in a much larger benefit, calculated as increasing the temperature from 2.5 to 6.7 °C.

The scenario represented in Figure 9-5 is that of high wastewater flow and high strength, such as might occur during the week between the Christmas and New Year holiday. The influent temperature was assumed to be 8 °C, the ambient air temperature was 0 °C and there was no wind. In this case, the impact of covering the basins would be relatively insignificant, raising the reactor temperature only from 8.7 to 9.0 °C. This very low impact, again, was mainly due to the complete lack of wind, but also due to the high wastewater flow. When the wastewater flow is high, the wastewater spends less time in the basins, which minimizes the amount of heat that can be transferred per unit of wastewater flow. In Figure 9-6, the exact same scenario is modeled, except that a wind speed of 6 mph was assumed. In this case, the covers result in a more significant increase from 7.8 to 9.0 °C.

Shown in Figure 9-7 is a scenario that might be close to a future design typical winter ski season condition (average weekday and weekend, not during a holiday period), except that the COD at 500 mg/L is somewhat higher than typical (typical is probably more like 350 to 400 mg/L). A mild wind was assumed at 3 mph. In this case, the covers resulted in a reactor basin temperature increase from 7.6 to 9.1 °C. With a 4 mph wind, the corresponding temperatures would be 7.3 to 9.1 °C. With a COD of 350 mg/L, all of these temperatures would be shifted down by about 0.2 °C, but the differences in temperatures with and without covers would remain the same.

From the graphs shown in Figures 9-3 through 9-7, it is interesting to note the relative magnitudes of the various heat transfer mechanisms. For example, with low flows and loads, the most significant heat gains without covers are due to solar radiation and power inputs. These heat gains remain important with high strength wastewater, but in the case of high strength and high flow, the heat produced from biological reactions becomes most important. The most important heat loss when there is no wind is due to atmospheric radiation. However, with wind, evaporative heat losses become very important and, if the wastewater is substantially warmer than the air, surface convection becomes very important.

In all cases modeled, heat losses through tank walls and floors were relatively insignificant because it was assumed that the tank walls were insulated, which is currently the case for Plants 1 and 2. Although the current equalization basin is surrounded by a stud wall with wood siding, it is believed that there is no insulation between the studs. It should be relatively easy to blow in insulation, and this was assumed. It was assumed that the proposed new equalization tank would have insulated walls also.

Input Parameters:

Wastewater Flow, Mgal/d	0.1
Influent COD, mg/L	200
Influent Temperature, C	5
Ambient Air Temperature, C	0
Wind Speed, mph	0

Calculated Effluent Temp., C:

Without Covers	5.6
With Covers	6.7

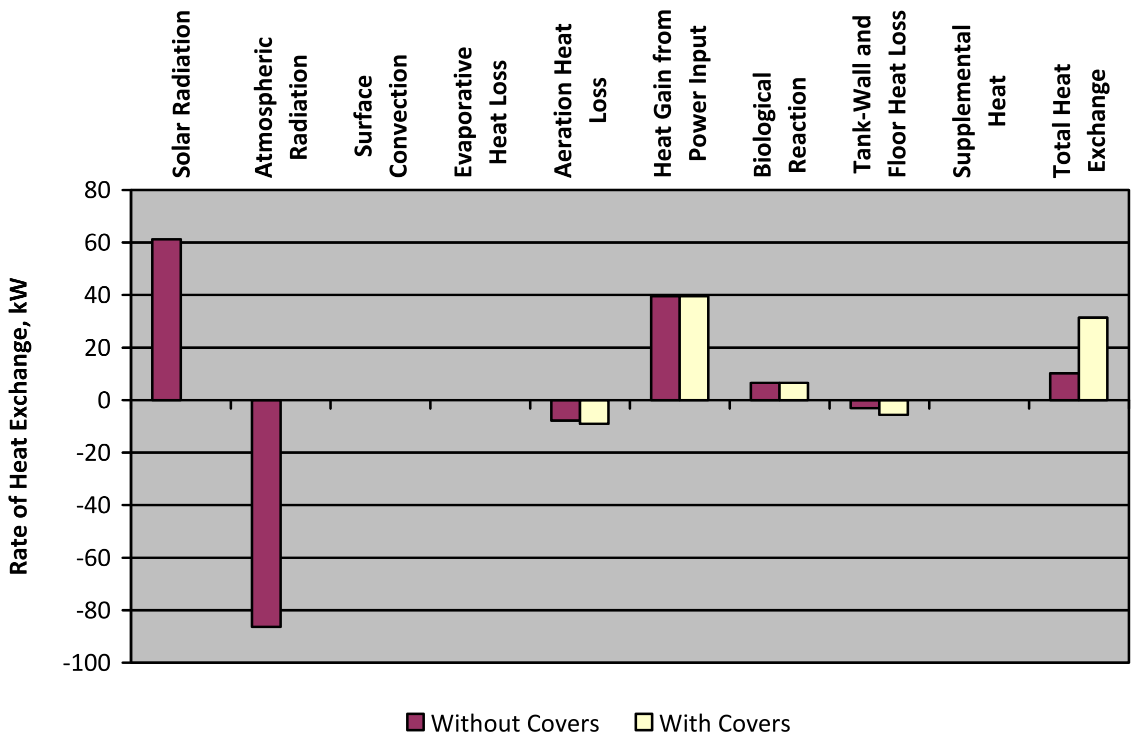


Figure 9-3
Heat Transfer Scenario with Low Flow, Low COD, No Wind

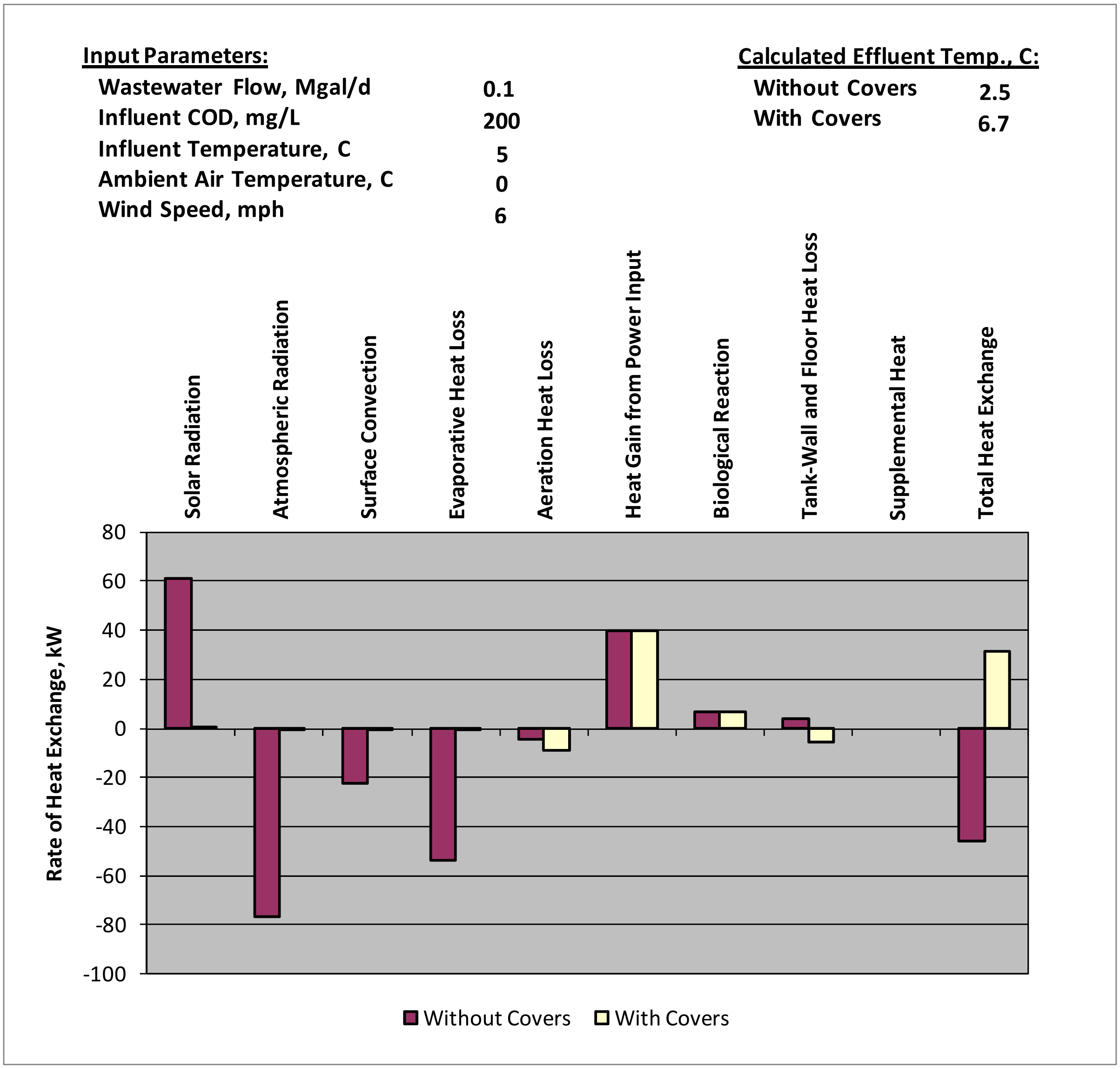


Figure 9-4
Heat Transfer Scenario with Low Flow, Low COD, 6 MPH Wind

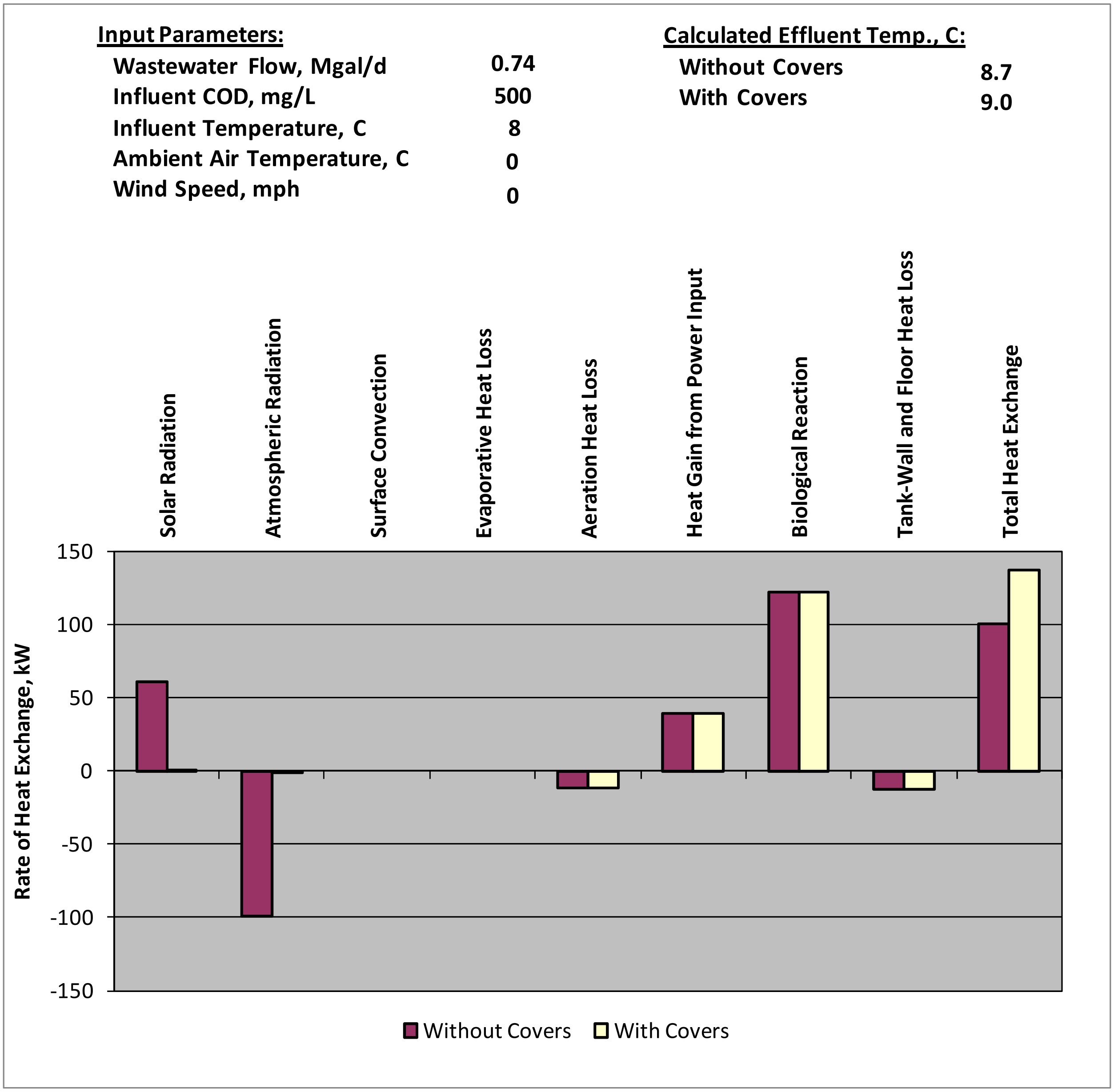


Figure 9-5

Heat Transfer Scenario with High Flow, High COD, No Wind

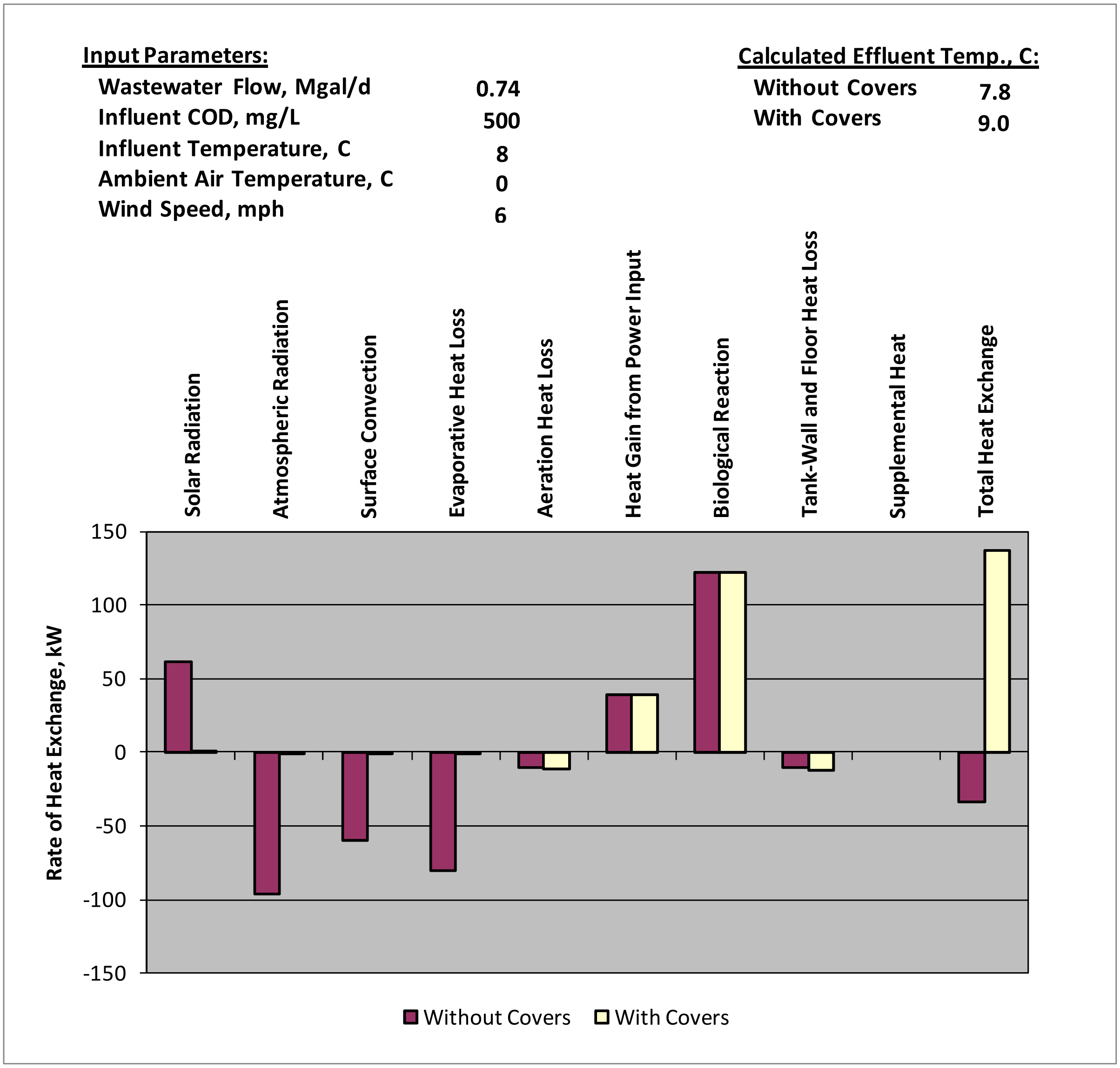


Figure 9-6
Heat Transfer Scenario with High Flow, High COD, 6 MPH Wind

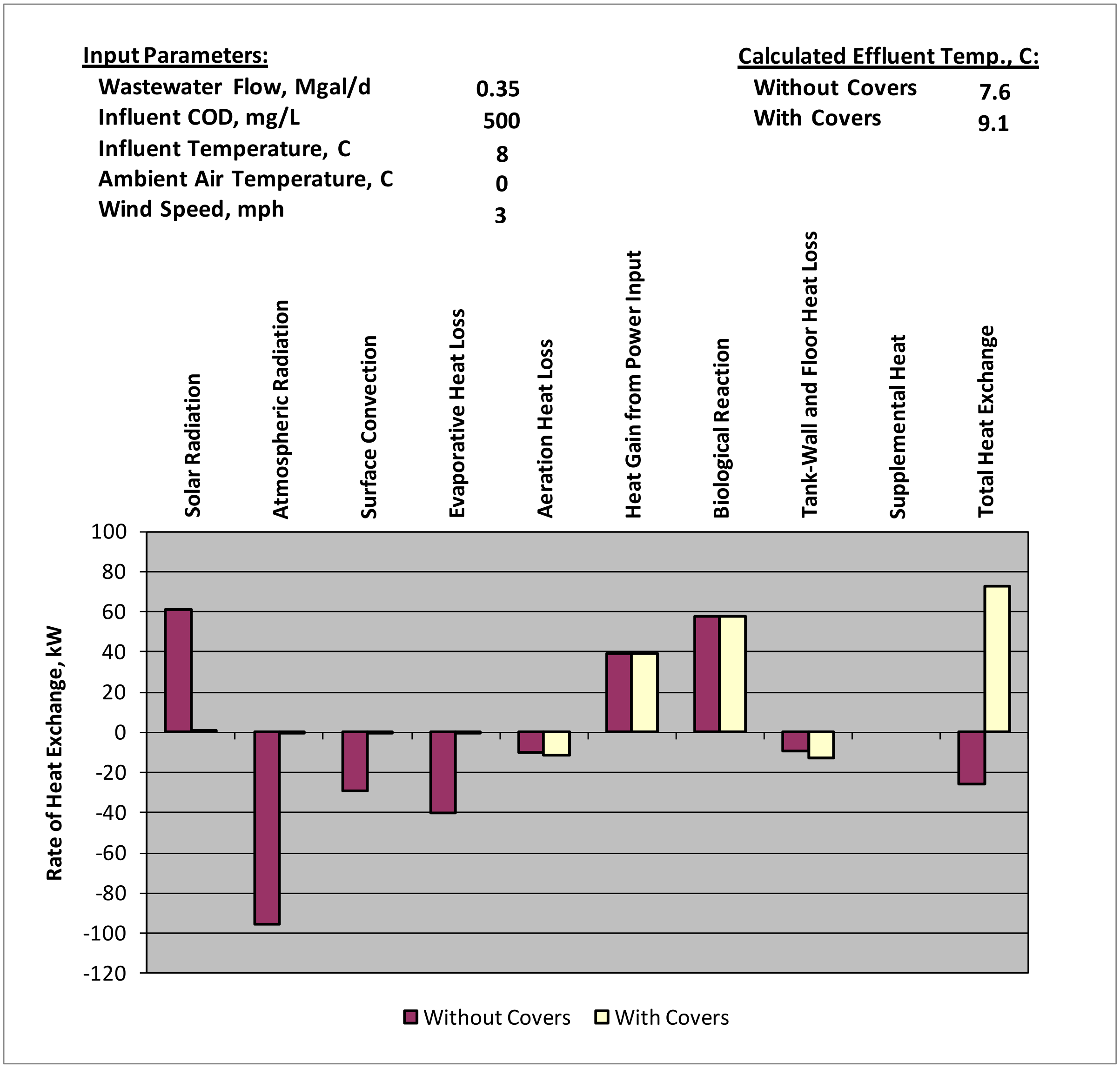


Figure 9-7
Heat Transfer Scenario with Typical Flow, High COD, 5 MPH Wind

Reactor basin temperatures with and without covers for all 144 scenarios considered are shown in Figure 9-8. The following general conclusions regarding the conditions modeled can be drawn from these results:

1. With basin covers, reactor basin temperatures would be expected to be about 1 to 2 °C higher than the influent temperatures during low flow conditions and 0.5 to 1 °C higher during high flow conditions.
2. Comparing reactor temperatures with and without basin covers (covers on equalization and reactor basins), it can be seen that the benefit of providing covers is highly dependent on the wind speed, especially during low flow conditions. At the flow of 0.1 Mgal/d, and without any wind, the benefit of providing covers is typically about 1 to 2 °C. However, with a 6 mph wind, the benefit of providing covers at the same low flow is 5 to 8 °C. At the high flow of 0.74 Mgal/d and with no wind, the benefit of providing covers is only about 0 to 0.5° C. With a 6 mph wind, the benefit is increased to 1 to 2 °C.
3. At a typical winter wind speed of 3 to 4 mph, the benefit of covers is about 4 to 6 °C at a flow of 0.1 Mgal/d, 1.5 to 3 °C at 0.35 Mgal/d, and 1 to 2 °C at 0.74 Mgal/d.

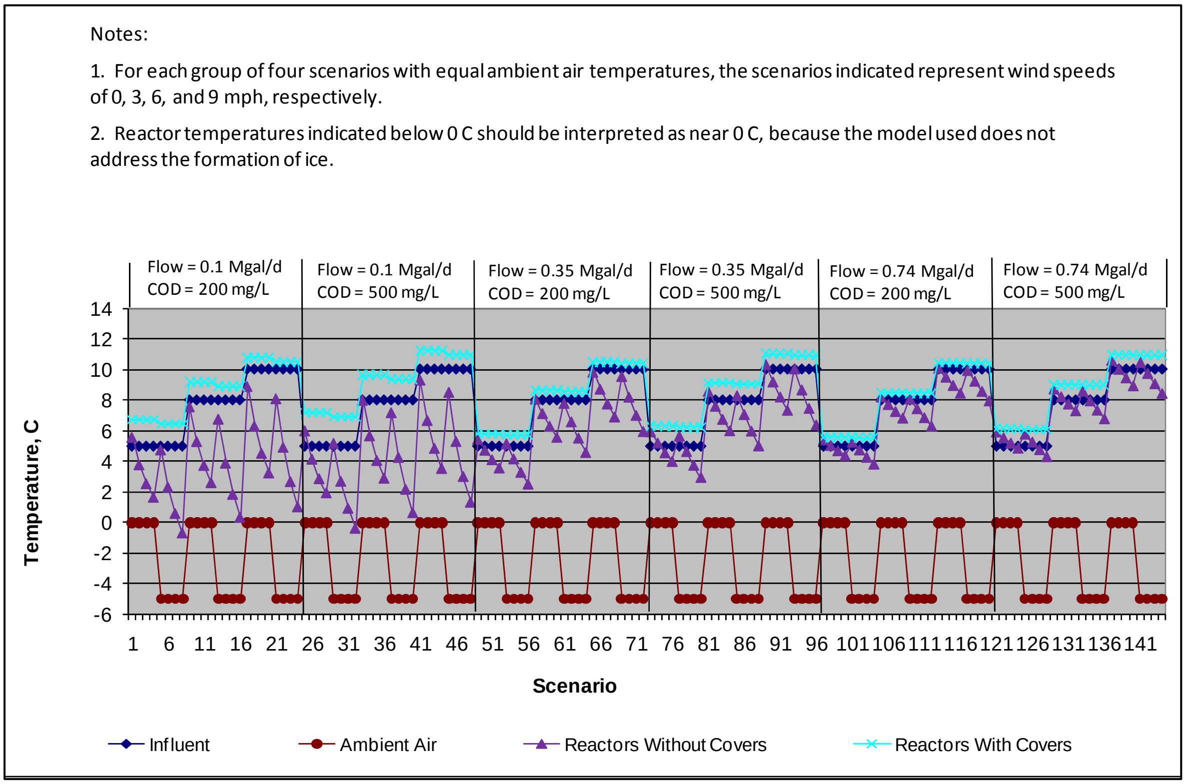


Figure 9-8
Input Data and Reactor Basin Temperature Results for 144 Scenarios

Based on the information presented above, estimated typical benefits of covers at various times of the year are indicated in Table 9-1. When elimination of precipitation falling into the basins is considered, the actual benefit of covers could be substantially greater. For example, with a sustained precipitation rate of 1 inch per day (measured as liquid water) falling as snow on the basins, and assuming the temperature of the snow to be 25 °F (-3.9 °C), the heat required to melt the snow and raise the temperature of the melted snow to 7 °C would be enough to lower the temperature of the wastewater flowing through the plant by about 2.8, 1.6, and 0.7 °C at wastewater flow rates of 0.20, 0.35 and 0.74 Mgal/d, respectively. These differential temperatures would be additive to the benefits of covers shown in Table 9-1. Of course, a sustained precipitation rate of 1-inch per day, lasting a week or longer would be relatively rare; therefore, the normal temperature benefit of eliminating precipitation on the basins would be less. At the average precipitation rate from December through March (about 0.29 inches per day), the differential temperatures due to eliminating precipitation on the basins would be less than 1/3 those indicated at 1 inch per day (again assuming snow at 25 °F).

Because actual influent temperature data are not available, it is not possible to state whether the benefits indicated in Table 9-1, supplemented with the benefits of eliminating snowfall on the basins, would be adequate to assure reactor basin temperatures above 7 °C, but there is a very good chance that would be the case most of the time.

It is considered likely that the raw influent temperature (before equalization storage) would be greater than 6 °C during the fall and winter and, if so, reactor temperatures should be above 7 °C with basin covers. Even though the surface water supply temperature is as low as 4 °C in the winter, the wastewater would be expected to be somewhat warmer, since some of the water is heated for use and warm waste products are added during use. One exception might be the condition of rain on snow, producing very cold and high infiltration and inflows. In this case, however, skiing conditions would be poor and the strength of the wastewater entering the wastewater treatment plant would be reduced, lessening the possibility of an ammonia breakthrough, despite the lower temperatures. For the same reason, lower influent temperatures that could occur during the spring snowmelt period, which is characterized by very low occupancy in the Donner Summit area, are probably not a significant concern. Also, in the spring, ambient temperatures would be higher and daylight hours would be longer, so there would be less heat loss and more heat gain in the wastewater treatment plant basins.

Possible capital costs for providing covers on the equalization storage tanks and on existing Plants 1 and 2 are shown in Table 9-2. These costs are based on very preliminary information from suppliers of tanks and geodesic dome covers. More detailed engineering analyses that are beyond the scope of this Facilities Plan would be required to verify actual costs. Such analyses, if desired by DSPUD, should be accomplished during preliminary design of plant improvements.

Table 9-1
Estimated Typical Benefits of Basin Covers Under Various Conditions

Time of Year and Conditions	**Temperature Increase in Reactor Basins Due to Basin Covers = Reactor Temp. w/Covers – Reactor Temp. w/o Covers** (a, b)		**Estimated Reactor Temp. Compared to Influent Temp. With Basin Covers = Reactor Temp. w/Covers – Influent Temp.** (a, b)	
November and early December, when flows would be generally low (0.1 to 0.2 Mgal/d) and it is desired to buildup the nitrifier population.	3 °C	3 °C	+1.5 °C	+1.5 °C
Late December through mid-March with typical flows (average of weekdays and weekends, about 0.35 Mgal/d).	2 °C	2 °C	+1 °C	+1 °C
Winter peak periods around holidays (flows up to 0.74 Mgal/d as a weekly average) and peak spring snowmelt periods.	1 °C	1 °C	+0.5 °C	+0.5 °C

(a) Covers on equalization and reactor basins. Average wind speed of 3 to 4 miles per hour in all cases.

(b) The benefits of covers would be higher than indicated in this table when elimination of direct precipitation impacts is considered (see text).

Table 9-2
Possible Costs of Providing Tank Covers on Equalization Basins and Plants 1 and 2[a,b]

Item	Cost, $
Retrofit Cover on Existing Equalization Basin, 1520 ft^2 @ \$100 ft^2	152,000
Retrofit Accessible Cover on Plant 1, 9852 ft^2 @ \$125/ft^2	1,231,000
Retrofit Accessible Cover on Plant 2, 7543 ft^2 @ \$125/ft^2	943,000
Incremental Cost of Cover on New Equalization Basin 2, 3120 ft^2 @ \$75/ft^2	234,000
Ventilation and Electrical Modifications Required with Covers, Plants 1 and 2	100,000
Subtotal 1	2,660,000
Contingencies, 20%	530,000
Subtotal 2	3,190,000
General Conditions, Overhead and Profit, 20%	640,000
Total Construction Cost	3,830,000
Engineering, Administration & Environmental, 20%[c]	770,000
Total Capital Cost	4,600,000

[a] Costs are preliminary estimates, subject to verification after more detailed structural analyses, which are beyond the scope of this study.
[b] First quarter 2010 cost level, ENR 20-Cities CCI = 8700.
[c] Some design engineering already included in pre-engineered cover costs.

One of the cost components indicated in Table 9-2 is for ventilation and electrical improvements that would be required in connection with covers on Plants 1 and 2. Under the National Fire Protection Association Standard 820 (NFPA 820) for Fire Protection in Wastewater Treatment and Collection Facilities and the National Electric Code, the entire enclosed space under the covers of all tanks would be Classified as Class I, Group D, Division 1 areas, because of the fire and explosion hazards associated with possible flammable materials contained in sewage. This is not a significant issue for the equalization basins, because regular human access is not needed and there would be insignificant electrical and instrumentation equipment inside. For Plants 1 and 2, however, regular operator access is needed, and there would be more significant electrical and instrumentation equipment inside the covers. All exposed electrical and instrumentation features would have to be explosion proof or intrinsically safe. Ventilation and explosive gas monitoring would be required. The full extent of required improvements would have to be determined during design, but a rough allowance has been made in Table 9-2.

9.1.2 Consideration of Providing Supplemental Heat

As an alternative to covering the basins (and possibly as a supplement to covering the basins), DSPUD could use a boiler and heat exchanger system to provide supplemental heat to the reactor basins when needed. If such a system were to be used without basin covers, the maximum capacity of the system should perhaps be adequate to increase a reactor basin influent temperature of 5 °C to 7 °C at a flow rate of 0.74 Mgal/d and also to assure no reduction in that temperature as heat is lost in the reactor basin under critical design conditions. This is estimated to require a heat input of about 1.7 million Btu/hr (based on wind speed of 6 mph, ambient air temperature of -5 °C and precipitation of 7 inches per week, measured as equivalent water, but occurring as snow).

The boiler and heat exchanger would be used as needed to maintain a temperature of at least 7 °C at all times in the reactor basins. Based on analysis of plant effluent flow and temperature data from January 2002 through March 31, 2008, heating would have been required on 95 days per year on average. In the analysis it was assumed that no heating would be provided in the spring after March 31. During the days in which heating would have been required, the average flow was 0.26 Mgal/d and the flow weighted average effluent temperature was 5.8 °C (including the impacts of snow falling on the basins). Therefore, on those days, if heating were provided to raise the reactor basin temperature to 7 C, enough heat would have been added to raise the effluent temperature by 1.2 °C and also to offset the additional reactor basin heat losses at the higher temperature. The heat required to produce a temperature increase of 1.2 °C in a flow of 0.26 Mgal/d is 195,000 Btu/hr. Based on evaluations using the heat transfer model previously described, it is estimated that an additional 20 percent heat would have been needed to offset the additional heat losses at the higher temperature, resulting in a total heating requirement of 234,000 Btu/hr average over the 95 days. Therefore, the total average heating requirement would have been about 530 million Btu per year.

When flows increase in the future, average heating requirements should go down slightly, but not significantly. Even though there would be more flow to heat, the amount of heat required would be about the same or slightly lower because of three main factors: 1) at higher flows, the same rate of heat loss in the reactor basins would result in less temperature decrease in the higher flows, 2) more heat would be produced by biological reactions at the higher load, and 3) with higher flows, the temperature of the wastewater treatment plant influent would be expected to increase due to a higher fraction of warmer flows from residences and businesses as compared to colder infiltration and inflow. The net result is that heating should be needed on fewer days per year and the amount of heat required on those days should be slightly lower. Accordingly, a conservative estimate of future average heating requirements is 530 million Btu/yr.

Boilers and heat exchangers would be used to provide supplemental heat to the biological treatment process. Mixed liquor from the reactor basins, return activated sludge and/or reactor basin influent could be circulated through the heat exchangers. There are several possible configurations for such a system; the best configuration would be determined during detail design based on the final biological process selection and its configuration. For this Facilities Plan, it is presumed that two separate 1.0 million Btu/hr boiler and heat exchanger systems would be used. With two systems, half of the total capacity would still be available in the event of failure of one of the systems. The boilers, heat exchangers and associated recirculation pumps and ancillary facilities would be located in a new building, probably together with other process equipment, depending on the biological treatment alternative to be chosen.

The boilers can be fired with either propane gas or diesel. Diesel has more heating value than propane, is readily available, and less expensive. Therefore, for this study, it is assumed that diesel would be used. However, the choice between diesel and propane should be confirmed during preliminary design, including consideration of air quality permitting issues. If diesel is used, maximum weekly diesel consumption could be up to about 2,500 gallons (based on a maximum heating requirement of 1.7 million Btu/hr for seven days); therefore, two 2,000 gallon diesel fuel tanks would be recommended. Average annual diesel usage (in a typical year), based on the 530 million Btu requirement indicated above, would be about 5,000 gallons.

Estimated capital, annual, and present worth costs for the boiler and heat exchanger system are shown in Table 9-3.

Table 9-3
Boiler/Heat Exchanger System Costs[a]

Item	Cost, $
Capital Cost	
Two 1.0 Million BTU/hr Boilers and Heat Exchangers	500,000
Recirculation Pumps	24,000
Diesel Storage Tank and Fuel Supply System	80,000
Building Space: 600 ft^2 at $200/$ft^2$	120,000
Piping and Valves	15,000
Subtotal 1	739,000
Electrical and Instrumentation	185,000
Site Piping and Sitework	110,000
Subtotal 2	1,034,000
Contingency @ 20%	207,000
Subtotal 3	1,241,000
General Conditions, Overhead and Profit @ 20%	248,000
Total Construction Cost	1,489,000
Engineering, Administration & Environmental @ 25%	372,000
Total Capital Cost	1,861,000
Annual Costs	
Diesel Fuel	15,000
Power	1,000
Labor	3,000
Maintenance Materials	3,000
Total Annual Cost	22,000
Present Worth Costs	
Present Worth of Annual Costs[b]	327,000
Total Present Worth Cost	2,188,000

(a) First quarter 2010 cost level, ENR 20-Cities CCI = 8700.
(b) 20 years at inflation-adjusted discount rate of 3%, PWF = 14.88

9.1.3 RECOMMENDED HEAT TRANSFER AND TEMPERATURE MANAGEMENT IMPROVEMENTS

Based on the approximate analysis of costs for providing tank covers (Table 9-2), compared to the estimated costs of providing supplemental heat with boilers and heat exchangers (Table 9-3), it is apparent that the most cost-effective heat transfer and temperature management scheme is probably to provide the boilers and heat exchangers. If desired by DSPUD, this topic could be investigated in more detail during preliminary design.

9.1.4 References Used for Heat Transfer and Temperature Calculations

The following references were used in preparing the heat transfer and temperature calculations discussed above:

Gillot, S. and Vanrolleghem, P. (2003). Equilibrium temperature in aerated basins – comparison of two prediction models. *Wat. Res.*, 37, pp 3742-3748

Novotny, V. and Krenkel P. (1974). Evaporation and heat balance in aerated basins. *AIChE*. Vol 70 No. 136, pp 150-159

Sedory, P. E. and Stenstrom, M. K. (1995). Dynamic prediction of wastewater aeration basin temperature. *J. Env. Eng., ASCE, Vol. 121, pp 619-618*

Talati, S. N. and Stenstrom, M. K. (1990). Aeration-basin heat loss. *J. Env. Eng., ASCE, Vol. 116, pp 70-86*

9.2 BIOLOGICAL TREATMENT ALTERNATIVE ANALYSIS

After preparation of the "Donner Summit Public Utility District, Preliminary Investigation of Wastewater Management Options", dated June 10, 2009 (the document is included herewith as Appendix C, DSPUD authorized the investigation of five biological treatment alternatives as part of this Facilities Plan. The alternatives are as follows:

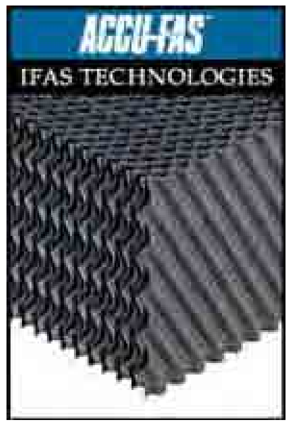
1. Upgrade/Expand the Existing AccuWeb System Using a Two-Stage Reactor Configuration
2. Upgrade/Expand the Existing AccuWeb System Using a Four-Stage Reactor Configuration
3. Membrane Bioreactors
4. New Integrated Fixed Film Activated Sludge
5. Submerged Attached Growth

The specific requirements and layouts associated with these alternatives are considered in the following subsections. A comparative cost analysis is presented in Section 9.2.5.

9.2.1 Upgrade and Expand Existing Integrated Fixed Film Activated Sludge System

This alternative was intended to be based on upgrading and expanding the existing AccuWeb IFAS system, using the same web-type media as existing. However, Brentwood Industries, which supplied the existing facilities, no longer offers AccuWeb media due to their strong concerns regarding potential red worm infestations. Another company that now offers the web-type media, Entex Technologies, was contacted to see if they would propose on this project. The other company also declined to offer web-type media, saying their recommendation would be to use loose/moving IFAS media, such as being considered under the "New IFAS" alternative in Section 9.2.2.

Instead of using web-type media, Brentwood Industries is now recommending use of their new product, which is a structured sheet media, called AccuFAS. The AccuFAS media basically consists of large blocks of corrugated plastic sheets joined together and mounted on stands above aeration diffusers. Illustrations of an AccuFAS module and AccuFAS towers in an aeration basin are shown in Figure 9-9. Brentwood claims that they have done extensive testing of the AccuFAS media and are able to accurately predict performance of systems with this media. However, full scale municipal wastewater treatment plant performance data are quite limited, since there are only three existing installations, two of which have been in operation for only about one year and none of which are combined nitrification and denitrification systems, such as required at DSPUD.



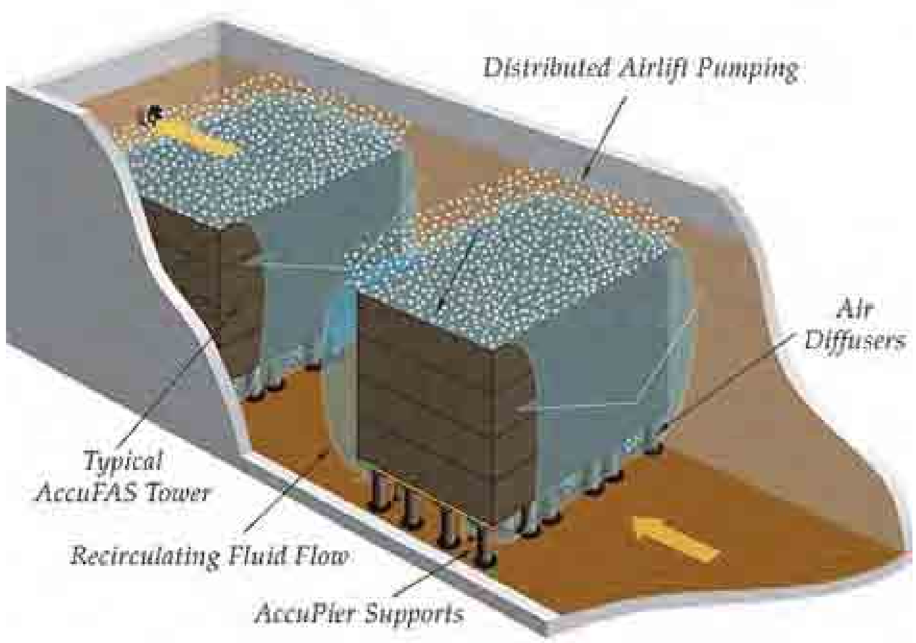


Figure 9-9
Brentwood AccuFAS Module and Conceptual Rendering of AccuFAS Towers in an Aeration Basin, Courtesy of Brentwood Industries

It was intended that two options for upgrading and expanding the existing IFAS system would be developed and investigated: a two-stage option (anoxic-aerobic) and a four-stage option (anoxic-aerobic-anoxic-aerobic). However, after analysis, Brentwood stated that the two-stage option is not recommended due to poorer performance, as compared to the four-stage option. Furthermore, in their analysis, they did not model conditions in the late fall with extensive supplemental ammonia addition, which would be most challenging for the two-stage option. Therefore, based on Brentwood's independent assessment and the assessment of ECO:LOGIC Engineering, only the four-stage option should be considered, and that is the basis of the analysis presented below.

Brentwood's recommendation is to use the existing clarifier compartments of the two existing steel package plants (Plant 1 and Plant 2) for pre-anoxic and post anoxic basins (divided with a new baffle wall). The annular reactor volume around the center anoxic basins would be mostly used for the first aerobic zone, but a smaller post-aeration basin would also be included. Both the first aerobic zone and the post-aeration basin would be fitted with AccuFAS media.

A separate splitter box, two new external clarifiers and a new RAS pump station would be required. The external clarifiers would be 40 feet in diameter (sizing determined by ECO:LOGIC, the same as for the New IFAS alternative) and would be fitted with modern features such as energy dissipating inlet, flocculating centerwell, density current baffles, and launder covers.

A flow diagram for the AccuFAS system is shown in Figure 9-10.

The proposed reactor design was developed by Brentwood Industries based on steady state simulation of the critical peak week loading condition. While this should be adequate to generally size the basins, if this alternative is selected for further consideration, dynamic modeling should be developed to refine the reactor design and confirm performance and chemical requirements with the variable flows and loads and supplemental ammonia feeding throughout the winter months, such as completed herein for the MBR alternative.

Estimated capital annual, and present worth costs for this alternative are presented in Section 9.2.5.

9.2.2 NEW INTEGRATED FIXED FILM ACTIVATED SLUDGE

Like the existing web-based IFAS system and the AccuFAS system discussed above, the new IFAS system considered in this section would provide biological treatment using both suspended and attached biomass growth. The most significant difference between the new IFAS option considered here and the previous options is that the media that would be used to support attached growth would be loose plastic shapes that are free to circulate throughout the reactor basin in which they are located. Screens or sieves would be used to keep the media from flowing out of the reactor basin with the mixed liquor outflow. A leading manufacturer of these types of systems (Kruger) provided the proposed design upon which this analysis is based. The specific product offered by Kruger is called Hybas, which is short for "Hybrid Activated Sludge". The media used in this system is AnoxKaldnes Moving Bed^{TM} media.

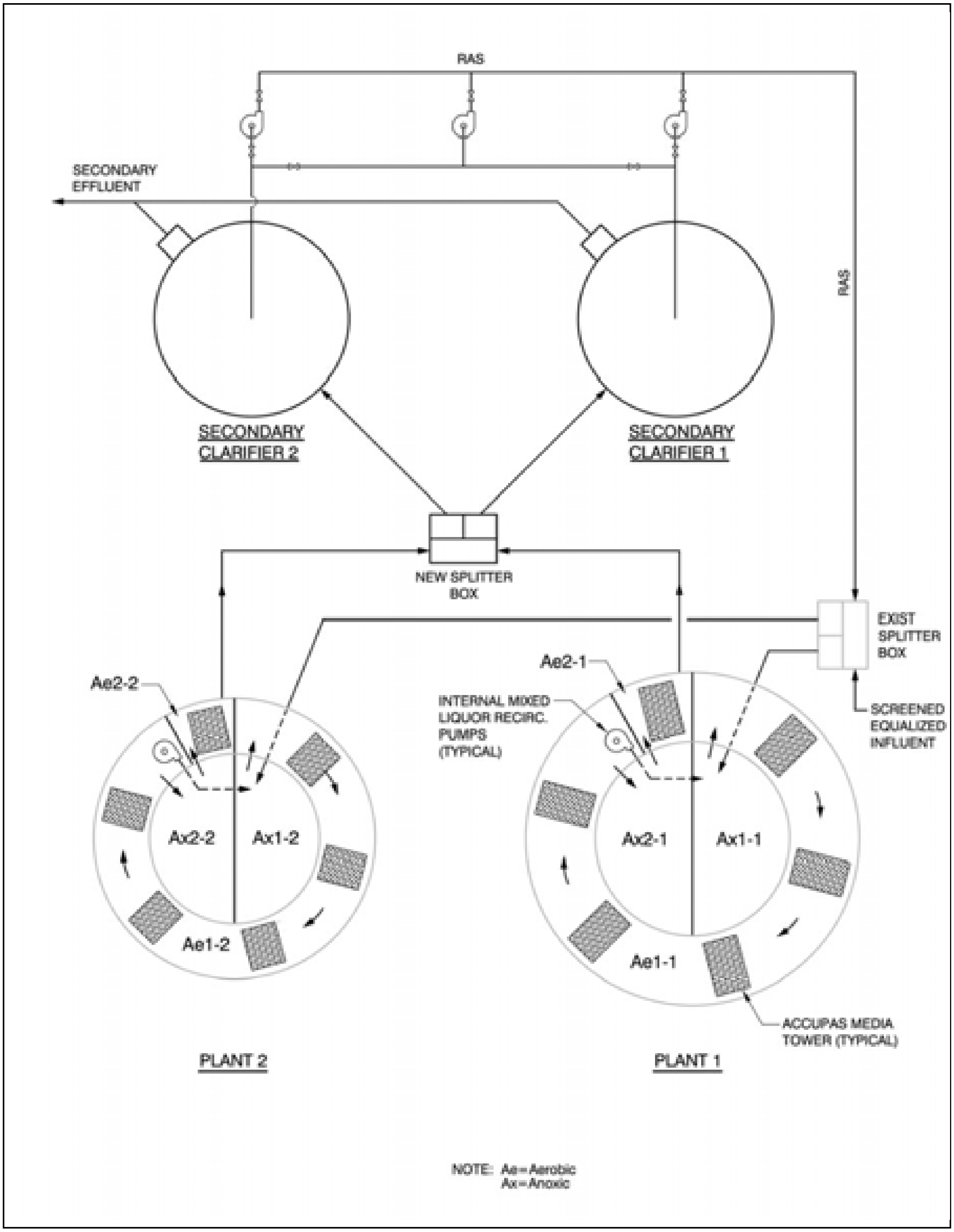


Figure 9-10
AccuFAS Flow Diagram

According to Kruger, the AnoxKaldnes Moving Bed™ process design is based on more than 20 years of experience with Moving Bed Biological Reactors (MBBR) and IFAS systems. There are more than 400 wastewater treatment facilities in 45 different countries, including the US, that utilize the Kruger AnoxKaldnes treatment process.

A photograph of an AnoxKaldnes biofilm carrier element is presented in Figure 9-11. Photographs showing typical screens used to retain the media in a reactor basin are presented in Figure 9-12.

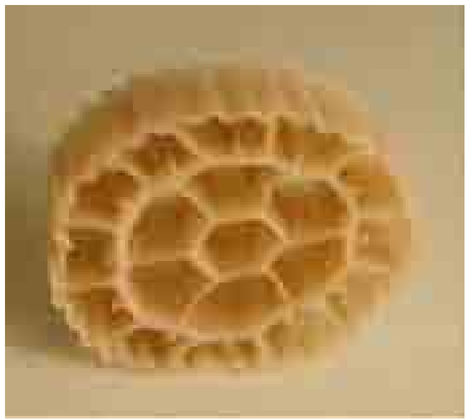


Figure 9-11
AnoxKaldnes K3 Media (Diameter About 1"), Courtesy of Kruger

Kruger recommended a four-stage process configuration, such as described for upgrading the existing IFAS system (anoxic-aerobic-anoxic-aerobic). All reactor basin compartments would be within the existing Plant 1 and Plant 2 structures. In each plant, the existing central clarifier would be converted to the first anoxic zone, while the subsequent three zones would be located in the annular space between the external walls and the central anoxic zone as shown in Figure 9-13.

The proposed system design was developed by Kruger based on steady state simulation of the critical peak week loading condition. While this should be adequate to generally size the basins, if this alternative is selected for further consideration, dynamic modeling should be developed to refine the reactor design and confirm performance and chemical requirements with the variable flows and loads and supplemental ammonia feeding throughout the winter months, such as completed herein for the MBR alternative.

Since the existing central clarifiers would be converted to first anoxic zones, two new clarifiers, a splitter box, and a RAS pump station would be required separate from the existing structures, as shown in Figure 9-13. As for the previous alternative, the clarifiers would be 40 feet in diameter and fitted with modern features, including energy dissipating inlets, flocculating centerwells, density current baffles, and launder covers.

Estimated capital annual, and present worth costs for this alternative are presented in Section 9.2.5.

(a)

(b)

Figure 9-12
AnoxKaldnes Media Retaining Screens
(a) Closeup
(b) Screens Installed in a Reactor Basin

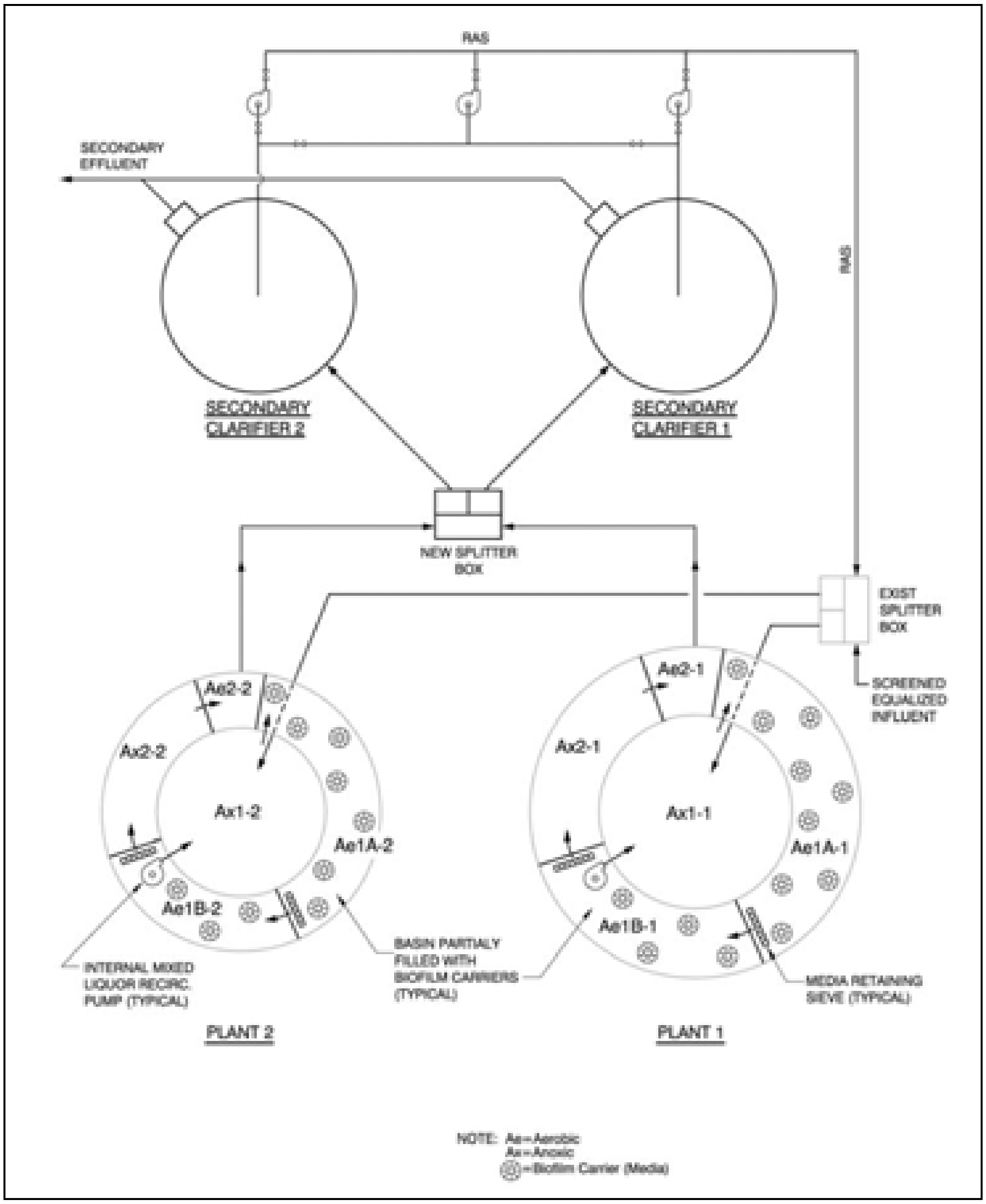


Figure 9-13
IFAS Flow Diagram

9.2.3 MEMBRANE BIOREACTORS

A membrane bioreactor (MBR) is a suspended growth biological treatment system like conventional activated sludge. However, in the MBR, the effluent clarifier is replaced by a membrane filtration system. Membrane filtration units are typically placed inside activated sludge reactor basins that are specifically designed and located for this use (membrane basins). Treated wastewater effluent is drawn through the membranes, leaving activated sludge solids behind. The membranes provide such a high level of solids removal, that the effluent from the MBR does not need further filtering through a granular media filter or equivalent device, such as required with conventional activated sludge. In fact, the MBR effluent is superior to the effluent of a conventional activated sludge system followed by effluent filters, having a typical effluent turbidity less than 0.2 NTU, compared to 2 NTU for the conventional system. The low turbidity is highly reliable because the membranes provide an absolute barrier to solids larger than the pore size of the membranes. Because solids settling in clarifiers is not required in a MBR system, mixed liquor solids concentrations can be typically about three times as high as those in a conventional activated sludge system. Because of this and because of the lack of clarifiers and separate effluent filters, the MBR system will have a much smaller footprint than a conventional system.

There are now a significant number of MBR manufacturers with many installations worldwide that could supply a system to meet the requirements at DSPUD. The membrane filtration systems of these various manufacturers are substantially different from each other and require different building and equipment layouts. Therefore, it is typical to have a separate bid process, evaluation, and selection of the MBR equipment prior to proceeding with detail design of the project. For this Facilities Plan analysis, proposals were received from two of the leading manufacturers (General Electric (previously Zenon)) and Enviroquip (licensed by Kubota)). The analysis presented herein and the costs are believed to be generally applicable to both of these manufactures, as well as others.

General Electric and several other manufacturers use tubular membranes arranged in modules, which are subsequently grouped in cassettes. A photograph of a General Electric membrane cassette is shown in Figure 9-14. Enviroquip and several other manufacturers use flat sheet membranes mounted on frames, which together form cartridges. Many cartridges are then placed side-by-side over an aeration system in a stainless steel box with open top and bottom through which mixed liquor is circulated. The completed assembly is called a membrane unit.
A conceptual rendering of several Enviroquip membrane units is shown in Figure 9-15.

Selection of Process Flow Diagram

Various process flow diagrams can be considered for the MBR alternative, depending on whether one or two anoxic zones are to be included and depending on how internal recirculation streams from the membrane basins and possibly the initial aerobic zone are configured. Five specific alternatives were investigated for this analysis, as shown in Figure 9-16. Each of the alternatives was evaluated under various flow and load conditions to determine recirculation flow and methanol addition requirements. For the alternatives with two anoxic zones, the relative amounts of denitrification and the methanol addition requirements for each of those zones were determined.

All of the alternatives include a recirculation flow from the membrane basins to one of the upstream basins. A minimum requirement for this recirculation flow is approximately four to six times the influent flow rate. The reason for this minimum is to keep mixed liquor solids concentrations around the membranes to a manageable level. Even with such high recirculation rates, the mixed liquor solids concentrations in the membrane basis will be about 17 to 25 percent greater than those in the other reactor basins, due to the fact that the final effluent is extracted through the membranes, leaving concentrated solids behind in the membrane basins.

Figure 9-14
General Electric Membrane Cassette with One Module Partly Removed, Courtesy of General Electric

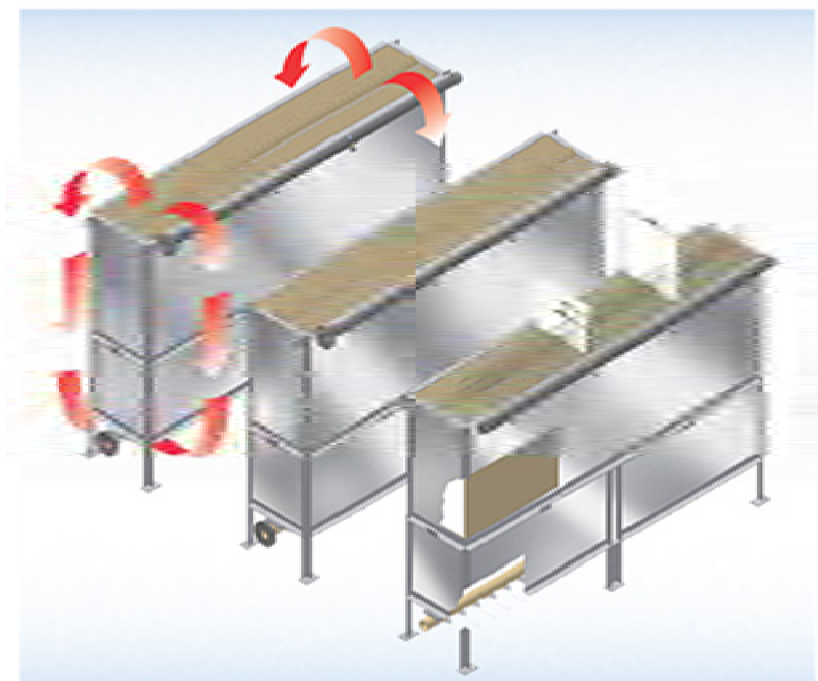


Figure 9-15
Conceptual Rendering of Enviroquip (Kubota) Membrane Units, Courtesy of Enviroquip

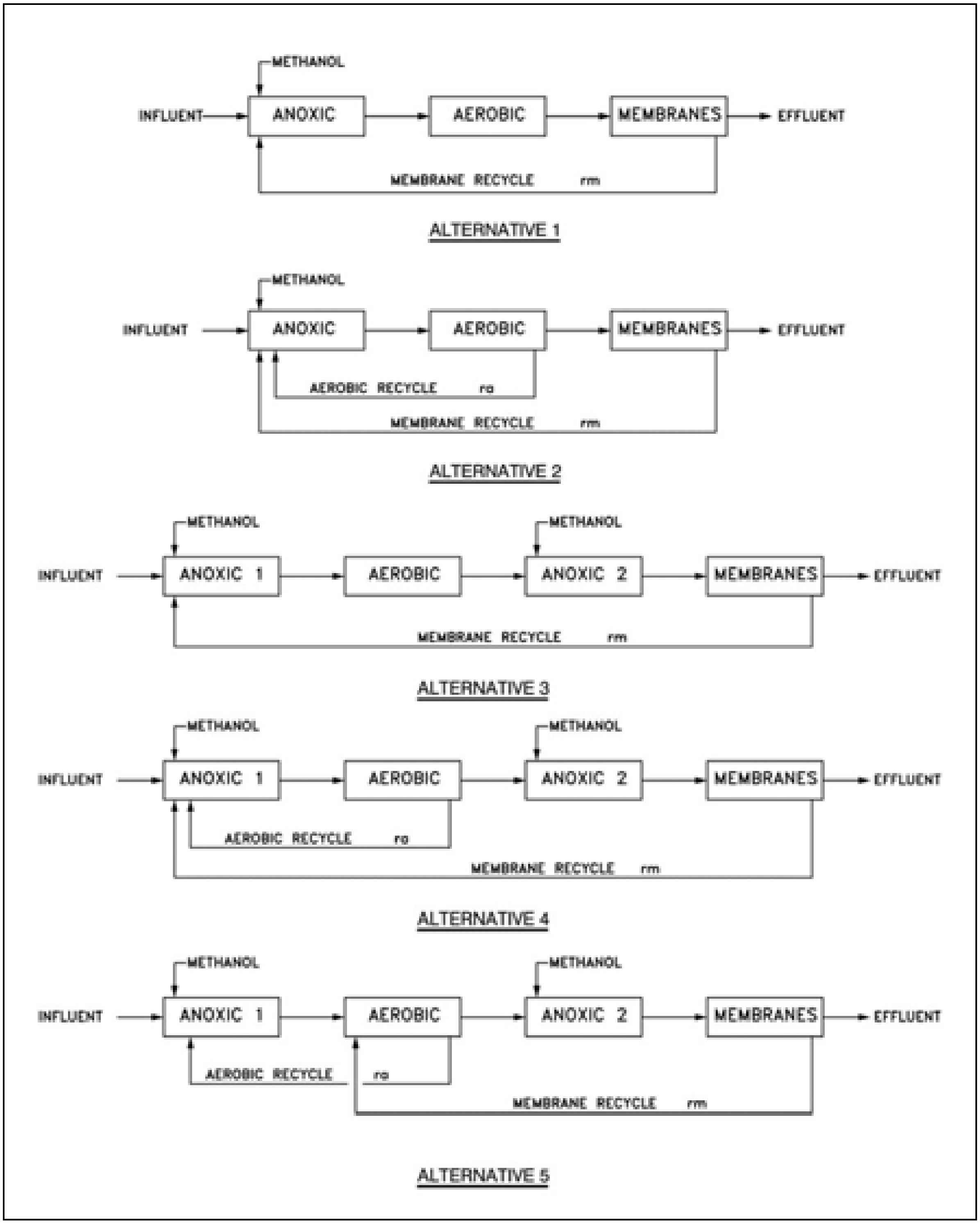


Figure 9-16
Alternative Flow Diagrams for MBR

Because of air scouring of the membranes, the dissolved oxygen concentration in the membrane basins will typically be quite high, perhaps around 6 mg/L, as compared to around 2 mg/L minimum desired for nitrification to occur. The high solids and dissolved oxygen concentrations in the membrane basins and in the recirculation flow from the membrane basins make the routing of this recirculation flow very important as regards basin sizing and methanol requirements for the anoxic zone(s).

Key results from the analysis of the five alternative flow diagrams are as follows:

1. Amongst the alternatives with only one anoxic zone, Alt. 2 is preferred over Alt. 1 for the following reasons:
    a. Less dissolved oxygen is delivered to the anoxic zone, resulting in lower methanol usage.
    b. It is much easier to recirculate flow from the first aerobic zone to the anoxic zone than from the membrane basins to the anoxic zone, because the first anoxic zone and aerobic zone will be in the same overall structure, separated by baffle walls, whereas the membrane basins will be completely separate.
2. Alt. 3 is undesirable because, with substantial ammonia supplementation, a large amount of denitrification would be forced to occur in the second anoxic zone, whereas at other times, most of the denitrification would occur in the first anoxic zone. To avoid over-sizing the second anoxic zone, it is best to have most of the denitrification occur in the first anoxic zone at all times. Additionally, methanol usage would be higher for this alternative than Alt. 4 due to the limited recirculation flow and/or high recirculation dissolved oxygen to the first anoxic zone.
3. Alt. 5 is undesirable because it would result in substantially diluting the mixed liquor solids concentration in the first anoxic zone, which would require enlarging the basin, as compared to Alt. 4. Additionally, Alt. 5 results in less of the total denitrification occurring in the first anoxic zone and less beneficial use of influent BOD for denitrification and, therefore, substantially more methanol usage as compared to Alt. 4.
4. Alt. 4 is preferred over Alt. 2 because having the second anoxic zone as a “second barrier” for nitrate would allow for more direct control and more reliable compliance with the effluent nitrate limit. Additionally, since some denitrification can occur in the second anoxic zone with Alt. 4, the undesirable extremely high recirculation ratios that would sometimes be required with Alt. 2 (this would occur with high influent ammonia concentrations due to supplementation) can be avoided. It is noted that the second anoxic zone for Alt. 4 would be relatively small and the total reactor basin volume would be about the same for Alts. 2 and 4.

Based on the above analysis, Flow Diagram Alternative 4 is recommended.

MBR System Description

The required basin volumes for the MBR alternative were determined based on conventional process design calculations as well as on computer simulations using the BioWin process simulator. For the relatively low design temperature of 7 °C and because of the presence of substantial anoxic volumes needed for denitrification, a design aerobic solids retention time in the range of 20 to 25 days is recommended. Recommended volumes for all of the reactor basins are approximately as follows:

Anoxic 1: 250,000 gallons (subject to swing zone adjustment)
Aerobic: 190,000 gallons (subject to swing zone adjustment)
Anoxic 2: 70,000 gallons
Membrane Basins: As Required by Manufacturer (typically under 60,000 gallons)

As noted above, the volumes of Anoxic 1 and Aerobic 1 can be adjustable by making a portion of the Anoxic 1 volume a swing zone that can be either anoxic or aerobic. This would allow the plant operators to fine tune process performance based on actual experience. All volumes would be subject to confirmation after selection of a final membrane manufacturer and refinement of the design based on manufacturer requirements.

It will be noted that the total reactor volume listed above, excluding the membrane basins is 510,000 gallons, which is the total volume contained in Plant 1 and Plant 2, including the clarifier basins. This volume is adequate for the future design condition and it is very beneficial to use the total volume available in these two structures to provide reliable treatment, while minimizing methanol usage. Because of the specific configuration requirements of the membrane basins, they have to be in a separate new structure.

The proposed MBR system would include two reactor basin trains, one in the existing Plant 1 structure and one in the existing Plant 2 structure. The proposed plan is to use the existing clarifier basins in the center of each plant to satisfy most of the volume requirements for Anoxic 1. The existing clarifier mechanisms would be removed and mixers would be installed. The remainder of Anoxic 1 (the swing zone), the aerobic zone and Anoxic 2, would be configured in the annular reactor basin area surrounding Anoxic 1. It is recommended that the aerobic zone in each plant be subdivided into two equal compartments to improve biological reaction kinetics, as compared to having one large zone. Steel baffle walls would be relocated or added as needed.

New fine bubble diffused aeration systems would be provided in the new aerobic zones and the swing zone. Existing anoxic mixers and mixed liquor recirculation pumps would be reused to the extent possible. The membrane basins and all of the MBR related equipment would be located in a new building.

A flow diagram for the MBR system based on utilizing the two existing plant structures as described is shown in Figure 9-17. Estimated capital annual, and present worth costs for this alternative are presented in Section 9.2.5.

Long-Term Dynamic Simulation of the MBR Alternative

To optimize and verify the effectiveness of the plan to feed supplemental ammonia and to verify overall compliance with ammonia and nitrate limits throughout the critical winter months, the proposed MBR system was simulated from September 1 through March 31 using BioWin process simulation software. Daily influent flows and loads for the simulation were based on actual data from 2007/2008, except that all flows were multiplied by 1.3 to provide conditions close to the future design criteria for peak month and peak week flows and loads.

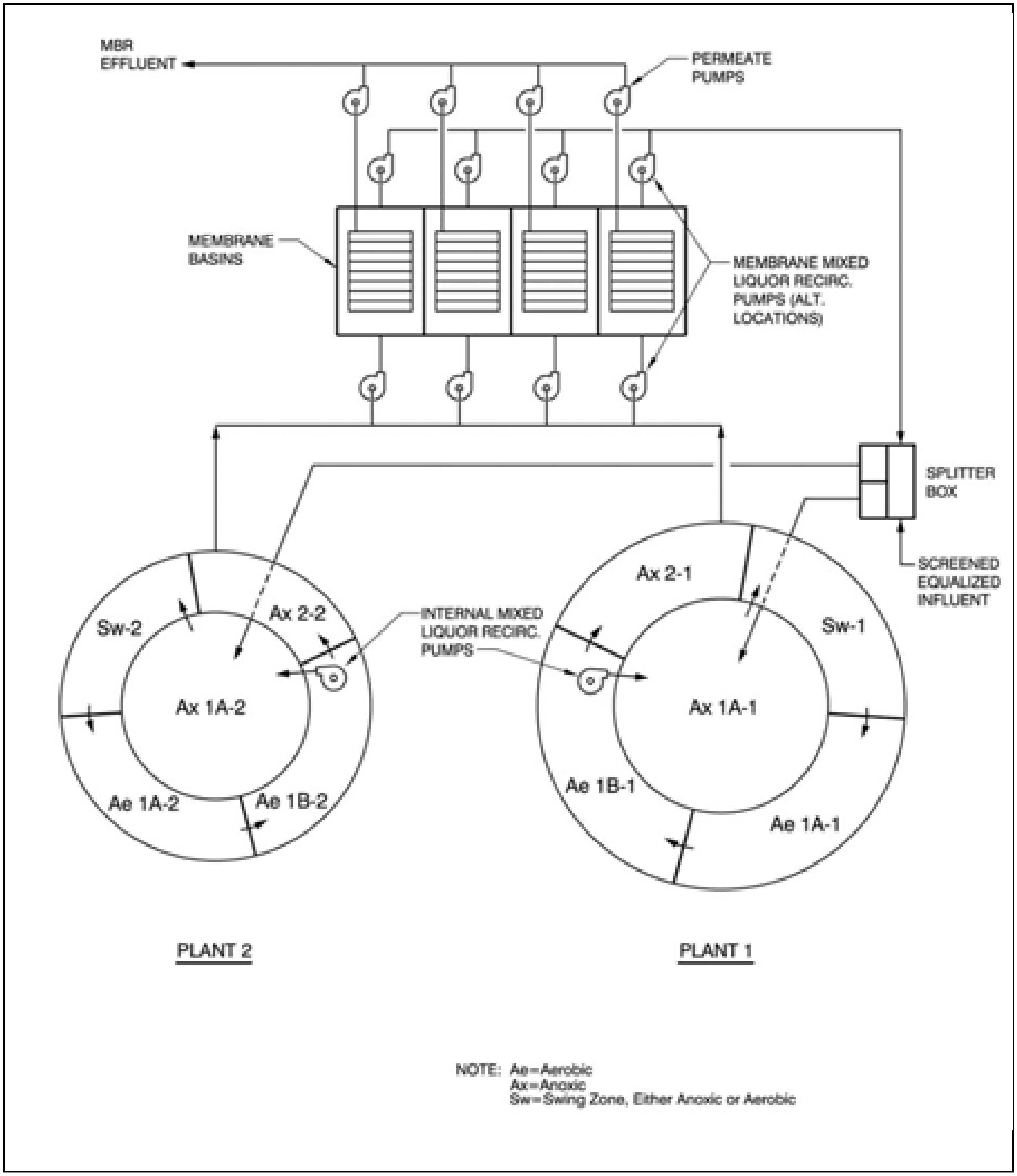


Figure 9-17
MBR Flow Diagram

The actual plant data for that period generally includes BOD monitoring only two times per week. Therefore, estimated influent BOD values were used for the missing days, assuming higher values on weekends than on weekdays. TKN was assumed to be 30 percent of the BOD. The dynamic simulation included influent flow equalization. The plant influent and equalized flows used in the simulation are shown in Figure 9-18, together with the resultant volumes in the equalization storage basin. Influent and equalized loading patterns for BOD and TKN (with and without supplementation) are shown in Figures 9-19 and 9-20, respectively. The bases of these flow and load patterns are discussed briefly below.

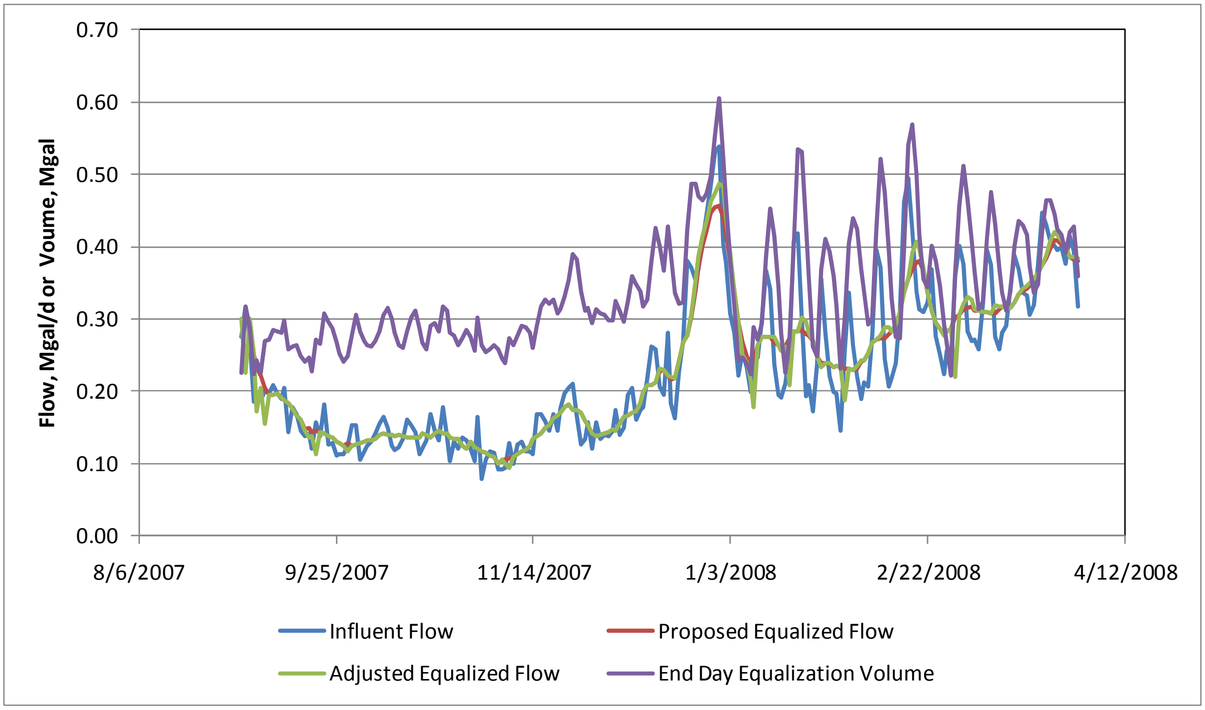


Figure 9-18
Influent and Equalized Flows and Equalization Volume for Dynamic Simulation

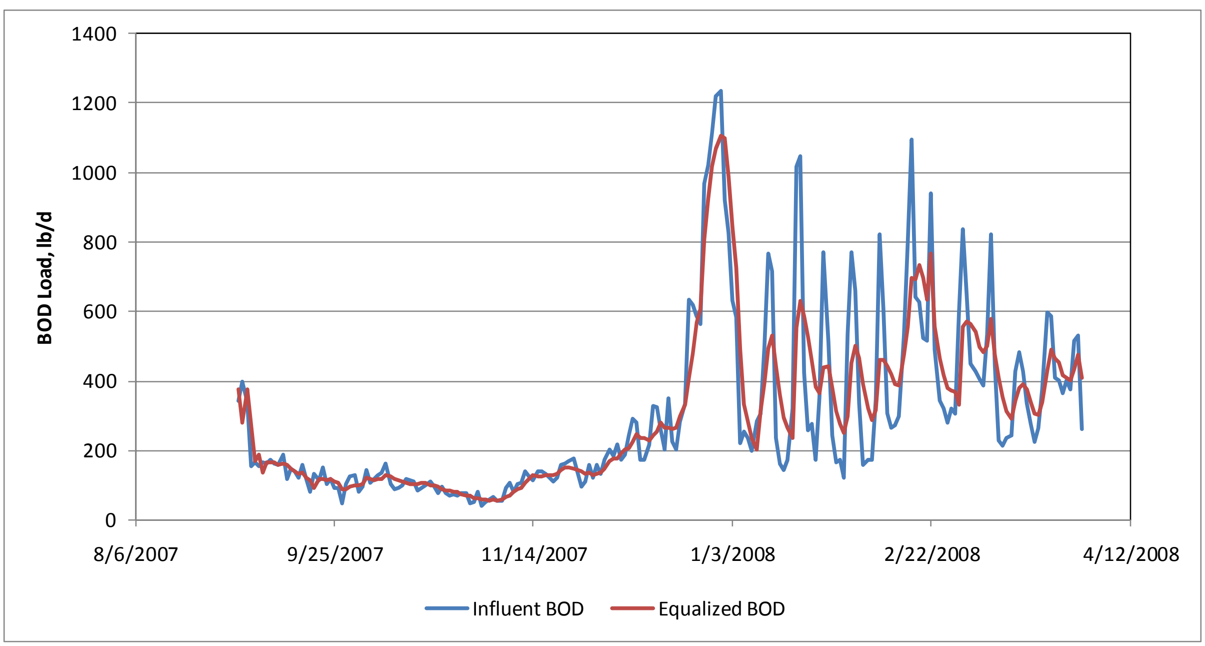


Figure 9-19
Influent and Equalized BOD Loads for Dynamic Simulation

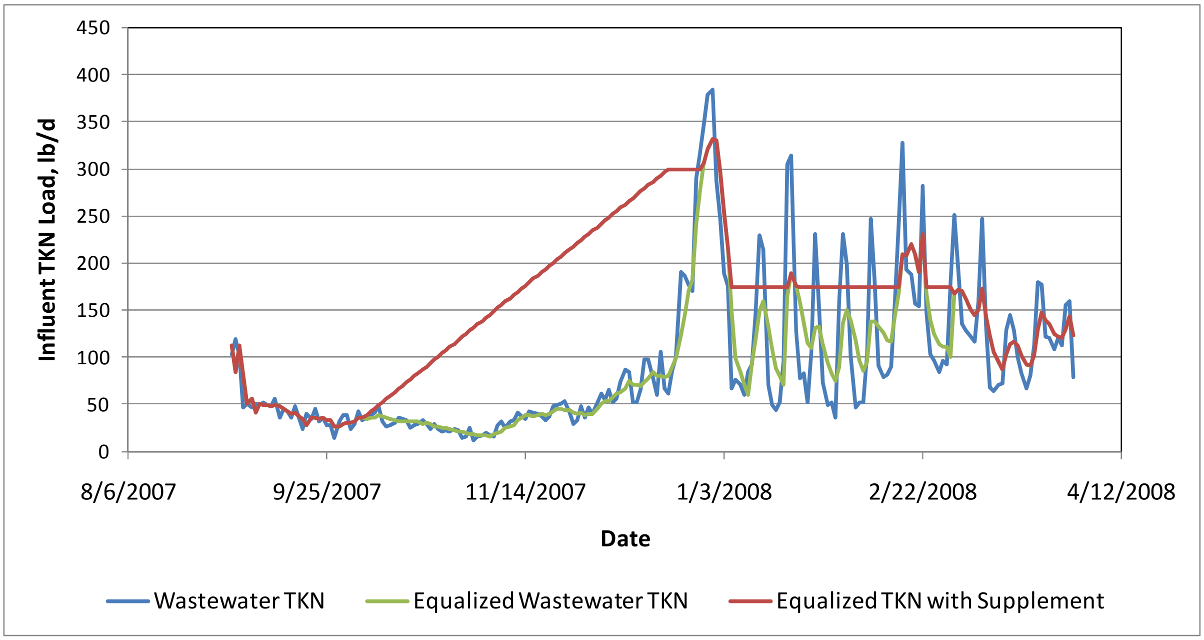


Figure 9-20
Influent, Equalized, and Supplemented TKN Loads for Dynamic Simulation

Flow Equalization Model

As discussed in Section 8, management of flow equalization volume accumulations and outflows is not an exact science; it requires estimates of future flows to be made by the operators and must include dynamic controls to adjust operator flow settings in the event that volume accumulations in the flow equalization basin approach levels that would threaten to drain the tank below minimum levels required for mixing or to fill the tank to overflow. To approximate real-world equalization controls, a model was developed in an Excel spreadsheet to determine “proposed” equalization outflow settings such as those that might be provided by an operator and then to track volume accumulations in the equalization basin using an appropriate control algorithm to adjust the proposed flow settings based on those volume accumulations. In the model, the daily influent flows were the actual adjusted flows from 2007/2008 mentioned previously. The proposed equalization outflow was determined daily as the seven-day average flow, including four days previous to the day in question through two days after the day in question. In essence, this methodology presumed the operator could estimate flows on the day in question and two days into the future based on recent flows, weather forecasts, and knowledge of whether the upcoming days would be ordinary weekdays, weekends or holidays. In reality, of course, there would be a substantial margin of error in the operator’s estimates of future flows. Knowing this, the operator may tend to be conservative and to pass more flow and accumulate less volume in equalization storage than would otherwise be possible. Nevertheless, the model is believed to give a reasonable representation of what an operator might do and how the automated equalization control system would respond.

For the model described above, the total equalization volume was assumed to be 750,000 gallons. To avoid draining the tank below 150,000 gallons desired for mixing, the proposed equalization outflow setting was multiplied by a factor that decreased from a value of 1.0 at a volume of 250,000 gallons to 0 at a volume of 150,000 gallons. Thus, for example, if the volume in equalization storage was 200,000 gallons, the proposed flow would have been multiplied by 0.5 to slow equalization outflow and prevent excessive draining of the tank. On the high end, the flow adjustment factor ranged from 1.0 at 400,000 gallons to 1.16 at 750,000 gallons. While the volume in storage was between 250,000 gallons and 400,000 gallons, the proposed flow was used without adjustment.

In the spreadsheet model of flow equalization, concentrations of BOD and TKN in the equalization basin were calculated daily by assuming that the daily influent load would mix into the contents of the equalization basin at that time. The supplemented TKN loads were developed based on a program of ramping up from the raw influent TKN load at the beginning of October to a load of 300 lb/d by mid-December. Then, throughout January, February, and March, TKN was added as needed to maintain a minimum supplemented TKN of 175 lb/d. Other ammonia supplementation patterns were tried, such as starting the ramp up in TKN loading at the beginning of November instead of the beginning of October. In that case, ammonia removal results were not adequate during the ramp up period. Additionally, terminating ammonia supplementation altogether in January through March was tried, but was unsuccessful due to ammonia breakthrough during peak loading conditions near the end of February.

BioWin Simulation

The flow diagram used for BioWin simulation is shown in Figure 9-21. Although the actual MBR system, if implemented, would include two process trains, these were combined into one train to simplify the simulation. The influent flows and constituent concentrations used in the simulation were the scaled 2007/2008 data previously discussed. Equalization basin outflows in the simulation were established as those developed in the spreadsheet model discussed above. Constituent concentrations in the equalization basin outflow were calculated in BioWin and closely matched those determined in the spreadsheet model. The ammonia supplement flows used in BioWin were those developed in the spreadsheet model. Temperatures in the reactor basins were set at 18 °C in September, 15 °C in October, 10 °C in November, and 7 °C for December through March. These temperatures are generally in accordance with the historical effluent temperatures shown in Figure 9-1, with a minimum at 7°C due to presumed heating.

Daily required methanol flows to the pre-anoxic and post-anoxic zones were estimated in a spreadsheet model with appropriate process design calculations, taking into account equalized influent BOD and TKN loads, recirculation flows, dissolved oxygen concentrations in recirculation flows, and required methanol to nitrate and methanol to oxygen ratios. The daily feed rates calculated in the spreadsheet model were used as input data for the BioWin simulation. In BioWin, nitrate and methanol concentrations were tracked throughout the plant to allow checking of appropriate methanol feed rates. In real plant operations, methanol feed rates would be determined using on-line sensors for nitrate. Although a control module is available for use in

BioWin to simulate the automated controls for methanol feeding, the control module was not used for these simulations.

The ammonia and nitrate nitrogen results of the long-term BioWin simulation are shown in Figure 9-22. As shown, the effluent ammonia-nitrogen concentration was always below the monthly average permit limit of 2.1 mg/L and the effluent nitrate-nitrogen concentration was always below the monthly average permit limit of 10 mg/L. The target effluent concentration for nitrate- nitrogen was 8 mg/L. The fact that the simulated actual effluent concentrations were under 8 mg/L most of the time indicates that methanol feed rates used in the simulation could have been reduced somewhat.

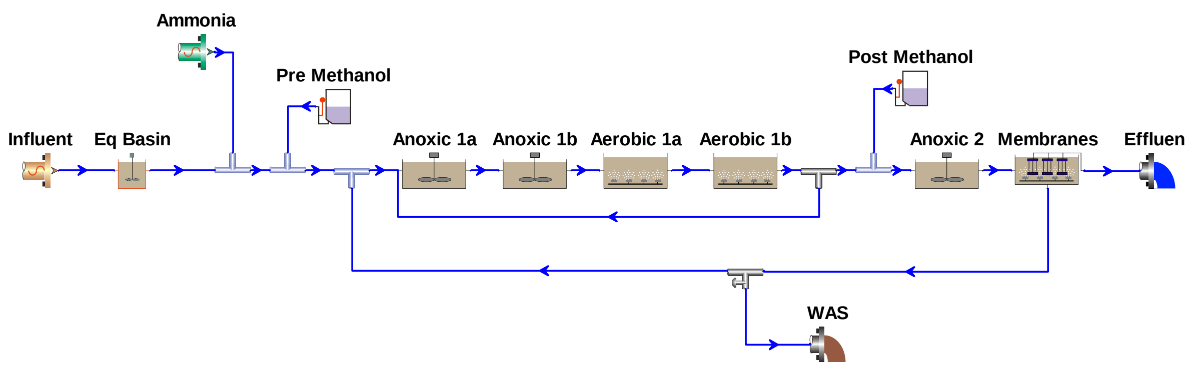


Figure 9-21
Flow Diagram for BioWin Simulation

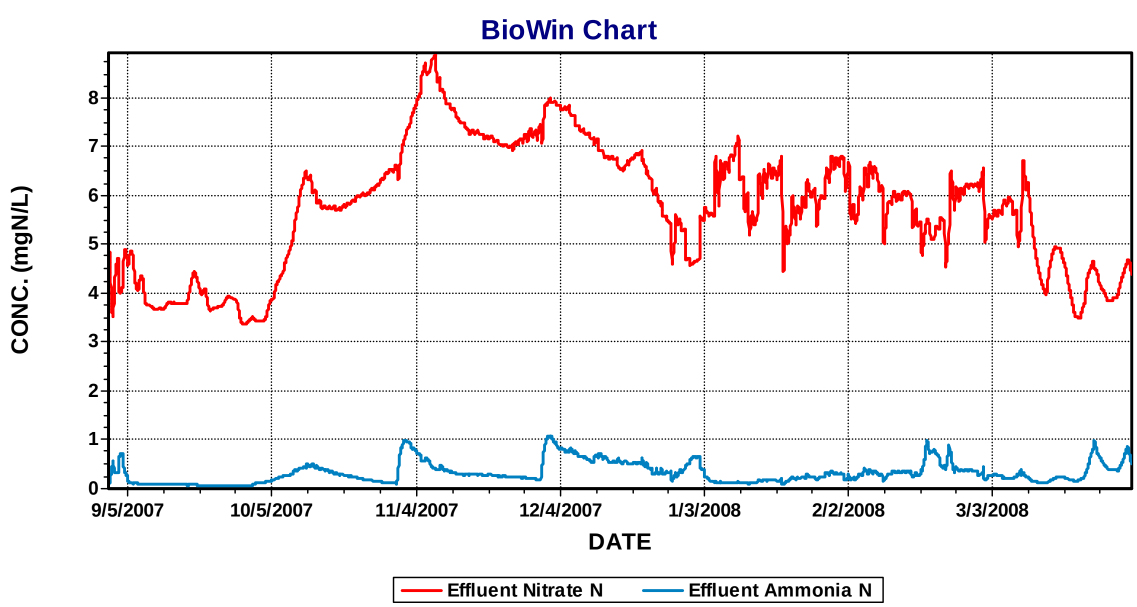


Figure 9-22
BioWin Simulation Results for Ammonia and Nitrate

9.2.4 SUBMERGED ATTACHED GROWTH

This treatment alternative utilizes an upflow biological aerated filter (BAF) with a submerged media bed. The microorganisms responsible for treatment grow on the surface of the media. The media bed, in addition to supporting biological growth, also acts to filter suspended solids from the wastewater. As biomass and wastewater solids accumulate in the bed, head loss increases, leading to the need for frequent backwashing.

The analysis in this section is based on the BIOSTYR system developed by Kruger, a leading manufacturer of submerged attached growth systems. According to Kruger, there are currently over 100 BIOSTYR installations worldwide, the earliest of which has been in operation in France since 1990. The operating facilities in the US range in size from 0.64 Mgal/d average (2.6 Mgal/d peak) in Mystic Lake, MN to 84 Mgal/d average (126 Mgal/d peak) in Syracuse, NY. Kruger's BIOSTYR plants are designed to accomplish BOD removal, nitrification and/or denitrification. The Tahoe Truckee Sanitation Agency recently upgraded their wastewater treatment plant with a BIOSTYR process.

For DSPUD, Kruger proposed to provide their proprietary MULTIFLO chemically-enhanced, high-rate primary sedimentation system ahead of the BIOSTYR reactors. The MULTIFLO process would remove significant amounts of BOD and TSS in a compact area, allowing substantial size reductions in the downstream BIOSTYR facilities. The BIOSTYR system would include two stages; the first to accomplish BOD removal, nitrification and partial denitrification and the second to complete denitrification. The first stage is called the nitrification/denitrification (NDN) stage and the second is called the post denitrification (PDN) stage. Each of these system components is discussed below.

Chemically-Enhanced High-Rate Primary Sedimentation

The MULTIFLO process combines coagulation and flocculation with plate settling technology and is carried out in a series of five tanks. The first three tanks are rapid mix tanks, the fourth tank is a flocculation tank, and the final tank is the settling tank containing inclined plates. Solids settle to the bottom of the fifth tank and are moved by a sludge scraper to a hopper at one end of the tank, from which they would be pumped to the solids handling facilities. The clarified effluent from the MULTIFLO system would be pumped to the nitrification/denitrification stage of the BIOSTYR system.

Two MULTIFLO process trains are proposed, with each train designed to handle the peak flow. The basin construction would be concrete, above ground structure with a footprint of approximately 65 feet long by 30 feet wide.

Nitrification/Denitrification Stage

BOD and ammonia will be completely removed and a significant amount of nitrate will also be removed in the nitrification/denitrification stage. Air is injected through an air grid located at the middle of the cell, thus creating a lower un-aerated zone and an upper aerated zone. In the upper aerated zone, BOD and ammonia are oxidized and nitrate is generated. A recycle stream is provided to transfer the nitrate formed in the upper zone to the un-aerated lower zone where nitrate is removed through denitrification, using the BOD in the influent wastewater as the carbon source. The BIOSTYR media consist of buoyant polystyrene beads that provide the surface area for biomass attachment. The media is retained in the BIOSTYR cell by a nozzle deck located above the media. The BIOSTYR backwash is a counter-current (downward) flow utilizing effluent water. Four NDN cells, including one redundant cell, would be required. Each cell would be fabricated of stainless steel and would be 13 feet in diameter and 25 feet high.

Post Denitrification Stage

The post denitrification stage is needed to remove the remainder of the nitrate not removed in the nitrification/denitrification stage. In this stage, methanol or an alternative carbon source must be added to facilitate denitrification. Three PDN cells, including one redundant cell, would be required. Each cell would be fabricated of stainless steel and would be 10 feet in diameter and 19 feet high.

BIOSTYR System Layout

A photograph of BIOSTYR reactors using stainless steel vessels similar to those that would be used at DSPUD is shown in Figure 9-23. However, considering winter conditions at DSPUD, the entire system would be enclosed in a building to provide operator access and protection of all of the mechanical equipment involved. An approximate 8000 square foot building would be required. Three concrete basins would be located under the floor of the building, including two clearwells and a mudwell. One clearwell would be required to receive the effluent from each stage of the BIOSTYR system. The water in the clearwell would be used for backwashing. One common mudwell would serve both stages and would receive spent backwash water for subsequent pumping back to the MULTIFLO system for treatment.

Figure 9-23
Steel Cell BIOSTYR System, Courtesy of Kruger

Cost Analysis

Estimated capital annual, and present worth costs for this alternative are presented in Section 9.2.5.

9.2.5 Comparative Cost Analysis for the Biological Treatment Alternatives

Capital, annual, and present worth costs for all of the biological treatment alternatives are presented in Table 9-4. This table, however, cannot be used independently to assess the overall cost effectiveness of the biological treatment alternatives, because the selection of a biological treatment alternative will impact other plant components. For example, the plant headworks and disinfection facilities would be substantially different for the MBR alternative than for the other alternatives. Additionally, the MBR alternative would not require separate effluent filtration, while the other alternatives would. Sludge handling costs would be substantially different for the submerged attached growth alternative than for the other biological treatment options. Therefore, the comparative evaluation of the biological treatment alternatives, including consideration of all other aspects of the plant that would be impacted by the choice of biological treatment alternative and consideration of non-economic factors, is presented in Section 17.

9.2.6 Process Design Comparisons Between the IFAS and MBR Alternatives

In Subsections 9.2.1 through 9.2.3 above, proposed designs for two IFAS alternatives and one MBR alternative are presented. It is useful to compare the proposed designs and to analyze the differences between them. The submerged attached growth alternative discussed in Section 9.2.4 is so different in concept that it cannot be included in the comparison.

Key process design characteristics for the two IFAS alternatives and the MBR alternative are shown in Table 9-5 and discussed below.

The process designs for all three alternatives are based on using the existing basins of Plant 1 and Plant 2 for the required reactor basins. For the MBR alternative, however, a separate membrane basin structure would be required, adding perhaps around 12 percent to the overall reactor volume, depending on the MBR manufacturer. The two IFAS designs are those suggested by the respective manufacturers, both of which determined that the existing basin volumes would be adequate, if new separate clarifiers are constructed.

Since wastewater treatment capacity is somewhat proportional to the amount of biomass that can be contained in the reactor basins, the maximum possible solids inventories in the systems is of primary importance. The MBR system can sustain mixed liquor suspended solids concentrations about double that which would be possible with the IFAS alternatives. However, the IFAS alternatives include additional solids inventory attached to the support media in the aerobic reactor basins. When all factors are considered, the MBR alternative can support about 80 to 100 percent more reactor basin solids inventory than the IFAS alternatives. This is not to suggest that the IFAS alternatives would be inadequate, rather that the MBR design would be very robust, with reserve capacity and operational flexibility, whereas the IFAS alternatives would have lesser inherent safety factors. In particular, the MBR alternative can hold much more biomass in the anoxic basins, which increases the amount of denitrification that can be accomplished without methanol addition.

Table 9-4
Biological Treatment Alternative Cost Analysis

Item	Upgrade Existing IFAS	New IFAS	MBR	Submerged Attached Growth
Capital Costs				
Demolition and Modification Inside Plant 1 and Plant 2 Structures	150,000	85,000	100,000	0
New Chemically-Enhanced Primary Clarification Structures	0	0	0	570,000
New Process Basins for Submerged Attached Growth	0	0	0	160,000
Main Process Flow Pump Stations in Treatment System	0	0	0	400,000
New Secondary Clarifiers and Splitter Box	930,000	930,000	0	0
New RAS Pump Station	300,000	300,000	0	0
Membrane Basins (b)	0	0	330,000	0
Main Vendor Equipment Package, Installed	450,000	1,300,000	1,900,000	3,100,000
Anoxic Mixers Installed	90,000	Included	90,000	0
Aeration Facilities Not in Main Equipment Package	250,000	50,000	250,000	0
Other Ancillary Facilities and Equipment	100,000	50,000	100,000	50,000
Internal Process Piping	50,000	50,000	300,000	300,000
Building Enclosures	150,000	150,000	950,000	2,000,000
Subtotal 1	2,470,000	2,915,000	4,020,000	6,580,000
Electrical and Instrumentation @ 25% of Subtotal 1	620,000	730,000	1,010,000	1,650,000
Site Piping @ 10% of Subtotal 1	250,000	290,000	400,000	660,000
Sitework @ 5% of Subtotal 1	120,000	150,000	200,000	330,000
Subtotal 2	3,460,000	4,085,000	5,630,000	9,220,000
Contingencies @ 20% of Subtotal 2	690,000	820,000	1,130,000	1,840,000
Subtotal 3	4,150,000	4,905,000	6,760,000	11,060,000
General Conditions, Overhead and Profit @ 20% of Subtotal 3	830,000	980,000	1,350,000	2,210,000
Total Construction Cost	4,980,000	5,885,000	8,110,000	13,270,000
Engineering, Administration and Environmental @ 25%	1,250,000	1,470,000	2,030,000	3,320,000
Total Capital Cost	6,230,000	7,355,000	10,140,000	16,590,000
Annual O&M Costs				
Labor	140,000	140,000	140,000	160,000
Power	35,000	35,000	40,000	20,000
Ammonia	20,000	20,000	20,000	16,000
Lime	12,000	12,000	12,000	11,000
Methanol (c)	17,000	17,000	14,000	17,000
Ferric Chloride	0	0	0	18,000
Other Chemicals	0	0	2,000	5,000
Maintenance Materials, Not Including Membranes	9,000	14,000	23,000	32,000
Membrane Replacement	0	0	0	14,000
Total Annual Cost	233,000	238,000	251,000	293,000
Present Worth Costs				
Present Worth of Annual Costs (d)	3,467,040	3,541,440	3,734,880	4,359,840
Total Present Worth	9,697,040	10,896,440	13,874,880	20,949,840

(a) First quarter 2010 cost level, ENR 20-Cities CCI = 8700.
(b) Depending on manufacturer, membrane basins may be prefabricated and part of equipment package.
(c) Methanol is assumed herein, but other carbon sources can be used and should be investigated during design.
(d) 20 years at inflation-adjusted discount rate of 3 percent, Present Worth Factor = 14.88.

Table 9-5
Comparison of IFAS and MBR Process Designs

Parameter	MBR	Upgrade Existing IFAS	New IFAS
Basin Volumes, gal			
Anoxic 1	250,000	85,700	173,000
Aerobic 1 (Includes 1a and 1b where applicable)	190,000	292,600	235,600
Anoxic 2	70,000	85,700	109,100
Re-aeration or Membrane Basins	60,000 (a)	47,100	5,300
Total	570,000	511,100	523,000 (b)
Maximum Mixed Liquor Suspended Solids, mg/L			
All Except Membrane Basins	8,000	3,000 (c)	3,000 (c)
Membrane Basins	10,000	---	---
IFAS Media and Related			
Media and Biofilm General			
Media Specific Area, ft2/ft3	---	86	152
Average Biofilm Solids, g TSS/m2	---	26	10.3
Aerobic 1 (Includes 1a and 1b where applicable)			
Volume Media, ft3	---	12,128	14,268
Media Surface Area, ft2	---	1,042,983	2,168,736
Solid Mass in Biofilm, lb	---	5,549	4,571
Maximum Solids in Mixed Liquor, lb	12,677	6,785	4,083
Maximum Total Solids Inventory, lb	12,677	12,334	8,653
Re-aeration or Membrane Basins			
Volume Media, ft3	---	2,021	0
Media Surface Area, ft2	---	173,831	0
Solid Mass in Biofilm, lb	---	925	0
Maximum Solids in Mixed Liquor, lb	5,004	1,089	133
Maximum Total Solids Inventory, lb	5,004	2,014	133
Total Aerobic			
Volume Media, ft3	---	14,149	14,268
Media Surface Area, ft2	---	1,216,814	2,168,736
Solid Mass in Biofilm, lb	0	6,473	4,571
Maximum Solids in Mixed Liquor, lb	17,681	7,874	4,215
Maximum Total Solids Inventory, lb	17,681	14,348	8,786
Maximum Solids Inventory, lb			
Total Anoxic	21,350	4,288	7,058
Total Aerobic	17,681	14,348	8,786
Grand Total	39,031	18,636	15,844

(a) Varies, depending on manufacturer.
(b) Minor adjustment in volumes to be made during pre-design to give total volume of 511,000 gal.
(c) Actual manufacturer proposed designs based on 3000 mg/L. Up to about 4000 mg/L would be possible.

Between the two IFAS alternatives, very substantial differences in fixed film surface area can be seen. The upgrade of the existing IFAS system proposed by Brentwood Industries includes about 1.2 million square feet of media surface area, compared with about 2.2 million square feet for the New IFAS alternative. However, Brentwood indicates that the biomass density on their media will be more than double that indicated for the New IFAS alternative (26 versus 10.3 g/m^2). Design and operational factors that would explain the difference in biomass density were not determined. The net result of all these differences, coupled with a somewhat larger aerobic volume for the Existing IFAS alternative, is a much greater aerobic solids inventory as compared to the New IFAS alternative.

The proposed anoxic volumes and anoxic solids inventories of the two IFAS alternatives are also quite different, with those of the New IFAS alternative being more than 60 percent greater than those of the Existing IFAS alternative.

The fact that the New IFAS alternative includes a lesser aerobic solids inventory and a greater anoxic solids inventory than the Existing IFAS alternative is indicative of quite different design approaches. For the existing IFAS alternative, Brentwood expects that the majority of the required denitrification will actually occur in the aerobic media-filled reactors through simultaneous nitrification and denitrification (SND) within the biofilm. While Kruger (the manufacturer upon which the New IFAS alternative is based) recognizes that significant SND can occur on their media also, they do not take any credit for that; they presume that all required denitrification will occur solely in the anoxic zones. In both systems dissolved oxygen concentrations in the media reactors would be relatively high (around 4 to 5 mg/L) to assure adequate penetration of dissolved oxygen into the biofilm to assure essentially complete nitrification.

Because of the substantial differences in proposed designs between the two IFAS alternatives, combined with the comparison with the MBR alternative, it is warranted to undertake further investigations to confirm that any proposed IFAS design will meet the District's treatment objectives. For many IFAS applications, pilot studies are conducted to refine the design criteria for the full-scale system. Alternatively, or in addition, DSPUD may seek to obtain a very substantial process performance warranty for any proposed system. These options should be discussed with prospective manufacturers of the type(s) of system(s) that DSPUD wishes to investigate further. Regardless of what other actions are taken, DSPUD staff and engineers should certainly visit existing IFAS installations of the type(s) being considered to see these systems first-hand and to discuss operational issues with agencies that have already installed them.

9.3 CHEMICAL STORAGE AND FEED SYSTEMS ASSOCIATED WITH BIOLOGICAL TREATMENT

For all of the biological treatment alternatives considered above, it would be necessary to feed supplemental ammonia during low-load periods in the fall and winter to develop and maintain an adequate population of nitrifying bacteria to handle peak loads during the winter ski season. Furthermore, all of the alternatives include methanol (or alternative carbon source) addition to assure adequate denitrification and alkalinity addition to maintain a stable pH. The facilities required for storage and feeding of these chemicals that are common to all of the biological treatment alternatives are discussed below.

This section does not include the chemical storage and feed systems required as part of the chemically-enhanced primary clarification system associated with the submerged attached growth biological treatment alternative. The costs of the chemically-enhanced primary clarification system, including the associated chemical feed systems, are included in the cost estimate for the submerged attached growth process.

This section also does not include consideration of chemical storage and feed systems not specifically related to the biological treatment alternatives, such as those associated with disinfection or sludge dewatering. Those systems are considered in the sections of this report dealing with the corresponding facilities.

9.3.1 AMMONIA STORAGE AND FEED SYSTEM

There are two existing ammonia feed systems, one for Plant 1 and one for Plant 2. For both systems, 150 pound cylinders of anhydrous ammonia are used as the supply. The cylinders contain ammonia liquid and ammonia gas under pressure. The ammonia gas is withdrawn from the cylinders and fed to the treatment plants through pressure regulators and other controls and is dissolved in the reactor basins through a submerged diffuser in each plant. As the ammonia gas is withdrawn from the cylinders, some of the remaining ammonia liquid is evaporated to maintain the gas supply. Evaporation consumes heat and cools the cylinders, which limits the maximum rate of evaporation and ammonia withdrawal from each cylinder to 32 lb/d at a room temperature of 70 °F. The feed system for Plant 1 has six cylinders connected together on a common manifold, while the system for Plant 2 has four cylinders, resulting in feed capacities of 192 and 128 lb/d, respectively.

The design peak week influent TKN (includes ammonia-nitrogen and organic nitrogen) load for the proposed project is 310 lb/d. This peak week loading condition would typically be expected to occur between the Christmas and New Year Holidays. In November and early December, when flows and loads are low, it is planned to feed ammonia so that the total influent TKN with supplement will increase gradually, reaching the 310 lb/d level just before Christmas. The amount of ammonia to be supplemented at any given time will be the target total TKN minus the TKN contained in raw sewage. As a reasonable worst-case in early to mid-December, it is projected that an ammonia-nitrogen feed rate of up to 220 lb/d could be required. Considering that ammonia contains approximately 82 percent nitrogen, the corresponding ammonia feed rate would be about 270 lb/d. Approximately 59 percent of this amount, or 159 lb/d should go to Plant 1, while 111 lb/d should go to Plant 2, for the biological treatment alternatives making use of these existing basins.

The design ammonia feed requirements presented above apply to all biological treatment options, except the submerged attached growth option considered in Section 9.2. For submerged attached growth, the design ammonia feed rates would be reduced by about 15 percent due to TKN removal in the primary clarifier and the resultant reduction in peak week design load to the secondary treatment process. However, it is presumed that the capital cost of required ammonia feed improvements would be about the same for all biological treatment alternatives. Differences in annual chemical usages and costs are included in the analyses of the biological treatment alternatives.

Although the proposed design ammonia feed rates are within the existing ammonia feed capacities, the existing ammonia feed systems include various deficiencies that should be addressed in the proposed project:

1. Spare ammonia cylinders (to replace empty cylinders when needed) are currently stored outside the existing Equipment Building in a covered walkway. These cylinders should be stored inside a building.
2. The ammonia feed system to Plant 1 was originally installed as a temporary system and includes plastic piping and unreliable pressure and feed rate controls. It is desired to convert the ammonia feed system for Plant 2 to be like that for Plant 1.
3. There are no emergency shutoff valves on the ammonia cylinders to shut off the supply in case of a major leak and there is no containment and scrubbing system for leaking ammonia. Since ammonia is a hazardous gas, appropriate emergency features should be provided in accordance with the Uniform Fire Code.

It is recommended that a new and larger ammonia storage and feed room be provided as part of the proposed project. The location of the new facilities depends on which biological treatment alternative is selected for implementation and what new buildings are required.

Estimated capital costs for the ammonia feed improvements are shown in Table 9-6.

Table 9-6
Ammonia Storage and Feed System Capital Cost Estimate[a]

Item	Cost, $
New Ammonia Storage and Feed Room (200 sq. ft. @ $200)	40,000
Replace Plant 2 Ammonia Feed System	50,000
Relocate Plant 1 Ammonia Feed System	10,000
Emergency Shutoff Valve System	75,000
Subtotal 1	175,000
Electrical and Instrumentation	30,000
Subtotal 2	205,000
Contingency, 20%	40,000
Subtotal 3	245,000
General Conditions, Overhead and Profit, 20%	49,000
Total Construction Cost	294,000
Engineering and Administration, 25%	74,000
Total Capital Cost	368,000

[a] First quarter 2010 cost level, ENR 20-Cities CCI = 8700

9.3.2 METHANOL (OR ALTERNATIVE CARBON SOURCE) STORAGE AND FEED SYSTEM

In the denitrification process, influent organic matter must be metabolized by the microorganisms as they convert nitrate to nitrogen gas. The organic matter is referred to as a carbon source because the organic compounds that are used as food for the microorganisms are carbon-based. When the amount of organic matter in the influent wastewater is inadequate and when denitrification is otherwise to be accomplished after the influent organic matter is consumed, a supplemental carbon source must be added. Typically, methanol has been used for this purpose. However, other carbon sources are possible. For example, some plants located nearby industrial areas can make use of waste products from those industries, particularly food processing wastes. Additionally, there are commercially available carbon source liquids that are specifically manufactured for wastewater treatment plant use. For this study, it is assumed that methanol will be used at DSPUD; however, other carbon sources should be investigated before final plant design. Because methanol is a flammable liquid, special precautions must be taken in the design of storage and feed systems. Therefore, it is likely that the costs developed herein based on methanol will be adequate to cover other carbon source options.

Design maximum month methanol feed requirements for the various biological treatment options are expected to be in the range of 40 to 60 gpd. For practical purposes, the methanol facilities would be essentially the same for all options and would include storage tanks, a receiving station, a feed pump system, and a fire control system. To always have an approximate one month supply on hand, to facilitate economical deliveries, and to provide reliability, two 2,000 gallon double-wall methanol storage tanks are recommended. The tanks would be similar to those used to store diesel fuel and would be located outdoors, away from the plant's main traffic area to minimize dangers associated with the possibility of explosions caused by methanol fumes. A metal canopy over the tanks is recommended for protection from direct sunlight, rain, and snowfall. Explosion proof metering pumps would be used to feed the methanol and would be located in a small building adjacent to the storage tanks. The pump room would be equipped with a fire suppression system. Estimated capital costs for the methanol storage and feed system are shown in Table 9-7.

9.3.3 ALKALINITY STORAGE AND FEED SYSTEM

The biological process of nitrification consumes alkalinity, which if not replaced, can result in pH depression that would inhibit proper treatment and cause various discharge permit violations. Alkalinity is also consumed during chlorination and dechlorination with chlorine and sulfur dioxide. Additionally, for the submerged attached growth alternative, alkalinity would be consumed in the chemically-enhanced primary treatment system. Design peak monthly average alkalinity addition requirements are expected to range from about 600 to 800 lb/d as calcium carbonate for the various biological treatment options, with minor differences depending on the method of disinfection to be selected. For short-term peaks and under critical operating conditions, such as might occur on peak days during the peak week, the requirement could be as much as about three times the monthly average, or up to about 2,400 lb/d.

Table 9-7
Methanol Storage and Feed System Capital Cost Estimate[a]

Item	Cost, $
Storage Area Slab and Canopy (400 sq. ft. @ $150)	60,000
Pump Room (100 sq. ft. @ $200)	20,000
Methanol Storage Tanks	60,000
Methanol Feed Pumps	30,000
Piping	20,000
Receiving Station	10,000
Fire Suppression System	25,000
Subtotal 1	225,000
Electrical and Instrumentation	30,000
Subtotal 2	255,000
Contingencies, 20%	51,000
Subtotal 3	306,000
General Conditions, Overhead and Profit, 20%	61,000
Total Construction Cost	367,000
Engineering and Administration, 25%	92,000
Total Capital Cost	459,000

[a] First quarter 2010 cost level, ENR 20-Cities CCI = 8700

At the present time, the plant includes an alkalinity storage and feed system based on the use of soda ash (Na_2CO_3). The system includes a bulk chemical storage silo together with a slurry batch and feed system. The system can store up to 35 tons of soda ash and can feed soda ash in solution at up to a rate of about 6,000 lb/d. It takes 1.06 pounds of soda ash to equal 1.0 pound of alkalinity. Therefore, using the 800 lb/d and 2,400 lb/d maximum monthly and peak design rates given above, the soda ash required would be about 850 lb/d and 2,500 lb/d, respectively. Therefore, the existing system has adequate capacity and would not need to be modified, if continued use of soda ash was planned.

The addition of soda ash does not provide any hardness to the wastewater, and hardness is desirable to minimize the toxicity of certain metals, such as copper and zinc. The allowable concentrations of these metals in the wastewater treatment plant effluent are increased with increased hardness. Therefore, it is desirable to switch from using soda ash for alkalinity addition to using hydrated lime ($Ca(OH)_2$), which would substantially increase the wastewater effluent hardness (the calcium in the lime would contribute to effluent hardness). It takes only 0.74 pounds of hydrated lime to equal 1.0 pound of alkalinity. Therefore, using the 800 lb/d and 2,500 lb/d maximum monthly average and peak design feed rates given above, the corresponding hydrated lime requirements would be about 590 lb/d and 1,800 lb/d, respectively.

The existing soda as feed system can be converted to feed hydrated lime, with minor modifications. Based on preliminary discussions with the manufacturer, it is estimated that only the slurry feed pumps would have to be changed out, at an estimated base cost of about $20,000. With associated electrical work, contingencies, and general contractor markups and profit, the total construction cost estimate is $36,000.

Tables

Figures

Section 10

Tertiary Filtration

Section 10

Tertiary Filtration

The purpose of this section is to evaluate the existing effluent filtration system and determine the required improvements to these facilities.

10.1 Existing Facilities

Secondary effluent flows by gravity from secondary clarifiers to three filter cells located inside the Advanced Treatment Building. Each filter cell is 72 square feet and can process a design flow of 0.52 Mgal/d at the design filtration rate of 5 gpm/ft^2. The original design is expandable to total of four filter cells. However, adding a new filter cell, if needed, would require moving the east wall of the filter room further east (as provided for in the original design), thereby encroaching upon the size of the maintenance shop. Filtered effluent flows by gravity to the chlorine contact basin.

There are three identical 25 hp self priming pumps, associated with the filters; one pump is used for backwash supply, one is used to return spent backwash water to the equalization storage basin, and the third is a standby pump that can be used for either of these functions. The backwash pump withdraws chlorinated effluent from the end of the chlorine contact basin and pumps it through the filter to wash the media. As mentioned in Section 5, during each backwash cycle, the chlorine contact basin is drawn down, causing the plant effluent flow to stop until the basin can refill after completion of backwashing. This starting and stopping of plant effluent flow creates problems with the flow-paced control of the sulfur dioxide feed system used for plant effluent dechlorination.

10.2 FILTER SYSTEM CAPACITY ASSESSMENT

The effluent filtration system must be designed to accommodate the peak wet weather flows through the wastewater treatment plant. As developed in Section 4, the design average day maximum weekly influent flow is 0.74 Mgal/d. Adding a reasonable allowance of about five percent for recycling of filter backwash water, the peak flow that needs to be filtered is about 0.78 Mgal/d, under normal conditions. However, it is desirable to have a higher capacity to allow for emergency peak flows, such as might occur with premature filling of the equalization storage basin during peak flow events.

As mentioned previously, each of the three filter cells can handle a flow of 0.52 Mgal/d at the design filtration rate of 5 gpm/ft^2. However, the filters have relatively coarse anthracite media (1.5 mm effective size), which is capable of sustaining filtration rates substantially greater than 5 gpm/ft^2. Although some decrease in filter performance might occur with higher filtration rates, this should not be a major concern for emergency peak flow events lasting a few hours or perhaps even a few days. Therefore, for this analysis, it is assumed that emergency peak flows of 7.5 gpm/ft^2 (and even higher) would be acceptable. On that basis, the emergency peak flow capacity for each cell would be at least 0.78 Mgal/d. Then, with two cells in service and one cell in backwash, the capacity would be 1.56 Mgal/d. With all three cells in service, the capacity would be over 2 Mgal/d, although there may be other hydraulic constraints that would preclude such flows. The original design of the filtration system was based on a peak hydraulic capacity of 1.7 Mgal/d, expandable to 2.6 Mgal/d.

Based on the above discussion, the existing filtration system has adequate capacity to handle the proposed influent design average day maximum weekly flow of 0.74 Mgal/d, and emergency peak flows at least double that flow. Therefore, no filtration system expansion is needed. However, some modifications are appropriate, as discussed in the following pages.

10.3 TERTIARY FILTRATION SYSTEM IMPROVEMENTS

Proposed improvements to the tertiary filtration system include provision of a backwash supply system and, depending on the biological treatment alternative to be selected, possible provision of a flocculation basin. These improvements are discussed below.

10.3.1 BACKWASH SUPPLY SYSTEM

A new backwash supply tank is recommended so that it would not be necessary to draw down the chlorine contact basin and stop and start plant flows for filter backwashing. A conceptual layout of the backwash supply tank and the pump system that would be used to fill the tank is shown in Figure 10-1.The backwash supply tank would be constructed of steel (probably prefabricated and bolted on-site) and would have a volume of about 20,000 gallons (approximately two backwashes). The tank would be filled slowly by pumping at a gradual rate, less than the plant flow rate, from the end of the chlorine contact basin. At the time of filter backwashing, the backwash supply tank would be drawn down, leaving the chlorine contact basin filled and the plant effluent flow without interruption.

Self-priming pumps (one duty and one standby) located adjacent to the chlorine contact basin would be used to fill the backwash supply tank. The supply to the pumps would be through the existing backwash supply intake screen in the chlorine contact basin. The pumps would be provided with variable frequency drives to allow the operator to choose the rate of filling the backwash supply tank, depending on plant flows. Alternatively, the pumping rate could be determined and controlled automatically, again, depending on plant flow. Assuming a minimum total fill time of four hours, each pump would be sized for a maximum capacity of about 80 gpm (0.12 Mgal/d). Normally, the flow rate would be lower.

A cost estimate for the proposed backwash supply system is presented in Table 10-1.

Table 10-1
Capital Cost Estimate for Adding a Backwash Supply System

	Cost, $
Concrete slab	11,000
20,000 gal tank	50,000
Tank Fill Pumps	10,000
Piping, valves and fittings	15,000
Subtotal 1	86,000
Electrical and site work	26,000
Subtotal 2	112,000
Contingency @ 20%	22,000
Subtotal 3	134,000
General conditions, overhead, and profit	27,000
Total Construction Cost	161,000
Engineering and Administration @ 25%	40,000
Total Capital Cost	201,000

10.3.2 COAGULATION AND FLOCCULATION

Chemical addition and appropriate rapid and slow mixing facilities upstream of filters are frequently used to promote coagulation and flocculation of small particles so they can easily be removed in the filters, reducing effluent turbidity and TSS concentrations. Although coagulation and flocculation may not always be required to meet effluent limits, they can be useful to improve effluent quality during unpredictable plant upsets.

The existing filtration system includes provisions for injection and passive in-line mixing of a chemical coagulant ahead of the filters. Once inside the filter cells, gentle turbulence in the pool maintained above the filter media, can provide some flocculation. However, there are no formal rapid mix or flocculation facilities.

Whether or not coagulation and/or flocculation are needed depends on the performance of the upstream secondary treatment process. When an activated sludge or IFAS system is functioning well, there is little or no need for chemical coagulation or flocculation ahead of the filters. However, at times of process stress conditions, chemical coagulation and some level of flocculation are useful. As a possible alternative, chemicals can be fed to the secondary clarifiers to improve performance there, which is a practice frequently employed at DSPUD. In Table 10-2, the possible needs for chemical coagulation and flocculation ahead of the effluent filters are indicated for each of the biological treatment alternatives considered in this study. As indicated in the table, it is considered likely that coagulation and flocculation would be required with the submerged attached growth biological treatment alternative, but not for the IFAS or MBR alternatives. However, the need for coagulation and flocculation with submerged attached growth biological treatment has not been confirmed by review of performance of existing facilities. Further investigation would certainly be warranted if this biological treatment option is seriously considered for implementation.

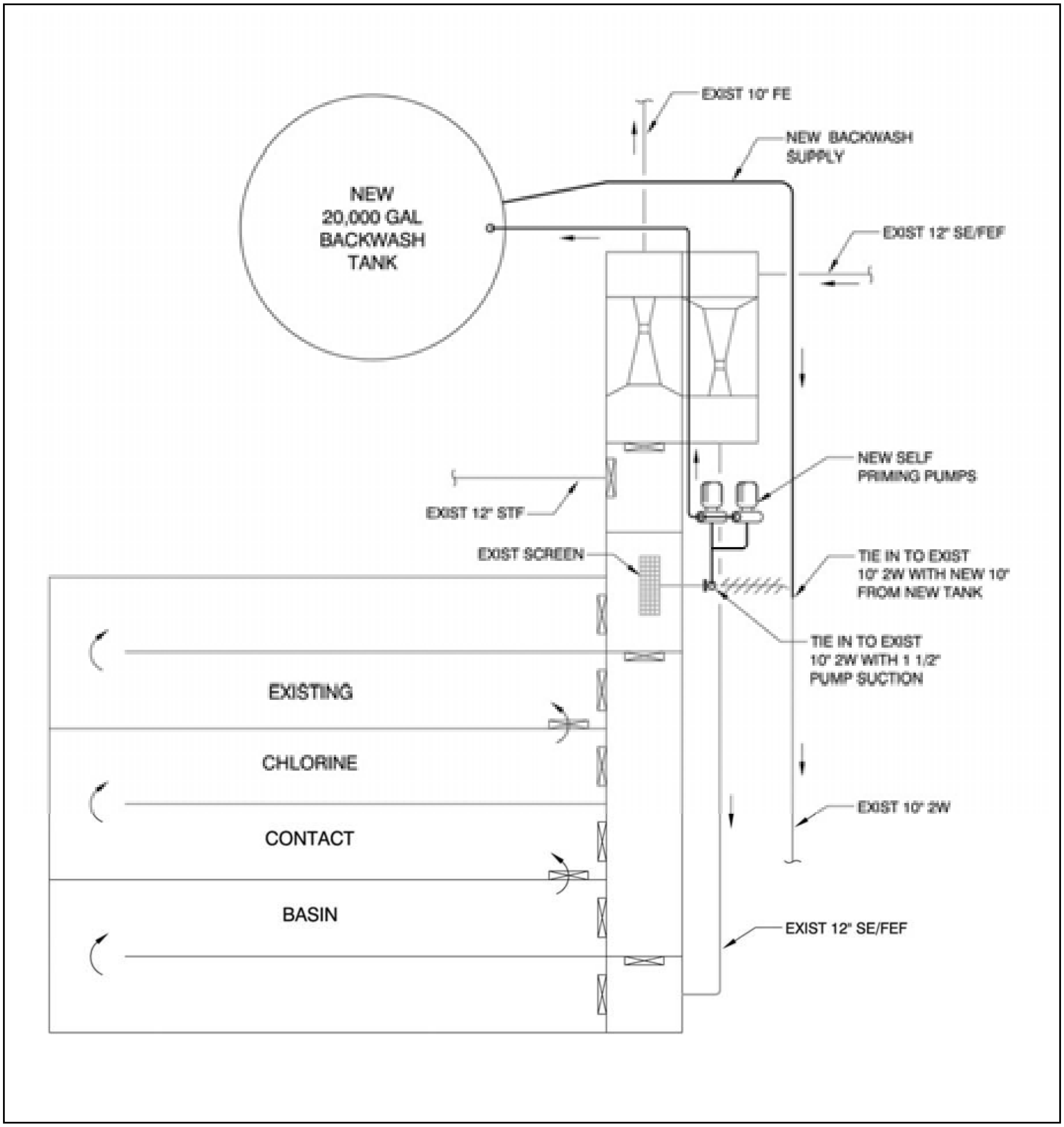


Figure 10-1
Filter Backwash Supply Tank and Pump System

If chemical coagulation and flocculation are ultimately recommended, analyses would have to be developed to determine the best configuration and location of these facilities. It is likely that construction costs could be around $500,000, resulting in a capital cost of about $625,000 (including engineering, etc.).

Table 10-2
Needs for Chemical Coagulation and Flocculation Ahead of the Filters Associated with the Various Biological Treatment Alternatives

Secondary Treatment Alternative	Need for Coagulation and Flocculation	Reasons
MBR	Not Needed	The MBR produces a membrane filtered effluent that is better than the sand filter effluent after a conventional activated sludge system. If the MBR alternative is selected, the sand filters would be abandoned and there would be no need for coagulation and flocculation.
IFAS	Likely Not Needed	The mixed liquor in the IFAS alternative is comparable to that in an activated sludge system. New secondary clarifiers will be constructed for this alternative. Modern design clarifiers have a center flocculation well which helps in floc formation. The secondary effluent from the IFAS alternative will likely be of higher quality than the current condition. Therefore, it is probably not warranted to add a formal coagulation and flocculation step for the IFAS alternative. However, provisions to add these features in the future should be considered.
Submerged Attached Growth	Likely Needed	Effluent suspended solids produced from a submerged attached growth system are sloughings from the fixed film media, which would not be flocculated in the process. These suspended solids may not be adequately removed in the effluent filters.

Section 11

Effluent Disinfection

Section 11

Effluent Disinfection

At the present time, chlorine gas, supplied in 150 lb cylinders, is used for wastewater disinfection. Sulfur dioxide gas, also supplied in 150 lb cylinders, is used for dechlorination. Although the existing dechlorination system is functioning well and the plant routinely meets effluent total coliform limits, there are two major concerns with continued use of chlorine: 1) chlorine safety issues and related Uniform Fire Code Requirements, and 2) disinfection byproducts. Both of these concerns are discussed more fully later in this section. Because of these concerns, and because there are other benefits (described later in this section), disinfection using ultraviolet light (UV disinfection) and disinfection using ozone are considered herein as alternatives to continued use of chlorine.

11.1 Continued Use of Chlorine and Sulfur Dioxide

A description of the existing storage and feed systems for chlorine and sulfur dioxide is included in Section 5. In this section, future capacity requirements are developed and compared to existing system capacities to determine required improvements based on capacity. Additionally, the concerns previously mentioned and improvements recommended to mitigate those concerns are discussed. Capital and annual costs for chlorination and dechlorination are presented in Section 11.4.

11.1.1 Chlorine Feed Capacity Evaluation

Chlorine can be used, not only for effluent disinfection, but also keeping effluent filter media and filter underdrain nozzles clean, for control of undesirable filamentous organisms in the activated sludge process and for influent odor control. To support these potential uses, the existing chlorination system includes three different chlorine ejector and feeder systems. One system is dedicated to effluent disinfection use. A second system serves as a backup for effluent disinfection, but can also be used for influent odor control or filamentous organism control. The third system can be used either for influent odor control or filamentous organism control. There is no separate ejector and feeder for chlorination of influent to the filters. Instead, a portion of the chlorine solution that is used for effluent disinfection can be routed to the filter influent.

Based on information provided by plant operators, there is no need to feed chlorine to the plant influent for odor control and this feature has never been used by the current staff. Also, polymer is used to assist settling in the secondary clarifiers and chlorine is not used for filamentous organism control. Therefore, the potential uses of chlorine for influent odor control and filamentous organism control are not considered necessary and are not addressed further.

Although the plant is currently achieving good disinfection results with typical chlorine doses around 6 mg/L, a significantly higher value should be used for design, especially if a portion of the chlorine used is to be fed ahead of the effluent filters. It is suggested that a reasonably conservative design chlorine feed rate is 123 lb/d, based on a dose of up to 20 mg/L for the design peak week equalized flow of 0.74 Mgal/d. Current capacities of the three existing chlorine ejector and feeder systems are 100, 50, and 50 lb/d, respectively. However, these systems can be easily modified to meet future requirements at relatively minor costs (about $3,000 each) by changing out certain internal parts.

Assuming a room temperature of 70 °F is maintained in the room where the chlorine cylinders are located, the maximum allowable withdrawal rate would be 70 lb/d per cylinder for a total of 420 lb/d, which is substantially greater than needed. In fact, it would be possible to operate with as little as two cylinders on-line and two on standby. However, that would require much more frequent changing of cylinders, so having additional cylinders connected is beneficial.

11.1.2 Chlorine Contact Basin Capacity Evaluation

The existing chlorine contact basin has a volume of about 22,000 gallons. The recommended future volume is 31,000 gallons based on a chlorine contact time of 60 minutes at the equalized peak week flow of 0.74 Mgal/d. Therefore, the chlorine contact basin should be expanded by knocking out the end walls and extending the channels, as provide for in the original design.

11.1.3 Sulfur Dioxide Feed Capacity Evaluation

As a rough rule, approximately 1 mg/L of sulfur dioxide should be fed for each mg/L of chlorine residual at the effluent end of the chlorine contact basin. Although typical chlorine residuals at the end of the chlorine contact basin are only about 2 mg/L, according to plant operators, a design sulfur dioxide dose of 15 mg/L is suggested. At the peak week equalized design flow of 0.74 Mgal/d, a sulfur dioxide feed capacity of 93 lb/d is indicated.

There are two existing sulfur dioxide ejector and feeder systems. One is intended to be a standby system. The feeders are currently configured with capacities of 200 lb/d and 50 lb/d respectively. The smaller system can be upgraded easily to a higher capacity at minor cost (about $3,000).

Assuming a room temperature of 70 °F is maintained in the room where the sulfur dioxide cylinders are located, the maximum allowable withdrawal rate would be 23 lb/d per cylinder for a total of 198 lb/d, which is greater than needed.

11.1.4 Chlorine and Sulfur Dioxide Safety Issues and Uniform Fire Code Requirements

Chlorine is highly toxic and sulfur dioxide is hazardous, so there are significant public health concerns related to potential leaks of these gases. To protect plant staff and the public, the Uniform Fire Code includes various provisions on the storage and use of these substances. In particular, automatic shutoff valves should be provided on all cylinders or a containment and scrubbing system should be provided to neutralize a potential leak. Currently the DSPUD facilities do not include either of these features. There are additional provisions in the Uniform Fire Code relating to ventilation systems, fire sprinklers, alarms, and other features.

Implementation of the Uniform Fire Code is subject to the discretion of local governmental agencies. Therefore, before a final determination of recommended improvements can be developed, the specific conditions at DSPUD would have to be reviewed by and discussed with the local Fire Marshal. For the purposes of this Facilities Plan, it is assumed that automatic emergency shutoff valves will be provided on all chlorine and sulfur dioxide cylinders connected for use. New ventilation features and an exhaust gas scrubber system might be a preferred and perhaps less expensive option; however, that is left for future evaluation and determination in conjunction with the Fire Marshal, if applicable.

11.1.5 DISINFECTION BYPRODUCTS

Chlorine produces disinfection byproducts, such as dichlorobromomethane, that are human carcinogens. Testing of the DSPUD effluent has shown a reasonable potential for exceeding water quality criteria for dichlorobromomethane, so there is an effluent limit in the existing discharge permit for this constituent. At the present time, the permit does not take into account dilution of the effluent in the South Yuba River. However, when the effluent is mixed with the river flow, the long-term average concentration of dichlorobromomethane should be at a safe level. Therefore, if DSPUD is to continue using chlorine, the District should take the steps necessary to obtain dilution credits for its discharge to the South Yuba River. This would require installation of an effluent diffuser across the river channel, installation of a flow metering station on the river near the point of discharge, a mixing zone study, and reopening of the permit to include the dilution credits, if allowed by the Regional Water Quality Control Board.

A possible alternative to using dilution credits for compliance with dichlorobromomethane requirements could be to practice chloramination, which involves feeding some ammonia together with the chlorine. Ammonia will be essentially completely removed in the biological treatment system and it would be impractical to accurately control the biological treatment system to leave in just the right amount of ammonia for chloramination. Therefore, a small amount of ammonia would have to be fed ahead of the disinfection process. Although the practice of chloramination has been successful in mitigating disinfection byproducts in other plants, this practice would have to be tested and proven for effectiveness under the specific conditions at DSPUD. If chloramination was found to be effective in reducing concentrations of dichlorobromomethane at DSPUD, it might still be desirable to obtain dilution credits, if available, as an additional safety measure against violations for this toxicant. For the cost estimates developed later in this Section (see subsection 11.4), it is presumed that dilution credits would be pursued for continued use of chlorine, without chloramination. However, DSPUD should investigate the availability of dilution credits and the likely magnitude of these credits (if available at all) before making any decisions on continuing with chlorine. Additionally, DSPUD may want to investigate the chloramination alternative further, including on-site pilot testing. The reader is referred to Section 17 for further discussion of this issue.

11.2 UV DISINFECTION

UV disinfection has become very prevalent in recent years, mainly due to the safety issues and disinfection byproducts associated with the use of chlorine. In UV disinfection, the wastewater effluent is brought into close proximity with submerged UV lamps under controlled hydraulic conditions. The time of exposure and the intensity of UV radiation being transmitted through the water determine the level of disinfection.

The UV disinfection system for DSPUD would be designed in accordance with the requirements for Disinfected Tertiary Recycled Water set forth in the Water Recyling Criteria in Title 22 of the California Code of Regulations (Title 22) and in accordance with the National Water Research Institute (NWRI) UV Disinfection Guidelines for Drinking Water and Water Reuse.

In accordance with NWRI guidelines, the design UV transmittance and dose for systems involving granular media filtration would be 55 percent and 100 mJ/cm^2, respectively. For the MBR biological treatment option, the corresponding values would be 65 percent and 80 mJ/cm^2, respectively. For this study, open channel UV disinfection systems were considered for both the filtered effluent and the MBR alternatives. For the MBR alternative, a closed vessel UV system was considered also. In all cases, UV system hydraulic capacity would be adequate to handle emergency peak flows substantially higher than 0.74 Mgal/d (perhaps as high as 1.7 Mgal/d), but at reduced UV doses (such high flows are not expected to occur). System configuration data for all three cases are shown in Table 11-1. Capital and annual costs are presented in Section 11.4.

Table 11-1
UV Disinfection System Configuration Data

Alternative	System Configuration	Structures
Filtered Effluent – Open Channel	2 Channels 3 Banks of UV Lamps per Channel (2 duty + 1 Standby) 3 Modules per Bank 6 Lamps per Module Total of 108 Lamps	Channel structure dimensions of 50 ft x 20 ft, including inlet and outlet basins and walkways. 1100 ft^2 air conditioned building to house electrical components.
MBR Effluent – Open Channel	2 Channels (1 duty, 1 standby) 2 Banks of UV Lamps per Channel 3 Modules per Bank 6 Lamps per Module Total of 72 Lamps	Channel structure dimensions of 45 ft x 20 ft, including inlet and outlet basins and walkways. 910 ft^2 air conditioned building to house electrical components.
MBR Effluent – Closed Vessel	1 Train 2 Vessels per Train 40 Lamps per Vessel Total of 80 Lamps (1 duty + 1 standby vessel or all lamps operating at 50 percent power)	No channel structure 400 ft^2 air conditioned building to house electrical components.

An example layout of an open channel UV system that would be similar to the one recommended for filtered effluent disinfection at DSPUD is shown in Figure 11-1. An open channel UV module is shown in Figure 11-2, while a typical closed vessel reactor is shown in Figure 11-3.

If the plant switches to UV disinfection, it would be desirable to eliminate the use of gaseous chlorine altogether, due to the risks and disinfection byproducts issues. Accordingly, not only would chlorine gas use for effluent disinfection be discontinued, but also use for filter media cleaning.

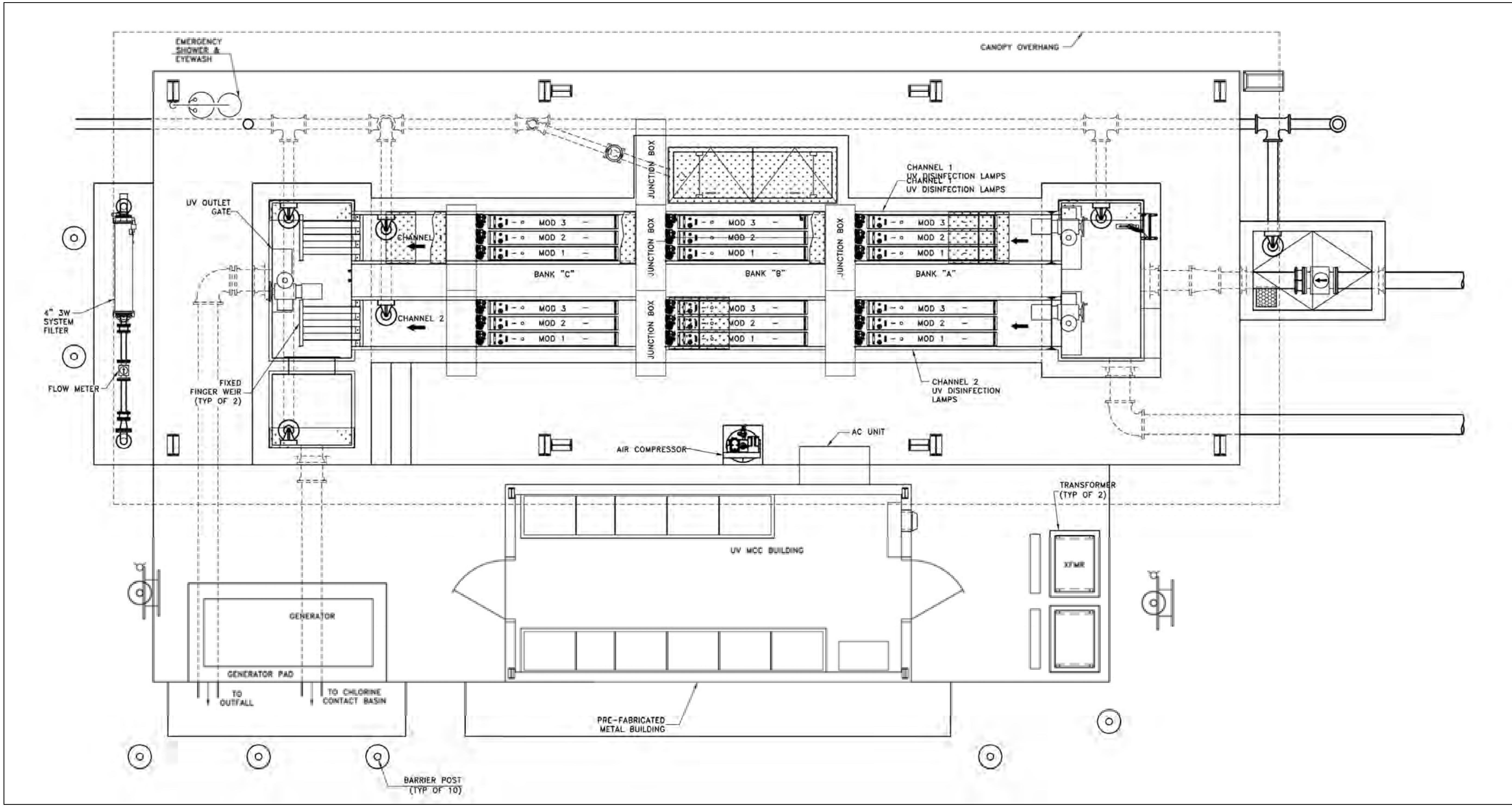


Figure 11-1
Example Layout of an Open Channel UV Disinfection System for Filtered Effluent

At this time it is unclear whether chlorine feeding ahead of the filters is really necessary, or just desirable. Typically, the normal combined air and water backwash should be able to adequately clean the media, even without chlorine. However, if normal backwashing is not adequate and it is found that occasional chemical cleaning of the filters is needed, chemicals other than chlorine could be investigated. If it is still desirable to use chlorine, it is suggested that liquid sodium hypochlorite (bleach) could be used in occasional batch cleaning operations (soaking) with the filter cell in question out of service. When cleaning is complete, the bleach remaining in the filter could be neutralized to remove all chlorine residual and then the spent cleaning solution could be recycled through the plant.

Figure 11-2
Open Channel UV Module Being Removed,
Courtesy of Wedeco, ITT Water and Wastewater

Figure 11-3
Closed Vessel UV Reactor,
Courtesy of Wedeco, ITT Water and Wastewater

11.3 OZONE DISINFECTION

Ozonation is a widely used disinfection process in potable water treatment but has been used rarely in municipal wastewater treatment. This is because of its higher capital costs, and higher operations and maintenance (O&M) costs compared to chlorination and UV in wastewater applications. However, use of ozonation as an effluent disinfectant is increasing because of its secondary benefits such as advanced oxidation of refractory organics (e.g., endocrine disrupting compounds [EDCs]) and effluent aeration. Improvements in ozone generation and injection efficiency are also reducing ozonation costs. Accordingly, it is appropriate to evaluate ozonation as a disinfection alternative for DSPUD.

As was discussed for UV disinfection, if use of chlorine gas for disinfection is discontinued in favor of ozone, all uses of chlorine gas, even for filter media cleaning, would also be discontinued. Other methods of cleaning the filter media would be used, if needed.

Presented in the following paragraphs are general descriptions of the ozonation process and its benefits, system components, design criteria, and system layout. Capital and annual costs are presented in Section 11.4.

11.3.1 GENERAL DESCRIPTION AND BENEFITS OF OZONATION

Ozone exists as a colorless gas at room temperature. It is highly unstable; and therefore has to be generated on-site using an oxygen source such as air or liquid oxygen (LOX). Ozone reactions in wastewater are classified into two groups: 1) direct reactions, and 2) indirect reactions. Direct reactions are between any chemical species and molecular ozone, which has an oxidizing potential of 2.07 V. By way of comparison, hypochlorous acid (the active chemical in chlorination) has an oxidizing potential of only 1.49 V. Indirect reactions are those between any chemical species and the hydroxyl radicals (OH•) formed from the decomposition of ozone initiated either by high pH (hydroxyl ions OH^-) or by the presence of hydrogen peroxide. The oxidizing potential of the OH• radicals is very high, 2.80 V. Ozone disinfects water by essentially oxidizing the pathogens, or parts of them such that the pathogens cannot reproduce and cause disease. Because ozone and its radicals have such high oxidizing potentials, ozonation has difficulty disinfecting the interior of large particulates, as the ozone is consumed before penetrating to the center. The consequences of this aspect of ozonation are that the level of effluent particle removal has a significant effect on the disinfection efficacy of ozone, particularly with regard to bacteria (in comparison to virus and protozoa). As will be discussed, ozonation of membrane filtered effluent (0.2 NTU turbidity) can be easily achieved. Ozonation of sand filtered effluent (2.0 NTU turbidity) requires special consideration. Another concern with ozonation (like chlorination) is disinfection byproducts. This is also discussed below.

For all disinfection systems that are not based on chlorine, Title 22 Tertiary Recycled Water disinfection criteria require 5 log removal of virus, and 7-day median effluent total coliform concentrations not exceeding 2.2 MPN/100 ml. Ozone is very effective at inactivating virus in comparison to its inactivation of coliform. Because of this, the cost and effectiveness of ozonation systems for effluent disinfection is dependent on the type of upstream effluent filtration system and several water quality parameters. The California Department of Public

Health (CDPH) has conditionally approved the HiPOx^{TM} Disinfection Technology (manufactured by APTwater, Inc.) at a CT (concentration, mg/L x contact time, min) of 1.0 mg-min/L for wastewater disinfection applications to meet the minimum coliform and virus disinfection criteria found in Title 22 for recycled waters that have received treatment through an accepted filtration process. However, recent pilot testing at wastewater treatment plants has shown that membrane filtration, as opposed to granular media or cloth disk filtration, may be needed to meet a total coliform limit of 2.2 MPN/100 ml at a CT of 1.0 mg-min/L. Also, in order to fully realize the secondary benefits of refractory organics removal (discussed below), higher doses and longer contact times than those required by CDPH for disinfection are needed. Thus, ozonation CT values greater than 1.0 mg-min/L are needed when membrane filtration is not used and/or removal of refractory organics is desired.

When ozonation is installed downstream of a membrane filtration system, most of the larger particles and associated coliforms are removed by the membrane filtration step. As a result, ozone dose and contact time required for inactivating virus and any residual coliform present in membrane filtered effluent are very low.

On the other hand, effluents from non-membrane filtration systems (e.g., sand filters or disk filters) contain larger particulates containing coliform, which makes it difficult to achieve levels of coliform disinfection required by Title 22 when using ozone. In order to achieve reliable Title22 coliform disinfection using ozone with sand filtered effluent, it is recommended that the ozonation system be followed by a small UV system or chlorination (and dechlorination, when needed). Since ozone is being considered as an alternative to chlorine and a method for mitigating disinfection byproducts related to chlorine, only ozone coupled with UV is considered herein.

A post-ozone UV disinfection system would be very small, probably a small in-line unit as used in some water treatment applications. This is because the UV transmittance (UVT) of ozonated effluent is high as a result of ozonation cleaving UV absorbing aromatic organic compounds into short-chain organic compounds. UVT is the critical process design parameter utilized in sizing UV systems. A moderate increase in effluent UVT caused by ozonation results in substantial reductions in UV system size and power requirements.

In addition to pathogen inactivation, ozonation improves various critical aspects of water quality including:

1. Effectively removing Endocrine Disrupting Compounds (EDCs), Pharmaceuticals and Personal Care Products (PPCPs), and trace organics. This reduces the estrogenic activity of effluent. EDCs and PPCPs are currently being investigated by various federal agencies, including the Environmental Protection Agency, Geological Survey, and Fish and Wildlife Service, for their potential effects on aquatic organisms (e.g., feminization of male fish).
2. Improving the biodegradability of refractory organics in treated effluent.
3. Increasing effluent dissolved oxygen concentration. Release of molecular oxygen is a natural byproduct of ozonation reactions. Ozonated effluent typically has DO concentrations close to saturation, which in some effluent discharge situations is necessary

to maintain adequate DO levels in the surface water receiving the effluent. Use of ozonation in the main wastewater treatment process train could eliminate the need for a separate reaeration process, thereby reducing capital and power costs.

4. Eliminating colorants and odor causing compounds present in effluent.

Formation of inorganic and organic byproducts is a critical concern with the ozonation process. Bromate is an ozonation byproduct of special concern because it has a drinking water Maximum Contaminant Level (MCL) of 10 μg/L, which may be lowered to 5 μg/L. Factors affecting bromate formation includes ozone dosage, presence of ammonia, and background bromide levels. Literature case studies have shown that formation of bromate is of less concern when influent bromide concentrations are less than 50 μg/L. Preliminary characterization of the District's filtered effluent samples showed bromide concentrations of 26 and 44 μg/L.

Other ozonation byproducts include short-chain aldehydes (such as formaldehyde and ethyl glyoxal), and nitrosoamines (such as N-Nitroso dimethylamine [NDMA]). Concentrations of these byproducts are found to be minimal at the ozone doses typically utilized for effluent disinfection.

11.3.2 System Components

General system descriptions of ozonation and UV installed downstream of ozonation are summarized below.

Ozonation System

Components of an Ozonation system include: 1) feed gas storage and preparation, 2) ozone generation, 3) ozone injection and mixing, 4) ozone contact, and 5) off-gas destruction.

Feed Gas Storage and Preparation

The feed gas preparation ensures that a clean, dry source of oxygen is fed to the ozone generator. High purity oxygen can be produced from liquid oxygen (LOX) or can be generated on-site using a cryogenic process. LOX feed systems are much simpler and found to be suitable for facilities that are similar to DSPUD's treatment facility. LOX feed systems consist of LOX storage tanks, LOX evaporators, gas filters, and gas pressure regulators.

Ozone Generation

Conversion of oxygen to ozone occurs with the use of electrical energy. Pure oxygen is passed through a high voltage electric discharge, i.e., corona discharge, which produces ozone up to 10-14 percent by weight. A substantial percentage of the electrical energy consumed during ozone generation is lost as heat. Higher temperature adversely impacts the production of ozone. Thus, adequate cooling should be provided to maintain the efficiency of the generator. Excess heat is removed by circulating water around the stainless steel shell of the ozone generators.

Ozone Injection and Mixing

Recent developments in the field of ozone transfer technologies have resulted in the use of highly efficient venturi injectors and static mixers over conventional bubble diffusers. A high efficiency injector utilizes a partial vacuum that pulls the ozone from the gaseous phase to the water stream. The ozone is typically injected into a side-stream, which is then injected into the main plant flow, normally via a low head loss static mixer.

Ozone Contact

Ozone contactor sizing is governed by ozone dosage required for disinfection and contaminant oxidation. CDPH allows the usage of pipeline contactors or baffled tank contactors for providing required minimum contact time for disinfection. An enclosed tank contactor is recommended for this project as it has been used in several ozone applications. The contact tank is covered to contain the off-gas.

Off-gas Destruction System

The off-gas from the ozone contactor must be contained and destructed because the concentration of ozone is usually much higher than the current OSHA maximum permissible limit of 0.1 mg/L (by volume) for an 8-hour shift. Ozone is readily destructed and converted back to oxygen at a high temperature in the presence of a catalyst.

Ozone Safety

Ozone is a toxic gas and the ozone generation and injection facilities should be designed to control it. Ambient ozone levels in the ozone building should be monitored continuously. The signal from an ambient ozone monitor will be used to alarm or shut down the ozone system. All rooms should be properly ventilated, heated and cooled to match the equipment-operating environment.

UV System Installed Downstream of Ozonation System

The UV system that would be provided downstream from ozonation, except when membrane filtration is used, would be a closed vessel system, similar to that previously described for closed vessel UV disinfection after MBR biological treatment. However, the required UV dose and number of lamps would be lower.

11.3.3 DESIGN CRITERIA

The design criteria for a DSPUD ozonation system are listed in Table 11-2. Criteria for the downstream UV system needed when granular media filtration is used are shown in Table 11-3.

Table 11-2
Ozonation System Design Criteria

Item	Unit	Value
Oxygen Source for Ozone Generation	-	Liquid Oxygen
Ozonation System Configuration	-	Side-stream Injection
Design Flow	Mgal/d	0.74
Average Influent Water Quality Parameters		
Nitrite	mg/L	<0.1
Turbidity	NTU	<2
TSS	Mg/L	<1
Alkalinity	mg/L	100
Bromide	µg/L	<35
Minimum SRT	days	15
Maximum Temperature	°C	20
Ozone Generator Size	lb/d	35
Ozone Feed-gas Composition	wt. %	10
Ozone Dosage	mg/L	5
Ozonation Contact Time	min	20
Ozonation System Transfer Efficiency	%	>93
Line pressure Upstream of Ozonation	psi	<2

Table 11-3
UV System Installed Downstream of Ozonation System Design Criteria

Item	Unit	Value
UV System Type	-	In-pipe
Design Flow	Mgal/d	0.74
Design UVT	%	80
Design UV Dose	mJ/cm2	50
Lamp Type	-	Low Pressure

11.3.4 PRELIMINARY OZONE SYSTEM LAYOUT

A preliminary ozone system layout (not including a downstream UV reactor) is shown in Figure 11-4.

11.4 DISINFECTION ALTERNATIVE COST ANALYSIS

Shown in Table 11-4 are capital, annual operation and maintenance, present worth of annual operation and maintenance, and total present worth costs for the various disinfection alternatives. As indicated in the table, the least cost disinfection alternative is to continue using chlorine gas, regardless of which biological treatment alternative is selected. The cost advantage of chlorine occurs even after accounting for the costs associated with obtaining dilution credits for disinfection byproducts (includes outfall diffuser, river gauging station, mixing zone study, anti-degradation analysis and permit modification). However, because of the concerns regarding chlorine safety and the potential that dilution credits might not be allowed, both the chlorine and UV alternatives are considered further in the overall project alternative analysis presented in Section 17.

Ozonation is considered to be cost-prohibitive at this time. For a granular media filtered effluent, the total present worth cost of ozonation (including the required UV add-on) would be nearly $4 million more than chlorination. Even with MBR biological treatment, when the add-on UV system would not be needed, ozonation would cost almost $2.5 million (total present worth) more than continuing with chlorination. The reason why ozonation was considered in this investigation is that ozone can significantly remove emerging contaminants of concern, such as pharmaceuticals and personal care products, which are not yet regulated, but for which future regulations are anticipated. DSPUD should wait to see how such regulations might evolve. Hopefully, if new regulations are adopted, the cost of ozonation or alternative technologies for meeting the requirements will be reduced by that time.

The reader is referred to Section 17 for further discussion regarding selection of a disinfection process.

Table 11-4
Disinfection Alternative Cost Analysis

Item	Cost for Indicated Alternative (a), $					
	Chlorine	UV-Filt	UV-MBR-OC	UV-MBR-CV	Ozone/UV-Filt	Ozone-MBR
Capital Cost						
Modify Existing Gas Feed Systems	12,000	0	0	0	0	0
Automatic Emergency Shutoff Valves and Controls	200,000	0	0	0	0	0
Expand Chlorine Contact Basin	60,000	0	0	0	0	0
Install River Diffuser	150,000	0	0	0	0	0
Install River Gaging Station	100,000	0	0	0	0	0
New Basins / System Piping / Ancillary Mechanical	0	170,000	150,000	50,000	310,000	260,000
Building Enclosures		290,000	240,000	110,000	400,000	320,000
UV Equipment, Installed	0	590,000	460,000	540,000	350,000	0
Ozone Equipment, Installed	0	0	0	0	800,000	800,000
Subtotal 1	522,000	1,050,000	850,000	700,000	1,860,000	1,380,000
Elect/Instrum, 25% of Subtotal 1, Unless Noted Otherwise (b)	50,000	263,000	213,000	175,000	465,000	345,000
Sitework, 5% of Subtotal 1 Unless Noted Otherwise	Included	42,000	34,000	28,000	74,000	55,000
Site Piping, 10% of Subtotal 1, Unless Noted Otherwise	Included	105,000	85,000	70,000	186,000	138,000
Subtotal 2	572,000	1,460,000	1,182,000	973,000	2,585,000	1,918,000
Contingencies, 20%	114,000	292,000	236,000	195,000	517,000	384,000
Subtotal 3	686,000	1,752,000	1,418,000	1,168,000	3,102,000	2,302,000
General Conditions, Overhead and Profit, 20%	137,000	350,000	284,000	234,000	620,000	460,000
Total Construction Cost	823,000	2,102,000	1,702,000	1,402,000	3,722,000	2,762,000
Engineering and Administration, 25%	206,000	526,000	426,000	351,000	931,000	691,000
Special Studies, Permitting (c)	170,000	0	0	0	0	0
Total Capital Cost	1,199,000	2,628,000	2,128,000	1,753,000	4,653,000	3,453,000
Annual Costs						
Labor	8,400	9,740	8,940	8,140	18,720	12,480
Power	1,000	17,000	17,000	19,000	20,000	10,000
Chemicals	8,000	0	0	0	4,000	4,000
Maintenance Materials	3,000	9,000	9,000	10,000	13,000	8,000
Total Annual Cost	20,400	35,740	34,940	37,140	55,720	34,480
Present Worth Costs						
Present Worth of Annual Costs (d)	304,000	532,000	520,000	553,000	829,000	513,000
Total Present Worth Cost	1,503,000	3,160,000	2,648,000	2,306,000	5,482,000	3,966,000

(a) First quarter 2010 cost level, ENR 20-Cities CCI = 8700.
(b) Chlorine alternative electrical and instrumentation cost not based on 25% of Subtotal 1.
(c) For dilution credits, need mixing zone study, anti-degredation analysis and NPDES permit revision.
(d) 20 years at inflation-adjusted discount rate of 3%, Present Worth Factor = 14.88.

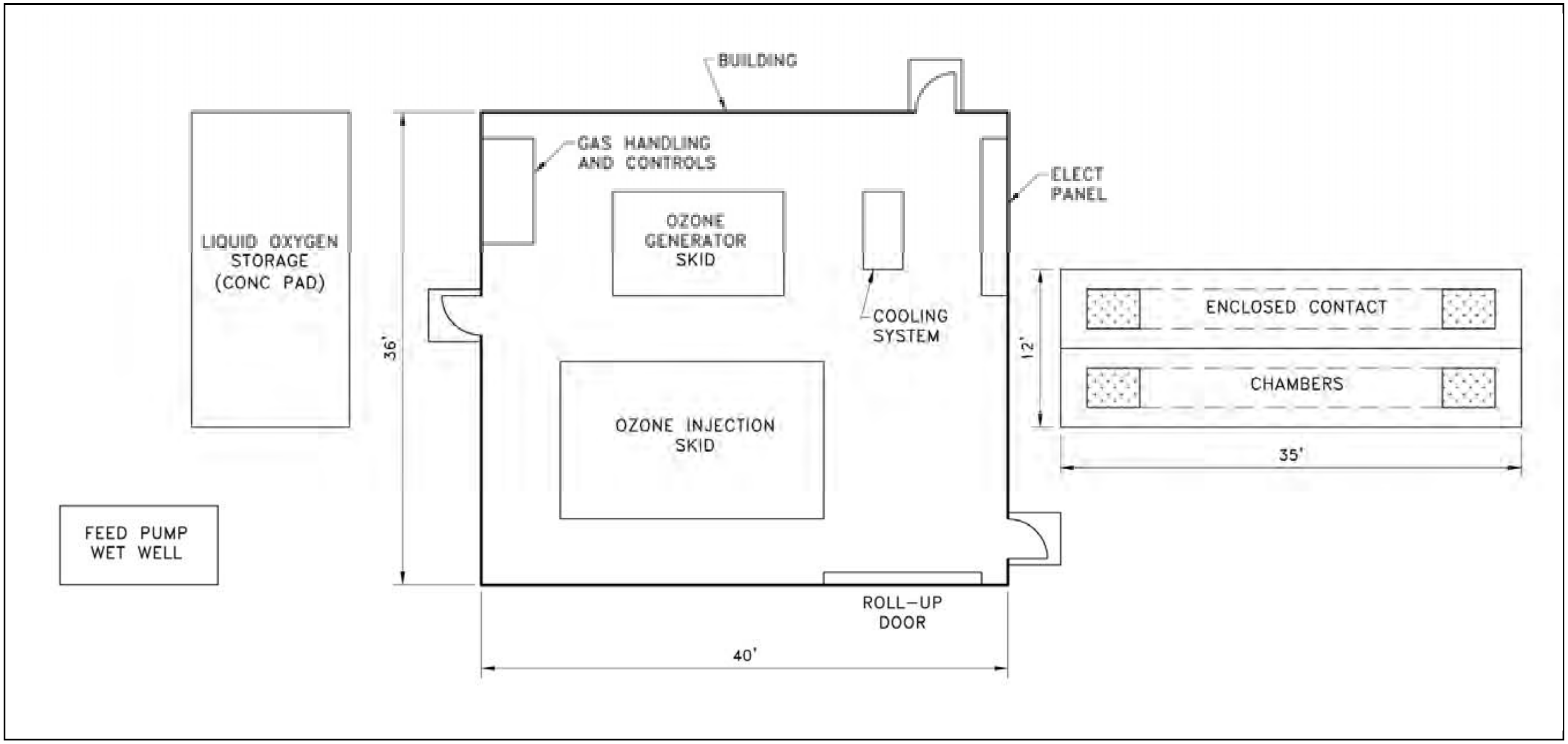


Figure 11-4
Ozone System Layout

Section 12

Emergency Storage and Irrigation Storage

Section 12

Emergency Storage and Irrigation Storage

Currently, there is a 1.56 Mgal open-top steel tank at the DSPUD wastewater treatment plant that is used for two purposes: emergency storage and irrigation operational storage. The emergency storage function is used primarily during the wet season to store any effluent that might not meet the standards for river discharge. Once stored, the noncompliant effluent can be returned through the wastewater treatment plant for retreatment and subsequent discharge. In the dry season, when the effluent is used for irrigation, effluent is stored in the tank between operational cycles of the irrigation system. Additionally, at the beginning of the dry season, the tank can be filled to allow cessation of river discharge several days before the beginning of irrigation operations.

The original design of the tank was based on storing winter peak flows from the wastewater treatment plant for three consecutive days. This was not based on any rules or regulations, but on engineering judgment. The 1.56 Mgal volume was more than enough for irrigation operational storage. Since the original design flow for irrigation disposal was 0.19 Mgal/d, the tank allowed storage for more than eight days between irrigation system operations, if desired.

For the proposed project, the design average day maximum weekly flow is 0.74 Mgal/d and design average flows during July, August and September, when irrigation disposal would normally be practiced, are 0.24, 0.20, and 0.17 Mgal/d, respectively. Therefore, the existing tank would provide for approximately two days of emergency storage with winter peak flows and at least six days of storage during the irrigation season. These capabilities are adequate, so there is no reason to expand the tank at this time. Further, it is noted that the proposed biostimulation storage reservoir discussed in Section 13 can be used for additional emergency and irrigation operational storage, except during the late spring, when its capacity would be needed for its main purpose: to allow curtailment of river discharge and thereby prevent the effluent from causing or contributing to biostimulation in the South Yuba River at a time when irrigation disposal is not possible.

Section 13

Effluent Storage to Mitigate Biostimulation in the South Yuba River

Section 13

Effluent Storage to Mitigate Biostimulation in the South Yuba River

This section includes an investigation of effluent storage facilities to allow curtailment of river discharge during times in the spring when nuisance algae growth could occur in the South Yuba River, but wet soil conditions preclude the initiation of effluent disposal/reuse by irrigation.

13.1 BACKGROUND

In June 2008, there were nuisance growths of filamentous algae attached to submerged surfaces in the South Yuba River downstream from the point of the Donner Summit Public Utility District (District) discharge. These growths were the subject of a citizen complaint to the California Regional Water Quality Control Board, Central Valley Region (Regional Board) and were documented in photographs provided with the complaint showing the vibrant green algae, dated June 19, 2008. The citizen complaint to the Regional Board included statements that the algae growths in 2008 were “highly unusual” and “I have witnessed many seasons and situations up here, but nothing like this”. As a result of the citizen complaint, field surveys to investigate the algae growths were conducted by the Regional Board on June 30, 2008 and by ECO:LOGIC Engineering on behalf of the District on July 2, 2008.

The ECO:LOGIC survey on July 2, 2008 began at Lake Van Norden, approximately two miles upstream from the District discharge, and extended to the Kingvale bridge, approximately three miles downstream from the District discharge. At that time, the spring snowmelt was essentially complete and South Yuba River flows had subsided substantially below those in mid-June. At the time of the survey, evidence of extensive filamentous algae growths was present from the District discharge point to about one mile downstream, but the growths had almost completely died and turned brown, even though the District discharge had been continuous from the previous fall to that date. The July 2 survey was documented in a report with extensive photographs, dated July 11, 2008, to the District and copied to the Regional Board.

As a result of the 2008 algae growths, there is now heightened concern regarding the potential for the District discharge to cause or contribute to nuisance biostimulation in the South Yuba River, and the Regional Board included in the District NPDES permit, adopted in April 2009, a requirement for the District to study the biostimulation issue.

On four occasions from May 31 to June 26, 2009, the District conducted extensive visual and photographic surveys of the South Yuba River from Lake Van Norden to Cisco Grove, a reach of river extending from two miles upstream to nine miles downstream from the District discharge, specifically to investigate and document the extent, if any, of attached filamentous algae. A key finding of the 2009 survey was that filamentous algae were essentially absent immediately downstream from the point of the District discharge, a location that had heavy growths in June 2008. There were, however, significant growths both upstream near Lake Van Norden and several miles downstream from the District discharge, neither of which could be attributed to the District discharge.

Based on a workplan submitted to the Regional Board on July 24, 2009, the District plans to continue studying the biostimulation issue through the spring and early summer 2010.

13.2 THE NEED FOR AND SIZING OF EFFLUENT STORAGE TO MITIGATE BIOSTIMULATION

Although the algae growths that occurred in 2008 are believed to be highly unusual and the District discharge is not known to have caused or contributed to nuisance biostimulation in the South Yuba River in prior years, it is apparent that curtailment of the District discharge to the river during times that nuisance algae growth could occur may be beneficial to the District. Storage could prevent a repeat of the 2008 biostimulation immediately downstream from the District discharge. However, a more general benefit is that, during times of storage, the District effluent would not be implicated as a potential cause or contributing factor to algae growths in the river anywhere downstream.

From the information available to date, it is apparent that the time of risk for biostimulation is near the end of spring when South Yuba River flows are subsiding as snowmelt is nearing completion. Earlier in the year, it is believed that the combination of conditions of limited solar exposure, low ambient temperature, and/or high river flows is not conducive to nuisance algae growth, with or without the effluent in the river. At the time of peak snowmelt and highest river flows, the river flow velocity and resultant scouring of the river bottom is probably the primary deterrent to nuisance algae growth. Soon after the completion of snowmelt and subsidence of river flows, soils in the Donner Summit area become dry enough to allow effluent disposal by irrigation so that storage is no longer needed. Later in the year, effluent disposal by irrigation is continued until climatic conditions preclude irrigation. It is believed that these same climatic conditions also prevent nuisance biostimulation in the river after irrigation is ceased and river discharge is restarted. There is no known evidence of nuisance biostimulation in the late fall after the re-initiation of the District discharge to the River. Even in 2008, when nuisance biostimulation occurred in the spring, it did not occur in the fall.

From the discussion above, it is believed that South Yuba River flows can be used as a good indicator of when storage is needed in the spring. It is suggested that storage should be initiated after peak river flows due to snowmelt have passed and the river flow has subsided to an appropriate “trigger” value that perhaps signifies the onset of river flow velocities that are conducive to attached algae growth. It is believed also that the end of storage and the beginning of irrigation disposal can be correlated to river flow, since the conditions that result in low river flows also result in lands dry enough for irrigation. Neither the beginning nor the ending of storage should be tied to calendar dates, because algae growth potential and the conditions that allow irrigation disposal depend on climatic conditions that vary from year to year, as do river flows. Finally, it is suggested that the South Yuba River Flow at Cisco can be used as the indicator of conditions affecting algae growth and irrigation disposal at Donner Summit.

South Yuba River flows at Cisco for the period from January 2002 through July 2009 are shown in Figures 13-1 and 13-2. In Figure 13-2, only flows up to 500 cubic feet per second (cfs) are shown to allow more detailed evaluation of the critical low flow periods. Key observations from Figure 13-2 are: 1) the major declines in river flows at the end of the spring snowmelt for the period of 2002 through 2009 occurred in June and/or July, and 2) the rates of decline in all of the years was nearly the same. In all years, the decline from 300 cfs to 50 cfs, for example, lasted two to three weeks. Minimum flows less than 10 cfs were always reached before the end of July.

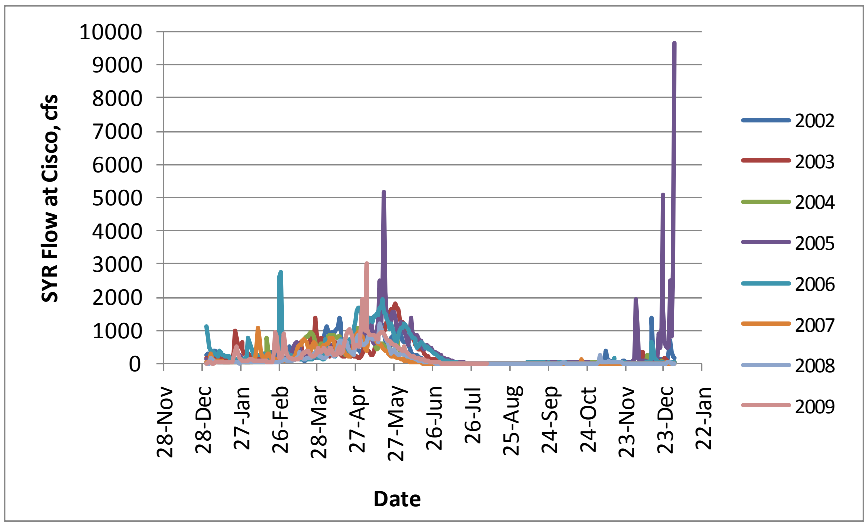


Figure 13-1
South Yuba River Flows at Cisco

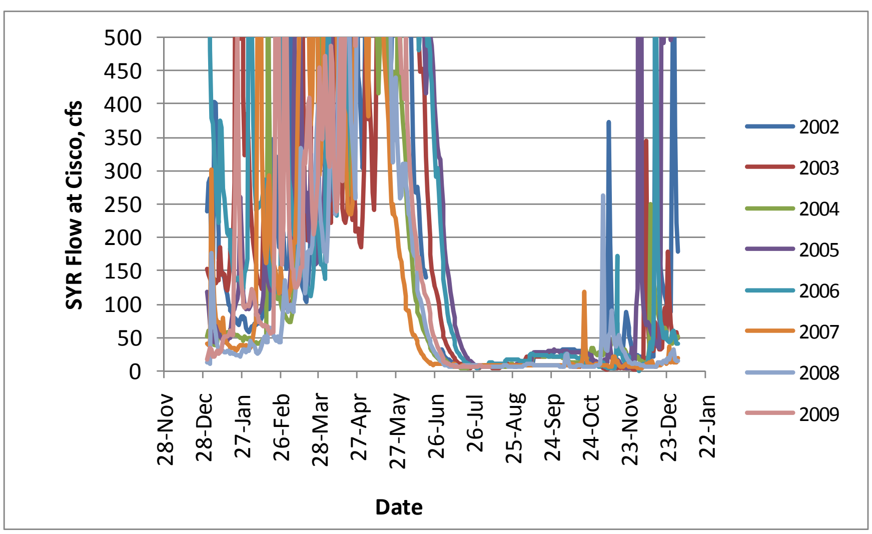


Figure 13-2
South Yuba River Flows at Cisco, Data Below 500 cfs

On June 19, 2008, the date that nuisance algae growths were documented in photographs provided with the citizen complaint to the Regional Board, the South Yuba River Flow at Cisco was 65 cfs. The date upon which nuisance algae growths first occurred in 2008 is not known, but it is estimated that this might have been two or three weeks before the June 19 photographs. The flows in the South Yuba River at Cisco two and three weeks before June 19, 2008, were 267 and 303 cfs, respectively. The trailing 7-day average flows on June 19 and two and three weeks before that date were 96, 285 and 360 cfs, respectively. On the basis of these flows, it is suggested that a reasonable trigger for discontinuing discharge to the river might be a 7-day average South Yuba River flow of 300 or 400 cfs at Cisco.

In Figure 13-3, South Yuba River flows at Cisco are shown as a function of the number of days before spray irrigation disposal was initiated in the years 2002 through 2008. Under the District NPDES permit, river discharge must be stopped and irrigation started when conditions allow, but no later than August 1. Actual dates of stopping river discharge and starting irrigation are shown in Table 13-1. During time lags between stopping river discharge and starting irrigation, the effluent was being stored at the WWTP. Also shown in Table 13-1 is the date on which the 7-day average South Yuba River flow at Cisco fell below 20 cfs and the number of days after this date on which irrigation was started. It is believed that the 7-day average flow of 20 cfs can be used as a reasonable indicator of when spray irrigation can start and storage for biostimulation mitigation can stop. In 2003 and 2006, irrigation was actually started about two weeks before this flow trigger occurred. In 2004 and 2007, however, irrigation was not started until about one month after this flow trigger. It is suggested that an earlier start in accordance with the proposed flow trigger might have been possible had that been an objective at the time. However, dates later than this flow trigger are evaluated herein.

Volumes of wastewater effluent that would have been stored in 2002 through 2008 based on starting and stopping storage when 7-day average South Yuba River at Cisco flows fell below 300 cfs and 20 cfs, respectively are shown in Figure 13-4. As indicated in the figure, the storage requirement would have ranged from 3.7 million gallons (Mgal) in 2005 to 5.8 Mgal in 2007. Even though river flows less than 300 cfs frequently occurred before May (see Figure 13-2), storage prior to May was not considered because this would have been before the peak snowmelt-related flows in the South Yuba River. Even when beginning the analysis in May, there is a "false start" shown in Figure 13-4 for the year 2003 when flows early in May were below 300 cfs and then increased substantially, not falling below 300 cfs again until near the end of June. In a real-time situation, the false start would probably have been avoided, knowing that there was substantial snow still on the ground and that the peak spring runoff flows had not yet occurred. In that case, the maximum accumulated storage in 2003 would have been 4.5 Mgal, instead of the 5.7 Mgal shown in Figure 13-4.

In Figure 13-5, a more conservative analysis is shown, based on starting storage when the 7-day average South Yuba River at Cisco flow fell below 400 cfs and stopping storage 14 days after the 7-day average flow fell below 20 cfs. As shown in the figure, the storage volume was still increasing as of July 31 in 2005 and 2006. However, the analysis was terminated on that date because it is presumed that August 1 would be the absolute latest that full irrigation disposal on land would be started. Neglecting the false start in 2003, the maximum storage requirement would have been 9.4 Mgal in 2007, followed closely by 8.9 Mgal in 2006 and 8.2 Mgal in 2008.

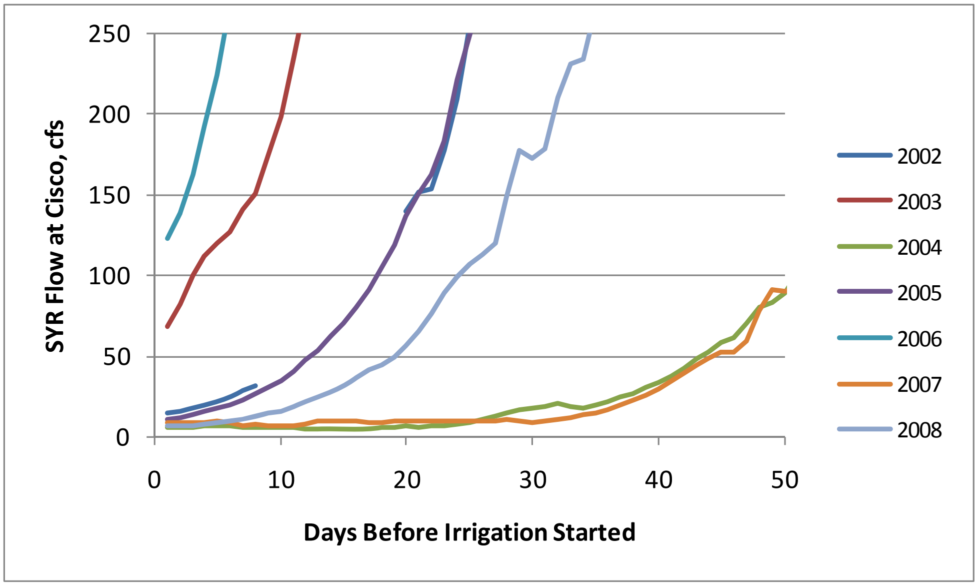


Figure 13-3
South Yuba River Flows Before the Beginning of Spray Irrigation

Table 13-1
Dates of Terminating River Discharge and Starting Irrigation and Related South Yuba River Flows

Year	Last Date of River Discharge	First Date of Irrigation	Date on Which 7-Day Average SYR Flow at Cisco Fell Below 20 cfs	Days from 7-Day Avg. SYR Flow <20 cfs to Date of First Irrigation
2002	July 5	July 9	July 9	0
2003	June 30	July 2	July 14	-12
2004	July 24	August 1	July 3	29
2005	July 23	July 27	July 25	2
2006	June 30	July 5	July 20	-15
2007	July 18	July 25	June 22	33
2008	July 2	July 9	July 2	7
Average	---	---	---	6.3

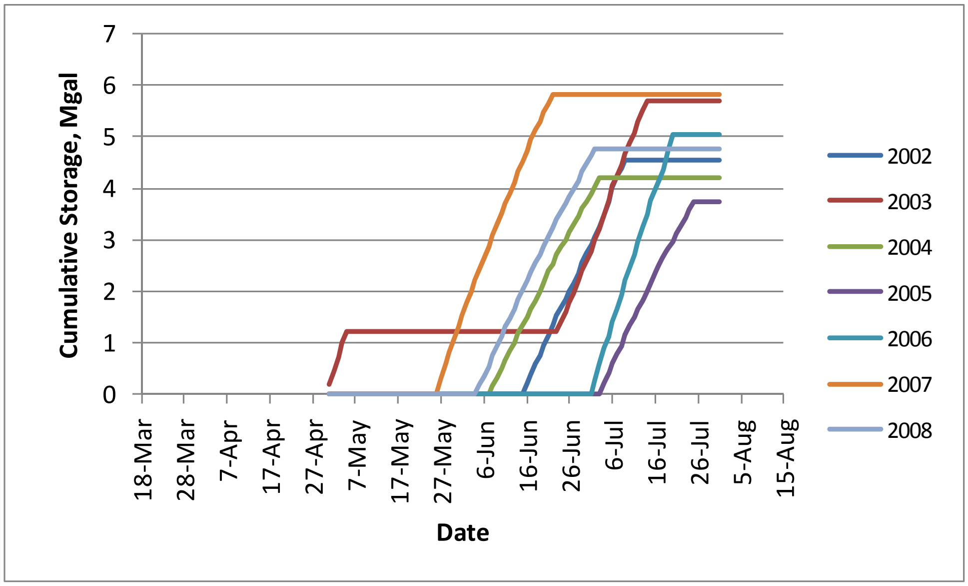


Figure 13-4
Effluent Storage Requirements Based on Starting and Stopping Storage Based on 7-Day Average South Yuba River Flows of 300 and 20 cfs, Respectively at Cisco

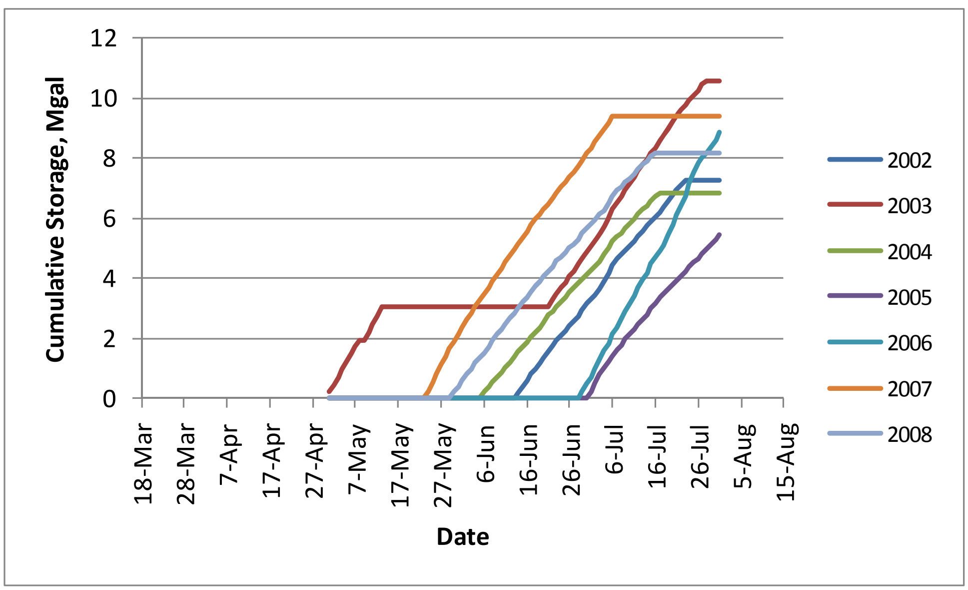


Figure 13-5
Effluent Storage Requirements Based on Starting Storage at 7-Day Average South Yuba River Flow of 400 cfs at Cisco and Stopping Storage 14 Days After the 7-Day Average Flow at Cisco Fell Below 20 cfs

Based on the foregoing analysis, the amount of effluent storage desired to mitigate possible biostimulation in the South Yuba River could be in the range of about 6 to 9 Mgal, based on effluent flows occurring in 2002 through 2007 (the highest storage requirements occurred in 2007). To extrapolate these storage requirements to the future design condition with additional growth in both the District and SLCWD, it is appropriate to multiply the storage volumes by the ratio of future average annual flows to current average annual flows, based on the data in Section 4 (the ratio is 0.28/0.23 = 1.2). Accordingly, reasonable biostimulation storage allowances for the future design condition could be in the range of about 7 to 11 Mgal.

The District may be able to satisfy a need for biostimulation storage with either an earthen reservoir, or a tank, or multiple tanks. The remainder of this section discusses options for both earthen reservoirs and tanks, providing a comparative analysis of the suitability and desirability of one storage facility option with another.

13.3 BIOSTIMULATION STORAGE ALTERNATIVE ANALYSIS

Six potential sites for earthen reservoirs and three potential sites for storage tanks to satisfy biostimulation storage requirements were identified and evaluated as set forth in this section. The criteria used to evaluate the various sites and the choice between earthen reservoirs and storage tanks are discussed below, followed by identification descriptions and evaluations of the sites.

13.3.1 STORAGE SITE EVALUATION CRITERIA

Each of the potential storage sites were evaluated for suitability for effluent storage based on the factors described below. Three primary criteria were identified to evaluate, compare, and select the proposed storage site, including distance from the District WWTP, engineering factors, and environmental factors.

Distance from the District WWTP

The distance of effluent storage from the District WWTP is a critical factor to avoid impacts related to pipeline and pump station construction. To a lesser degree, distance to existing and potential future irrigation disposal fields is also a consideration. Operational costs relative to effluent pumping and pipeline capital costs are also important reasons to minimize the distance between storage and the WWTP. There are significant capital cost savings for sites closer to the WWTP and, unless other more significant environmental or engineering constraints exist, location/distance from the District WWTP is considered a primary factor in determining a recommended location. Of the six reservoir sites and three tank sites considered, two reservoir sites (Outfall Site, Franz Site) and all tank sites are located less than 1 mile from the WWTP. These sites are considered most promising as storage locations.

Engineering Factors

Engineering factors affect the construction and overall layout of the reservoirs and include site access, topography, and geotechnical and soil conditions.

Site Access. During construction, access to the site with large earthmoving equipment and trucks will be necessary. Roadways used to access the site must be able to accommodate the additional traffic and be suitable for the size and weight of the equipment to avoid damaging the roadway and creating safety hazards. Long-term site access will also be important to obtain access for day-to-day operation and maintenance activities. In addition, ease of pipeline construction to the reservoir or tank location is a site access consideration.

Topography. Site topography will primarily affect reservoir layout and constructability at the site. Topographic features include the general slope of the land that can be incorporated into the site layout in order to minimize cuts and fills necessary to construct reservoir embankments. Although tanks require less area, topography also affects the amount of grading and site preparation needed. The topography of each of the sites varies considerably.

Geotechnical and Soil Conditions. Geotechnical factors are important to determine the appropriate construction methods, foundation design, types of structures, and the feasibility of construction. All of the reservoir sites have soils overlaying hard rock which will require blasting. However, some sites may require less blasting or have more ideal topography. Site topography and the ability to incorporate the natural slope into the site layout will affect reservoir layout and constructability at the site. Initial geotechnical site review was provided by Blackburn Consulting, Inc. (BCI) and is included as Appendix D. Further, more detailed geotechnical study will still need to be performed on the selected site during preliminary design.

Environmental Factors

Alternatives that require environmental permits and lengthy California Environmental Quality Act (CEQA) compliance have the potential to significantly increase costs and time, or directly impact the viability of a project should the permits become impossible to obtain, or the environmental mitigations become prohibitively costly or unreasonable. Development of the recommended reservoir site will require compliance with CEQA and may require compliance with the following regulations: Clean Water Act (CWA) Sections 401 and 404, Federal Endangered Species Act Section 7, National Historic Preservation Act Section 106, California Endangered Species Act, California Fish and Game Code Section 1600 *et seq.*, Regional Board general orders, and other local permits. The presence of jurisdictional waters of the United States, including wetlands, and special-status species can indicate the degree of environmental constraints for a given storage location. This section provides a summary of key environmental constraints (or lack thereof) for each storage site. Specific environmental factors are covered in detail in Section 16.

13.3.2 Reservoirs vs. Tanks Considerations

As discussed in Section 13.2, up to 11 MG may be needed for biostimulation storage. This storage could be accommodated in earthen reservoirs or tanks. Many of the evaluation criteria discussed above are more significant for determining an appropriate reservoir location. Tanks take up less space on the site, are less constrained by topography, and are less reliant on appropriate soil and geological conditions, unlike earthen reservoirs which are highly dependent upon the availability of suitable material for construction of embankments to make them cost effective to construct. Some environmental factors may also be mitigated through the use of tanks because of the smaller footprint and less disruption to the site both during construction and permanently during operation. To accommodate up to 11 MG, a very large storage tank or several (3 to 4) smaller tanks would be necessary.

Planning level cost estimates for tanks were solicited from several manufactures and compared to the cost estimates provided by Blackburn Consulting, Inc. (BCI) for earthen reservoirs (Appendix D). A comparison of the costs is provided in Figure 13-6. Since all storage would need ancillary facilities, such as piping and return pumps, the costs reflected in Figure 13-6 are capital costs for the basin itself and should be used for planning level comparison purposes only. The cost for a 4 MG reservoir is $1.26 million compared with approximately $3 million for tank storage. If building one basin for the 11 MG of storage needed, tank storage costs would range from about 3.5 to 7 times the amount of reservoir storage. Several smaller tanks would have a similar higher cost than a reservoir. Due to this very significant difference in basin cost, tanks were eliminated from consideration. However, the District may decide for other, overriding considerations to pursue storage tanks, therefore, potential tank sites are discussed below for completeness.

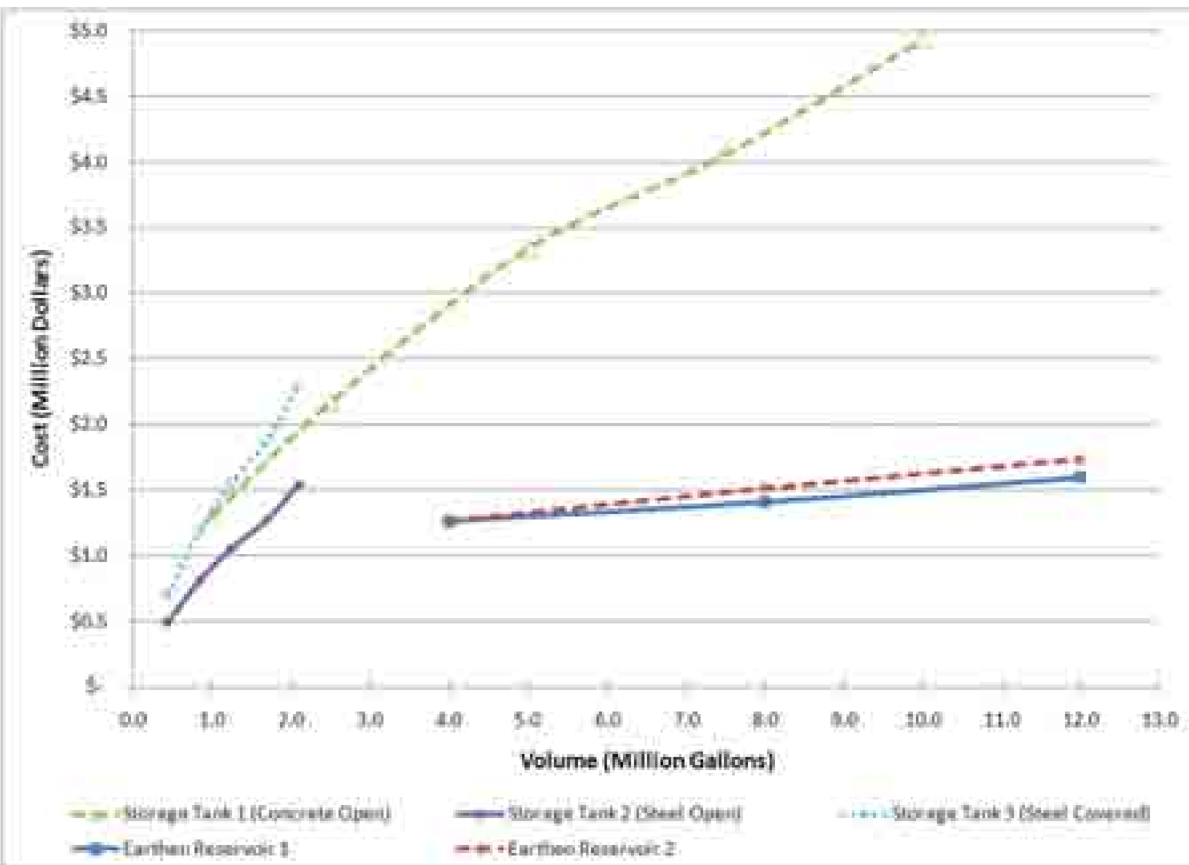


Figure 13-6

Planning Level Cost Comparisons of Earthen Reservoirs vs. Tanks

13.3.3 POTENTIAL STORAGE SITES AND INITIAL EVALUATION

The locations of the six potential earthen reservoir sites and three potential tank sites are shown in Figure 13-7. Location, topography, geotechnical constraints, environmental, and other general information for each site are considered below, leading to an overall assessment of suitability for biostimulation storage use. Most of the reservoir sites were observed during a site visit on October 1, 2009. Where property ownership is referenced, it is based on data provided at the time of the drafting of this section by the Assessor's Office of either Nevada or Placer County (dependent upon the location of the parcel).

Reservoir Site No. 1 – Myers Site

Reservoir Site No. 1 (Myers Site) is located approximately 1.5 miles northwest of the District WWTP and north of the South Yuba River (Figure 13-7). The 46-acre parcel (APN 047-350-07) is owned by Samuel Myers and is located in Nevada County. The property transitions into a steep, southwest facing slope as one moves north from the South Yuba River. Granitic rock and/or boulders are exposed along most of the surface. The October 1 site visit allowed observation of this parcel from an existing roadway. Access is difficult along steep, narrow roads.

The National Wetlands Inventory (NWI) maps do not show wetlands on this site (see Section 16). However, the site contains several drainages of various sizes (small to large) that appear to flow from the site, from above the site, and through the site to the South Yuba River, making these drainages potentially jurisdictional waters of the United States. If these features meet the criteria and are considered jurisdictional, this would require CWA Section 404 permitting if crossed or impacted. The potential for special-status species is low to moderate with the most likely special-status species being nesting raptors, since the site contains a number of large trees. Most of the trees include almost pure stands of lodgepole pine, with some intermittent Jeffrey pine and mountain hemlock.

Piping to the Myers Site would be difficult and costly due to shallow rock, the site's distance from the WWTP, and the steep slopes through which the pipeline would need to be constructed.

Due to this site's distance from the WWTP relative to other potential sites, difficulty of site access, potential for extensive environmental permitting, and difficult and costly pipeline routing, the Myers Site is eliminated from further consideration.

Reservoir Site No. 2 – Outfall Site

Reservoir Site No. 2 (Outfall Site) is located approximately 0.8 miles northwest of the District WWTP and north of the South Yuba River, directly across from the existing District effluent outfall (Figure 13-7). This 21-acre parcel (APN 047-330-05) is owned by Gretchen and Timothy Geiser (trustees) and is located in Nevada County. This site was observed and evaluated from parcels owned by the District located (south) across the South Yuba River from the Geiser property. The terrain is mostly gentle, rising from the South Yuba River to the north. A small "draw" meanders in a north-south orientation through the portions of the parcel observed.

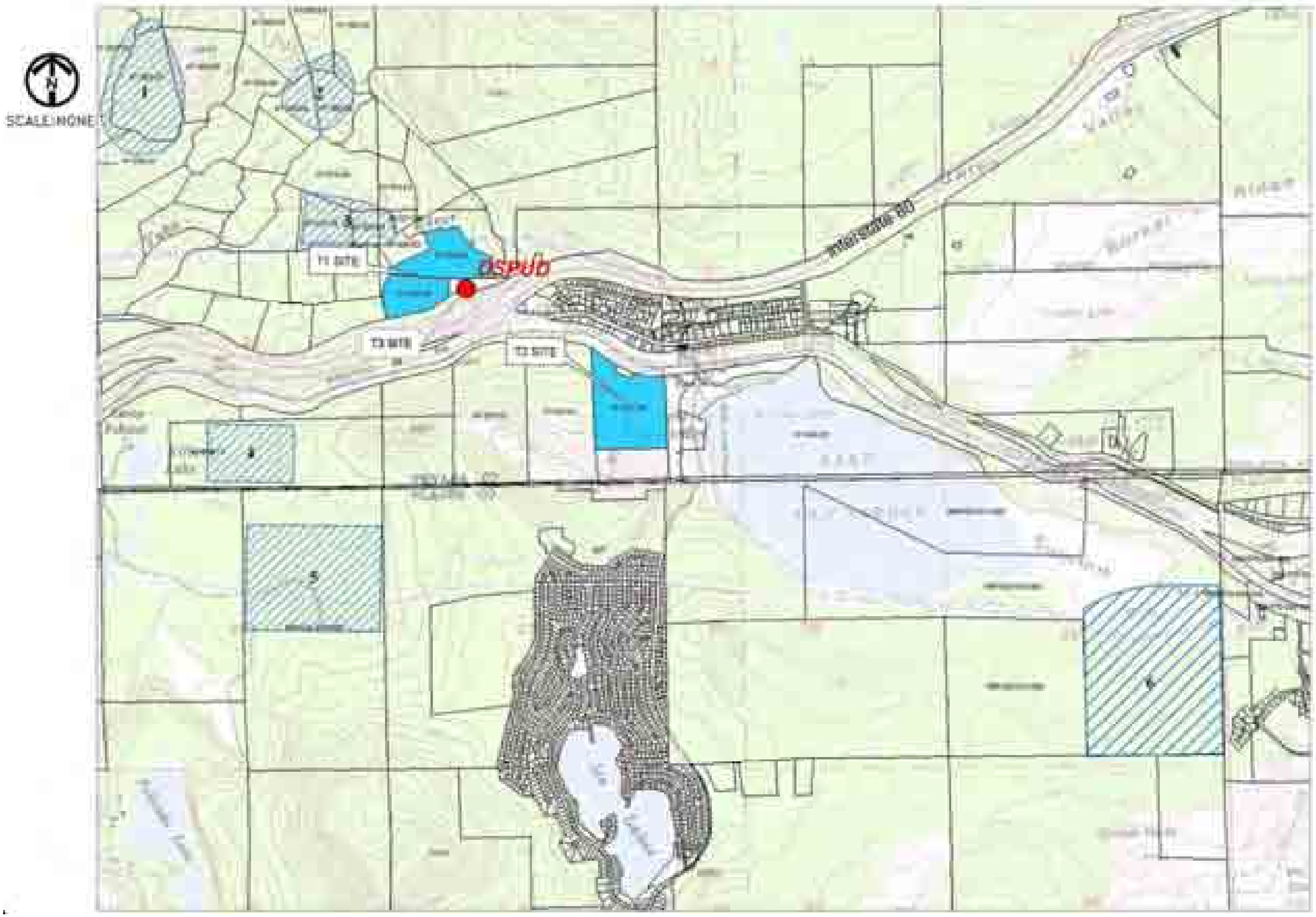


Figure 13-7
Potential Storage Sites Considered

The South Yuba River runs along the southern boundary of this site and the floodplain is not very wide, given the surrounding topography. However, areas of riparian trees and vegetation (mostly willows) line both sides of the river. The Outfall Site would be classified as woodland in areas upland of the riparian zone. The NWI maps do not show any wetlands. However, several drainages (generally small) pass through the site to the South Yuba River, making these drainages potentially jurisdictional waters of the United States. If these features meet the criteria and are considered jurisdictional, this would require CWA Section 404 permitting if crossed or impacted. The Outfall Site also contains dense woodland that would most likely require a moderate level of mitigation for trees protected by Nevada County and under CEQA.

The potential for special-status species is moderate with the most likely special-status species being nesting raptors, since the site contains a number of large trees. Results of a search of the California Natural Diversity Data Base (CNDDB) are summarized in Section 16. Several special-status species are identified in the CNDDB results as occurring in the vicinity of Site 2. The starved daisy (CNPS listed 1B species) is the only known special-status plant species in the vicinity of the Outfall Site. This particular plant is also the only known special-status plant species to occur in the vicinity of any of the effluent storage sites considered here.

Although the Outfall Site is favorably located near the existing effluent piping and District property, it is also on the opposite (north) side of the South Yuba River from the District. Therefore, piping to the site would need to cross the South Yuba River, which would require a CWA Section 404 (Corps of Engineers), CWA Section 401 (Regional Water Quality Control Board), and Section 1600 *et seq.* Streambed Alteration Agreement (California Department of Fish and Game) permits. These permits would necessitate mitigation and monitoring requirements.

The Outfall Site is located near existing piping, the District WWTP, and District-property (to the south, across the South Yuba River). In addition, the site geotechnical conditions appear to be favorable to reservoir construction. Based on this initial screening, the Outfall Site appears to be a potentially viable location for construction of a new reservoir. However, the presence of the drainages and special-status species, as well as the necessity of a pipe crossing of the South Yuba River, are a significant consideration.

Reservoir Site No. 3 – Franz Site

Reservoir Site No. 3 (Franz Site) is located approximately 0.5 miles northwest of the District WWTP, south of the South Yuba River, and immediately north of a regional power transmission line and easement (Figure 13-7). This 21-acre parcel (APN 047-310-18) is owned by Jennifer Franz and is located in Nevada County. The area of interest at the Franz Site is the southern portion, adjacent to the regional power transmission line and easement. Site review by BCI indicates some rocky tributaries between the transmission line and the South Yuba River (Appendix D). The higher ground between the first tributary (as one moves across the parcel from south to north) and the South Yuba River appear to have 5 or more feet of overburden soil. A substantial amount of borrow material may be required to construct a 4- or 3-sided reservoir.

In addition, rocky tributary channels may be difficult to excavate and require diversion of surface flows.

Though the NWI maps do not show any wetlands on this site, the tributary channels and drainages that pass through the site could be jurisdictional waters of the United States, which would require CWA permitting if crossed or impacted. The potential for special-status species is moderate with the most likely special-status species being nesting raptors since the site contains a number of large trees. Results of a search of the CNDDB are summarized in Section 16. Several special-status species are identified in the CNDDB results as occurring in the vicinity of this site. This site also contains dense woodland that would most likely require a moderate level of mitigation for trees protected by Nevada County and under CEQA.

The existing effluent pipeline crosses the transmission line easement and pipeline access to the Franz site could likely run along the easement to the site.

The Franz Site is located near existing piping, the District WWTP, and District-property and is on the south side of the South Yuba River. The geotechnical and environmental reviews indicated that the location is reasonable to consider for the reservoir site. Based on this initial evaluation, the Franz Site appears to be a potentially viable location for construction of a new reservoir.

Reservoir Site No. 4 – Royal Gorge North Site

Reservoir Site No. 4 (Royal Gorge North) is located approximately 1 mile southwest of the District WWTP, south of Interstate-80 (Figure 13-7). The parcel (APN 047-010-13) is owned by Royal Gorge, LLC. and is located in Nevada County. The central portion of the site includes a gentle topographic depression, or “bowl”. BCI estimated between 5 and 10 feet of soil in this area. Excavation of the bowl might allow the slopes to be used to build an enclosed reservoir.

The NWI maps do not show wetlands on this site. However, the site contains several drainages of various sizes that most likely drain to the South Yuba River or nearby lakes, making these potentially jurisdictional waters of the United States. If these features meet the criteria and are considered jurisdictional, this would require CWA Section 404 and 401 permitting if crossed or impacted. Generally, on the south side of the South Yuba River, red fir trees are very plentiful. However, lodgepole pines, mountain hemlock, and western white pine are also intermixed. The density of trees on the Royal Gorge North Site is very high, with a dense stand of lodgepole pines in and adjacent to the “bowl”.

The potential for special-status species is moderate with the most likely special-status species being the nesting raptors, since the site contains a number of large trees. The CNDDB shows several special-status wildlife species in the vicinity of the Royal Gorge North. A summary of these species is provided in Section 16. The starved daisy (CNPS listed 1B species) is the only known special-status plant species in the vicinity of the site. Other documented sensitive species on the southern side of the South Yuba River include the Pacific fisher (CA species of concern, Federal Candidate), Sierra marten (CA species of concern), northern goshawk (CA species of concern), yellow warbler (CA species of concern), and Sierra Nevada mountain beaver (CA species of concern). Nesting raptors and birds would be the most likely sensitive species found on the Royal Gorge North Site.

Although piping access to the Royal Gorge North Site has less topographical constraints compared to some other locations, the pipeline would include crossing Interstate 80. In addition, this site is farther from the District WWTP than the previous two sites discussed.

Due to this site's distance from the WWTP and disposal fields relative to other potential sites and the pipeline crossing of Interstate 80, the Royal Gorge North Site is eliminated from further consideration.

Reservoir Site No. 5 – Royal Gorge South Site

Reservoir Site No. 5 (Royal Gorge South) is located approximately 1.2 miles southwest of the District WWTP and south of Interstate-80 (Figure 13-7). The parcel (APN 069-010-018) is owned by Royal Gorge, LLC. and is located in Placer County. The area of interest at the Royal Gorge South Site is the central northern portion, which includes a relatively flat terrain. BCI identified the site as mostly hard rock at the surface, with a wetland swale and transmission lines through the central portion.

The NWI map and site visit indicated that the site is crossed by a large drainage that contains several wetlands, including one very large wetland in the middle of the site. The site contains red fir, lodgepole pines, mountain hemlock, and western white pine are also intermixed.

Although piping access to the Royal Gorge South Site has less topographical constraints compared to some other locations, the pipeline would include crossing Interstate 80. In addition, this site is over one mile from the District WWTP.

The Royal Gorge South Site is the least desirable of previously mentioned sites due to the large drainage and wetland complexes within the site. Due to this site's distance from the WWTP relative to other potential sites, pipeline crossing of Interstate 80, and drainage and wetland constraints, the Royal Gorge South Site is eliminated from further consideration.

Reservoir Site No.6 – Sugar Bowl Site

Reservoir Site No. 6 (Sugar Bowl Site) is located approximately 2.5 miles southeast of the District WWTP, south of Interstate-80 and Donner Pass Road (Figure 13-7). The parcel (APN 069-020-070) is owned by Sugar Bowl Corp. and is located in Placer County. The Sugar Bowl Site is at the southeast end of old Lake Van Norden, which is currently mostly a large meadow. As discussed in more detail in Section 16, this meadow is considered a sensitive area. In addition, there are regional power transmission lines crossing the parcel in an east-west direction. This site includes a relatively flat to moderately sloping north-facing terrain and BCI indicated that there is likely volcanic rock within about 5 to 10 feet below ground surface (Appendix D). The portion of the parcel of potential interest is south and east of the sensitive meadow area and south of the transmission lines. Areas of higher ground exist outside of the meadow/wetland complex.

Pipeline access to the Sugar Bowl Site would include crossing Interstate 80. In addition, this site is located the farthest from the District WWTP and is approximately 2 miles from the existing spray irrigation disposal fields.

From a geotechnical perspective, the Sugar Bowl Site is viable. However, this site would require the longest pipelines to/from the site and potentially face significant public opposition due to its proximity to Lake Van Norden and the associated meadow. Therefore, the Sugar Bowl Site is eliminated from further consideration.

Tank Sites

Three potential storage tank sites were identified: Tank Site No. 1 (T1), Tank Site No. 2 (T2), and Tank Site No. 3 (T3, WWTP). The three tank sites would face similar engineering requirements. Tanks would require less space than the earthen reservoirs and, thus, would be less constrained by topography and possibly result in less environmental impacts.

Tank Site No. 1 (Site T1) is located adjacent to the District WWTP, to the north and west (Figure 13-7). The parcels (APN 047-320-05 and APN 047-320-06) are owned by John and Lisa Mohun and located within Nevada County. The terrain is gentle near the WWTP, but changes quickly to become moderately steep to the north and west. The surface is mostly silty sand with cobbles and numerous large boulders.

Tank Site No. 2 (Site T2) is located approximately 0.3 miles to the southeast of the District WWTP, south of Interstate-80 (Figure 13-7) on the existing District irrigation disposal site at Soda Springs. The parcel (APN 047-021-48) is owned by the Boreal Ridge Corporation and located within Nevada County. The terrain is moderate to very steep and has been logged for ski/tubing runs. Logistically, it may be difficult to site storage tank(s) on or near the existing ski slopes. In addition, although Site T2 is at the location of the existing spray disposal fields, additional pumping facilities would be required since the current irrigation pump station is located at the WWTP.

Tank Site No. 3 (Site T3 or WWTP Site) is located within the WWTP (Figure 13-7). Although the WWTP site is space limited, if mechanical sludge dewatering is used in the future, there is the potential to use the existing sludge drying beds location for construction of a biostimulation storage tank. Siting of a tank within the existing District WWTP site and on the sludge drying beds would limit environmental impacts since this area is already developed.

None of the tank sites contain NWI mapped wetlands; however, this does not preclude any of these sites from containing jurisdictional waters of the U.S. A formal assessment of each site would be required to determine if any jurisdictional features exist.

The CNDDB shows several special-status wildlife species in the vicinity of the WWTP site and Site T1. A more detailed description of those species is provided in Section 16. The starved daisy (CNPS listed 1B species) is the only known special-status plant species in the direct vicinity of the WWTP site and Site T1. Site T2 does not contain the dense woodland of red fir and lodgepole pines that are located on adjacent sites since it is a ski run; however, this site does contain visible wetland associated vegetation. However, the existing sprinkler operations at the Soda Springs site are supporting this vegetation. As such, these areas would not be considered jurisdictional and would not require CWA permitting.

Site T1 is adjacent to the WWTP property and is crossed by the existing effluent pipeline. Site T2 is relatively close to the District WWTP and would benefit from the existing irrigation pumping and conveyance infrastructure in place to deliver treated effluent to the existing Soda Springs site.

The initial evaluation of the tank sites is considered in relation to the potential reservoir sites. Although the tank sites may have less engineering and environmental constraints, planning level estimates of tank costs compared to earthen reservoir costs indicate that tanks are several times more expensive than comparable storage in earthen reservoirs (Section 13.3.2 above). Due to this significantly higher cost, tanks are eliminated from further consideration.

13.3.4 SUMMARY OF ALTERNATIVE ANALYSIS AND RECOMMENDED ALTERNATIVE

Distance from the District WWTP is a key consideration in determining the appropriate location for effluent storage. Reservoir Sites 1, 4, 5, and 6 are located one mile or greater from the District WWTP. In addition, Reservoir Sites 1 and 5 have significant geotechnical limitations. From an environmental factors standpoint, Reservoir Site 1 is logistically unfeasible and Sites 5 and 6 have a significant number of wetlands nearby as well as permitting issues. These geotechnical and environmental constraints further reinforce the undesirability of these locations. Planning level estimates of tanks costs compared to earthen reservoir costs indicate that tanks are significantly more expensive.

Reservoir Sites 2 (Outfall Site) and 3 (Franz Site) are within one mile of the WWTP and appear to be viable storage locations from an engineering/geotechnical and environmental perspective. The Franz Site is located on the south side of the South Yuba River; whereas the Outfall Site is on the north side and would require a river crossing for pipeline access, more piping, and larger pumps to return flow to the WWTP. Both sites contain potential sensitive special-status species, most likely nesting raptors and birds. The Franz site is also the closest reservoir site to the District WWTP, is close to the existing outfall line, near District-owned parcels for potential future irrigation, and has the most potential for flexibility in the future.

Based on all of the above, an earthen reservoir at Site 3 (Franz Site) is the recommended alternative for biostimulation storage.

13.4 RECOMMENDED BIOSTIMULATION STORAGE FACILITIES

Presented below is a description and cost estimate for the recommended biostimulation storage facilities, followed by a discussion of operational and permitting issues.

13.4.1 DESCRIPTION AND COST ESTIMATE

BCI. developed preliminary site layouts/conceptual designs and cost estimates for 4, 8, and 12 MG reservoirs (Appendix D) on both Sites 2 and 3. These designs assume an 18-inch outlet pipe, encased in concrete, at the base of the embankment, and a simple overflow pipe for spillway considerations. Allowance was also made for a service road around the reservoir and a diversion ditch to direct surface runoff away from the reservoir. In addition to the basin itself, an access road from the existing outfall, site work, and a storage return pump station and piping will be necessary to bring the stored effluent back to the wastewater treatment plant. At the wastewater treatment plant, piping and valves will be provided to allow the return flow to be routed to either the existing emergency storage tank (for subsequent pumping to the effluent irrigation disposal system) or to one of the equalization storage tanks, for retreatment through the wastewater treatment plant.

A planning level cost estimate for a 12 MG reservoir and ancillary facilities on the Franz Site is provided in Table 13-2.

Table 13-2
Donner Summit Public Utility District Reservoir Site 3 (Franz Site)
Planning Level Cost Estimate

Facility	Cost, $
12 MG Earthen Reservoir Construction [a]	$1,148,000
Land Acquisition [b]	$200,000
Access Road [c]	$27,000
Washdown Facilities	$50,000
Return Pump Station [d]	$300,000
Pipeline [e]	$463,000
Subtotal 1	**$2,188,000**
Contingency (20%)	$438,000
Subtotal 2	**$2,626,000**
General Conditions, Overhead, Profit (20%)	$525,000
Total Construction Cost	**$3,151,000**
Engineering, Admin, Environmental, etc. (25%)	$788,000
TOTAL ESTIMATED CAPITAL COST (Rounded)	**$3,939,000**

(a) Includes mobilization, embankment, spillway, outlet, liner, and diversion channel. Per BCI Report (Appendix D).

(b) Land acquisition estimated at $25,000 per acre.

(c) Includes 400 feet road off the existing utility access road, consisting of 4-inch graveled, 10-foot wide road at $20/cubic yard placed gravel.

(d) Includes pump station, building, and electrical/instrumentation.

(e) Includes approximately 2,640 feet of 6-inch forcemain at $90/foot and approximately 1,500 feet of 10-inch gravity pipeline at $150/foot.

13.4.2 OPERATIONAL CONSIDERATIONS

Potential modes of operation for the reservoir and various operational issues are discussed below.

Modes of Operation

The main function of the reservoir will be to store all plant effluent during the time between the termination of river discharge and beginning effluent irrigation disposal. Subsequently, it may take several months during the summer to drain the reservoir down and prepare it for the fall and winter.

While the reservoir is partly drained during the summer months, it can be used to provide additional emergency/operational storage for irrigation. If for any reason, spray irrigation were not possible for a time, due to a failure in the system or other issue, the plant effluent could be accumulated in the biostimulation storage reservoir until the issue is resolved. Additionally, if in a future expansion it is desired to investigate irrigation disposal in the area of the biostimulation storage reservoir, the storage return pumps could potentially be used as irrigation supply pumps and the biostimulation storage reservoir would provide the needed operational storage at that location.

In the winter months, assuming the reservoir is kept empty, it would be available for emergency storage use. Any noncompliant plant effluent, exceeding the storage capacity of the emergency storage tank at the plant could be held in the biostimulation storage reservoir and then returned to the plant for retreatment.

Dealing with Algae and Cleaning the Reservoir After the Summertime Use

During the summer months, the reservoir will be full or partly full and will be an ideal location for algae to grow. For use on the Soda Springs ski area, however, the resultant green color should not be a major issue. The automatic self-cleaning strainer in the irrigation pumping system should prevent algae from clogging sprinklers.

Perhaps the biggest concern with algae is that dead algae will settle to the bottom of the reservoir, leaving a residue that must be cleaned out when the reservoir is drained. However, cleaning of the reservoir will be needed anyway because of windblown dirt and debris and other items that will accumulate. Because of this, the reservoir will be equipped with washdown hydrants around the perimeter. Operational staff will have to clean the reservoir each fall and return the dirty washdown water for treatment through the plant, prior to allowing any drainage from the reservoir to the river.

Dealing with Ice and Snow

As an open top earthen reservoir, the biostimulation storage facility will capture rainfall and snow during the fall, winter, and early spring. Near the end of spring, when the reservoir is needed for its main intended purpose, it is important that the reservoir volume needed for biostimulation storage be fully available. Therefore, the plan is to allow any accumulated precipitation to drain from the storage reservoir to the river, until the time that the effluent discharge to the river must be terminated.

One potential concern is that the reservoir will be partly full of snow and ice that has not fully melted away before storage of effluent is initiated. Such snow and ice would use up some of the available storage capacity of the reservoir. Therefore, that loss of volume must be accounted for in the design, or means to prevent the volume loss must be developed. Several potential methods for dealing with this issue are considered briefly below:

- **Physical Removal:** Physical removal of snow and/or ice is not recommended because of potential damage to the reservoir lining system.
- **Melting with Effluent:** Prior to the time that storage is required, the reservoir could be partly filled with effluent to help melt any remaining ice and snow. Then, the reservoir contents would be discharged to the river, hopefully without retreatment through the wastewater treatment plant. If needed, personnel could use the reservoir washdown facilities to help melt the ice and snow.
- **Maintaining a Pool:** The reservoir would not be empty during the winter. Instead, the reservoir would be filled with effluent at the beginning of the winter and the plant effluent would be routed through the reservoir on its way to the river discharge. Just before the reservoir is needed for biostimulation storage, it would be fully drained to the river to make the volume available for storage use. This option has the added advantage of providing effluent cooling before discharge. However, this option would eliminate the potential use of the reservoir for emergency storage during the fall and winter. A variation of this option could be to keep the reservoir only partly full, to maintain possible emergency storage volume.

The best method for dealing with ice and snow in the reservoir should be investigated further during preliminary design, including discussions with the Regional Board.

Review of Proposed Operations with the Regional Board

Prior to finalizing the plans for the biostimulation storage reservoir, all of the proposed operations and issues discussed above must be discussed with the Regional Board and any permitting issues resolved.

Section 14

Effluent Irrigation Disposal

Section 14

Effluent Irrigation Disposal

As developed in previous sections, DSPUD disposes of its wastewater effluent by direct discharge into the South Yuba River during times of the year that disposal on land is not practical (typically mid-autumn through spring). When land disposal is practical, however, river discharge is not allowed. At the minimum, river discharge is prohibited in August and September.

Since the mid-1980s, DSPUD has disposed of treated and disinfected effluent during dry periods (typically, in summer to mid-autumn) by irrigating portions of the Soda Springs Ski Area. In this section, the historical performance of the existing land disposal system is reviewed and the need for expansion is investigated based on: 1) projected growth and increased flows, as developed in Section 4, and 2) the impact of potential springtime effluent storage to mitigate biostimulation in the South Yuba River, as developed in Section 13.

Important base assumptions to this investigation are:

1. The current effluent irrigation practice is acceptable, i.e., it is not causing any unacceptable degradation of water resources or the environment.
2. The current effluent irrigation practice can be expanded without that expansion causing unacceptable degradation of water resources or the environment.
3. Dwelling unit occupancy rates (highly seasonal) and wastewater characteristics will remain similar to current values for the current level of community development, and for any increase in community development.
4. Changes in effluent disposal practices are limited to those specifically described and analyzed in this section.
5. There will be no change in climatic conditions that would significantly impact the design of the irrigation facilities, over at least the next 20 years.

14.1 EXISTING FACILITIES, OPERATIONS AND PERFORMANCE

In the paragraphs that follow, a description of the existing irrigation disposal system is presented, followed by a discussion of its operation and performance.

14.1.1 DESCRIPTION OF EXISTING IRRIGATION DISPOSAL SYSTEM

The effluent irrigation areas at the Soda Springs site are grass-covered slopes, used as ski/tubing runs during winter. The irrigated runs are located on the north side of an unnamed hill about five hundred feet south of the South Yuba River, west of Lake Van Norden, and north of Ice Lakes (see Figure 14-1). The existing sprinkler irrigation system covers an area of about 45 acres.

Figure 14-1
Existing Irrigation Disposal Facilities Location

Effluent is applied at agronomic irrigation rates such that the bulk of the applied effluent is evapotranspired by vegetation.

Average precipitation at the site is about 52 inches per year, most of which falls as snow. The average annual snowfall reported at the nearby Central Sierra Snow Laboratory is nearly 34 feet. Snow frequently remains on the effluent irrigation area until late May. Effluent irrigation typically does not begin until site soils have largely drained of snowmelt and dried somewhat. Because of uncertainty of when effluent can be applied to land at this site, the District's waste discharge requirements allow effluent discharge to the South Yuba River when land disposal is not feasible in all months except August and September. In these two months, effluent must be either stored or applied to land.

Effluent is delivered to the irrigation area via an irrigation pump station and an 8-inch force main from the DSPUD wastewater treatment plant. The pump station includes two (one standby) 15 hp pumps in series with two (one standby) 100 hp irrigation pumps, each with a capacity of 600 gpm. There is an automatic self-cleaning strainer between the two stages of pumps. The irrigation system, itself, consists of an extensive array of impact sprinklers. Within the irrigation area, piping to the sprinklers is all buried. Each sprinkler assembly is removable via a quick-disconnect coupling located in a valve box. In this way, the sprinkler risers and sprinkler heads can be removed and stored each winter, so as not to be a hazard when the area is used by skiers. Any irrigation water runoff from the irrigated area is collected in a ditch and routed to a runoff recovery pond, from which it is returned to the operational storage tank at the wastewater treatment plant for re-pumping to irrigation. In major rain events, only the initial rainfall runoff is captured in the runoff recovery pond and the remainder is allowed to flow from the site.

14.1.2 Existing Irrigation System Operation and Performance

Effluent disposal at the Soda Springs site is accomplished in accordance with the District's current waste discharge requirements. Specifically, irrigation disposal (or effluent storage) is accomplished in August and September as a minimum, but also at such other times as weather and soil conditions allow. In Table 14-1, the dates of stopping and starting river discharges and irrigation disposal from 2002 through the spring of 2009 are indicated. As shown in the table, irrigation has sometimes been possible for most of July, but sometimes irrigation has not been started until August. Typically, irrigation is continued throughout most or all of October and sometimes ends in early November. However, sometimes irrigation is terminated early in October.

The existing irrigation system is arranged in four pressure zones, with Zone A lowest on the slopes and Zone D highest. District operations staff report that Zone A soils seem to stay moist, even without operation of sprinklers in that zone. When sprinklers are operated in Zone A, runoff occurs fairly rapidly. It is hypothesized that the areas in Zone A that stay moist without irrigation are receiving moisture from groundwater migrating down the ski slopes. Since the velocity of horizontal water flow through the soil is very slow (perhaps on the order of one foot per day), the original source of the water is probably due to precipitation and snow melt on the slopes above over many months. In fact, since the slope length (measured up the slope to the point of rock outcroppings) is over 1000 feet, soil moisture at the base of the slope could be due to rainfall, snow melt and/or irrigation water entering the soil several years before.

Table 14-1
Dates of River Discharge and Irrigation Disposal

Year	Last Date of River Discharge	First Date of Irrigation Disposal	Last Date of Irrigation Disposal	First Date of River Discharge
2002	July 5	July 9	November 6	November 9
2003	June 30	July 2	October 31	November 7
2004	July 24	August 1	October 21	October 22
2005	July 23	July 27	November 3	November 4
2006	June 30	July 5	November 7	November 13
2007	July 18	July 25	October 9	October 17
2008	July 2	July 9	November 1	November 3
2009	June 26	July 1		

A soils study was completed on the ski slopes in October 1984 to support the original design of the existing irrigation system. In that study, moist and poorly drained soils were identified as a concern requiring special management practices in about the western half of what is now Zone A. However, the eastern half of the Zone A area was not identified as having that problem in October 1984 (before irrigation system installation and operation). Now that irrigation is practiced on the slopes above, even the eastern half of the Zone A area has moist soils, without direct irrigation.

Based on the discussion above, about 25 percent of the existing 45 acre irrigation system cannot be used effectively. Therefore, the effective area of the existing irrigation disposal system is estimated to be about 34 acres.

Despite the areas that are not used, the existing system apparently functions well and is able to accommodate existing flows in August and September (the months in which river discharge are absolutely prohibited) without problem. Of particular concern is how the system functions when stressed due to significant natural precipitation, which limits the amount of effluent disposal. District operations staff report that when precipitation events occur, effluent application operations are typically interrupted only for short periods of time (2 to 3 days). Furthermore, events in 1986 and 1989 were such that September rainfall amounted to approximately 6.5 inches (which represents a return frequency of 200 years). The District reports that, although some storage was required during individual rain events, the disposal site was sufficient to dispose of all treated effluent generated during that time. Wastewater flows at that time were not substantially different from current flows. Such empirical evidence suggests that the existing system can handle even extreme design events (high rainfall) at current flows.

14.2 FUTURE IRRIGATION AREA REQUIREMENTS

In this section, the required sizing of the irrigation disposal area to suit future design flows is investigated. In addition to future flow increases due to growth within the service area, the irrigation disposal system must be designed to accommodate any effluent stored prior to the start of annual irrigation operations to mitigate biostimulation in the South Yuba River. The timing and amounts of biostimulation storage are discussed in Section 13. It is helpful to first determine irrigation area requirements without biostimulation storage and then to investigate the impacts of biostimulation storage.

14.2.1 IRRIGATION LAND REQUIREMENTS FOR EFFLUENT FLOWS OCCURRING WHEN IRRIGATION IS ONGOING

It is appropriate to determine irrigation area requirements based on calculations for the months of August and September, when irrigation disposal (or storage) is absolutely required. Although July should not be used as a basis of design, it is included in the analysis presented below for general information and comparison to the evaluations for August and September.

The Regional Water Quality Control Board typically requires that water balances for land disposal systems be based on 100-year return frequency precipitation. Accordingly, 100-year return frequency total monthly precipitation amounts for Soda Springs in the months of July, August and September were obtained from the California Department of Water Resources (DWR) and are 4.43, 2.39, and 5.43 inches, respectively.

Design wastewater effluent flow rates for the months of August and September must be established for use in determining irrigation area requirements. These flow rates should be those most likely to occur at the same times as 100-year return frequency monthly precipitation amounts indicated above. In the winter months, it is common for precipitation to result in increased infiltration and inflow and, therefore, increased wastewater effluent flow rates. However, that is unlikely to be the case for precipitation occurring in August and September. At this time of the year, large precipitation events tend to be associated with short-term thunderstorms. Much of the precipitation would be expected to runoff, without soaking into the soil. The precipitation that does soak into the soil throughout the DSPUD and SLCWD sewage collection areas is unlikely to produce significant infiltration and inflow, because the soil would be largely dry before the storm and able to absorb a substantial amount of water, without producing localized groundwater accumulations above sewer lines.

In Figure 14-2, monthly average flows at the DSPUD wastewater treatment plant are plotted against monthly total precipitation amounts for July, August, and September from 2002 through 2009. No apparent trend toward higher flows with higher rainfall amounts is apparent.

Based on the discussion above and the data shown in Figure 14-2, it is considered appropriate to use average monthly flow projections for July, August, and September in conjunction with 100-year return frequency precipitation for these months. Coupling lower probability higher flows with the 100-year rain amounts would constitute an overall condition with a return frequency less than once in 100 years (i.e., less likely to occur).

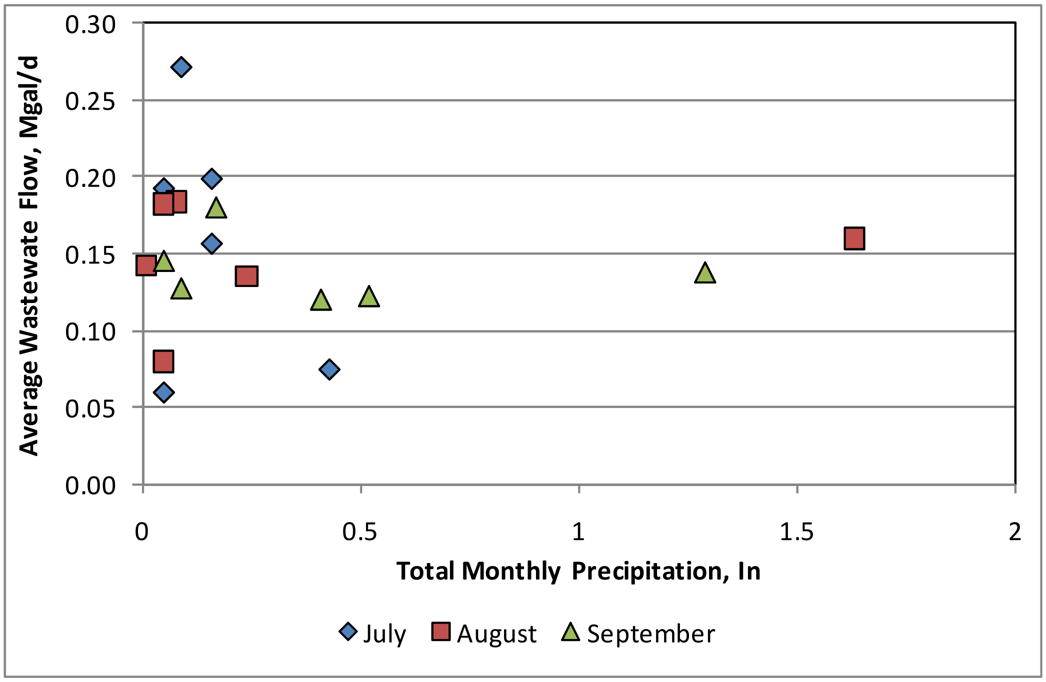


Figure 14-2
DSPUD Average Monthly Wastewater Flow versus Total Monthly Precipitation (2002-2009)

Calculations of irrigation land area requirements for July, August, and September (each considered independently) are shown in Tables 14-2 and 14-3 for existing and future flows, respectively. Independent monthly assessments using 100-year return frequency precipitation for each month are appropriate because the proposed design must assure disposal of all effluent in each of the months. The calculations do not imply that a 100-year July would be followed by a 100-year August and a 100-year September. Rather, the calculations reflect that in some years, abnormally high precipitation could occur in July, while in other years, abnormally high precipitation could occur in August or September. The disposal system must accommodate all of these possibilities.

As indicated in Tables 14-2 and 14-3, if only the months of August and September are considered, the irrigation areas required for existing and future conditions (not including any impact of future biostimulation storage) are estimated at 27.5 and 31.5 acres, respectively. Both of these are below the existing estimated effective area of about 34 acres. However, the calculations shown in Tables 14-2 and 14-3 are based on various key assumptions, which, if altered, could substantially change the results. These assumptions and sensitivity analyses are discussed below.

Table 14-2
Irrigation Land Area Requirements for Existing Flows

Parameter	Month		
	July	August	September
Input Data			
Influent Flow, Mgal/d	0.2	0.17	0.14
Precipitation, Inches	4.43	2.39	5.43
Precipitation Effectiveness Factor	0.5	0.5	0.5
Grass ET, Inches	8.99	7.75	5.7
Irrigation Efficiency, %	90	90	90
Allowable Net Volume to Storage, Mgal	0	0	1.5
Calculations			
Total Influent Volume for Month, Mgal	6.2	5.27	4.2
Total Volume to Irrigation for Month, Mgal	6.2	5.27	2.7
Total Water Demand, Inches	9.99	8.61	6.33
Effective Precipitation, Inches	2.22	1.20	2.72
Maximum Possible Irrigation, Inches	7.77	7.42	3.62
Irrigation Land Area Required, Acres	29.4	26.2	27.5

Table 14-3
Irrigation Land Area Requirements for Future Flows
(Not Including the Impact of Biostimulation Storage)

Parameter	Month		
	July	August	September
Input Data			
Influent Flow, Mgal/d	0.24	0.2	0.17
Precipitation, Inches	4.43	2.39	5.43
Precipitation Effectiveness Factor	0.5	0.5	0.5
Grass ET, Inches	8.99	7.75	5.7
Irrigation Efficiency, %	90	90	90
Allowable Net Volume to Storage, Mgal	0	0	2
Calculations			
Total Influent Volume for Month, Mgal	7.44	6.2	5.1
Total Volume to Irrigation for Month, Mgal	7.44	6.2	3.1
Total Water Demand, Inches	9.99	8.61	6.33
Effective Precipitation, Inches	2.22	1.20	2.72
Maximum Possible Irrigation, Inches	7.77	7.42	3.62
Irrigation Land Area Required, Acres	35.2	30.8	31.5

In designing irrigation facilities to meet the needs of grass or other vegetation, it is recognized that it is impossible to apply exactly the amount of water needed uniformly over the irrigated area. Because of sprinkler spray distribution patterns, some areas will receive more water than others. Therefore, to assure that the areas receiving the least water will receive as much water as can be used there, areas receiving the most water must be provided with an excess. Any excess water not used for evapotranspiration will simply runoff or percolate below the root zone of the plants and be “lost” as groundwater. In addition, some water will be “lost” due to drift in the wind or evaporation before hitting the ground in the intended area. To allow for these factors, an irrigation efficiency factor is typically included in the design of an irrigation system. A typical irrigation efficiency is around 80 percent, indicating that 80 percent of the applied water is used to meet the evapotranspiration needs of the vegetation in the application area and 20 percent escapes the application area as surface runoff, groundwater and/or as evaporated or drifting spray. Some deep percolation is required, at least occasionally, to prevent undesirable salt buildup in the root zone.

In a typical irrigation system design, the objective is to conserve water and apply no more than needed. Therefore, in that case, a relatively low irrigation efficiency factor (80 percent or less) produces a conservative design because it results in a higher water demand. For land disposal systems, the opposite is true; the objective is to dispose of as much water as possible and, therefore, it is conservative to assume a high irrigation efficiency. A high irrigation efficiency implies that very little water is "lost" by the mechanisms indicated above.

For this Facilities Plan analysis, a relatively conservative irrigation efficiency of 90 percent is assumed and is used in Tables 14-2 and 14-3. This implies that only 10 percent of the applied water would not be used for evapotranspiration in the application area and would escape by one of the mechanisms previously discussed. Some of this escaping water may be used by vegetation in areas further down the slope, which are not irrigated (Zone A, as noted above). However, some of the applied water will escape the site entirely. It is beyond the scope of this analysis to fully define the fate of all applied water. The relatively conservative irrigation efficiency of 90 percent was assumed because of shallow soils that would limit groundwater escape and because there is a runoff recovery system that would prevent surface runoff as a substantial mode of escape (for irrigation water, not extreme precipitation events). Further evaluations are appropriate during future design.

For the calculations presented in Tables 14-2 and 14-3, it is assumed that all wastewater produced in a given month would have to be disposed of in the same month, with the exception of September. For existing conditions in Table 14-2, it was assumed that 1.5 Mgal of the wastewater effluent generated in September could be stored and then retreated for river discharge in October (if land disposal is curtailed at the end of September). This is based on the capacity of the existing emergency storage tank. For future conditions in Table 14-3, it is assumed that 2 Mgal could be stored in September, based on the capacity of the existing emergency storage tank plus the capacity of the proposed Equalization Storage Tank 2, which could be used for this purpose (no reliance on possible biostimulation storage).

Another key assumption included in the calculations in Table 14-2 is that only 50 percent of the water from precipitation on the irrigated area would soak into the soil or would runoff and be captured by the runoff recovery system for subsequent irrigation use. This is represented by the "Precipitation Effectiveness Factor" indicated in the tables. The 50 percent factor is believed to be reasonably conservatively (i.e., on the high side), considering the high precipitation amounts indicated for the100 year return frequency events. In other words, it is expected that, with extreme summer thunderstorms, at least 50 percent of the precipitation would runoff and would not be captured by the runoff recovery system. For example, in September 1986, when there was a total of 6.45 inches of rain for the month, 4.58 inches occurred in a three-day period. Similarly in 1989, when there was a total of 5.0 inches of rain for the month, 4.01 inches fell in a three-day period. Considering that a normal irrigation application is only about 1-inch every three days, it is easy to understand that a large amount of runoff would be generated by extreme precipitation events, particularly if the soils were recently irrigated.

In Figures 14-3 and 14-4, the results of sensitivity analyses are presented to indicate the amount of irrigation land required for September as a function of the carryover storage amount and the precipitation effectiveness factor for existing and future flows, respectively. As indicated in Figure 14-3, with no storage in September, the existing estimated effective irrigation area of 34 acres is adequate only if at least about 70 percent of the 100-year precipitation amount runs off without being captured. With 1.5 Mgal of storage, only about 40 percent runoff is needed. For the future flow conditions shown in Table 14-4, with no storage and with 2 Mgal of storage, the corresponding runoff amounts are about 85 percent and 45 percent respectively.

Despite the range of possible results indicated in Figures 14-3 and 14-4, the best estimates of irrigation land area requirements for existing and future flows are currently believed to be those indicated in Tables 14-2 and 14-3, respectively. Therefore, without consideration of the impact of biostimulation storage, no additional irrigation area would be required to suit future flows. The impact of biostimulation storage on this assessment is addressed below.

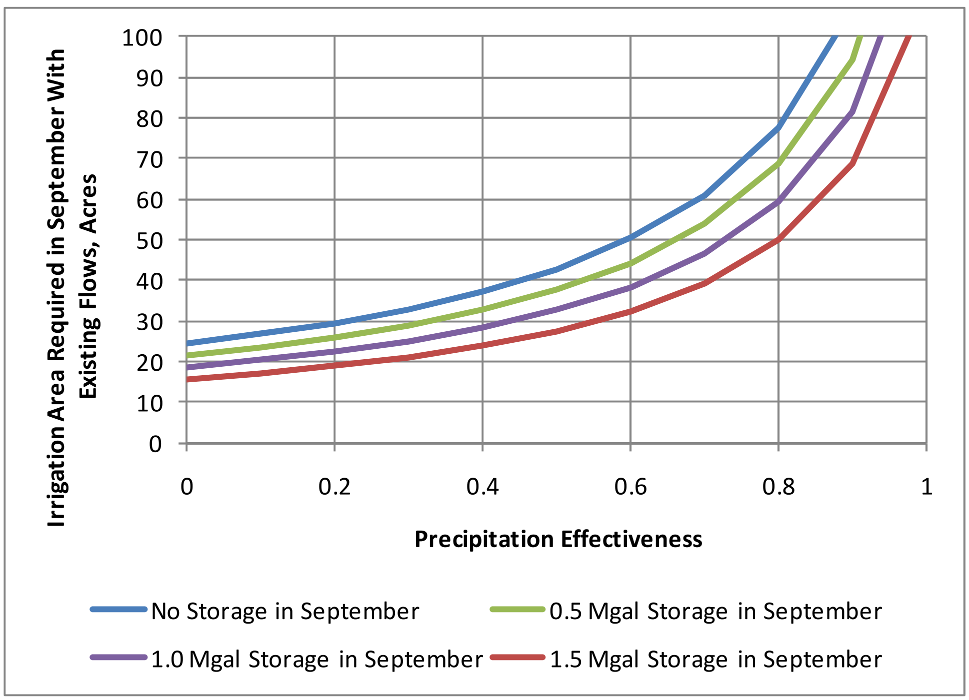


Figure 14-3
Irrigation Area Required in September With Existing Flows, Sensitivity to Storage and Precipitation Effectiveness

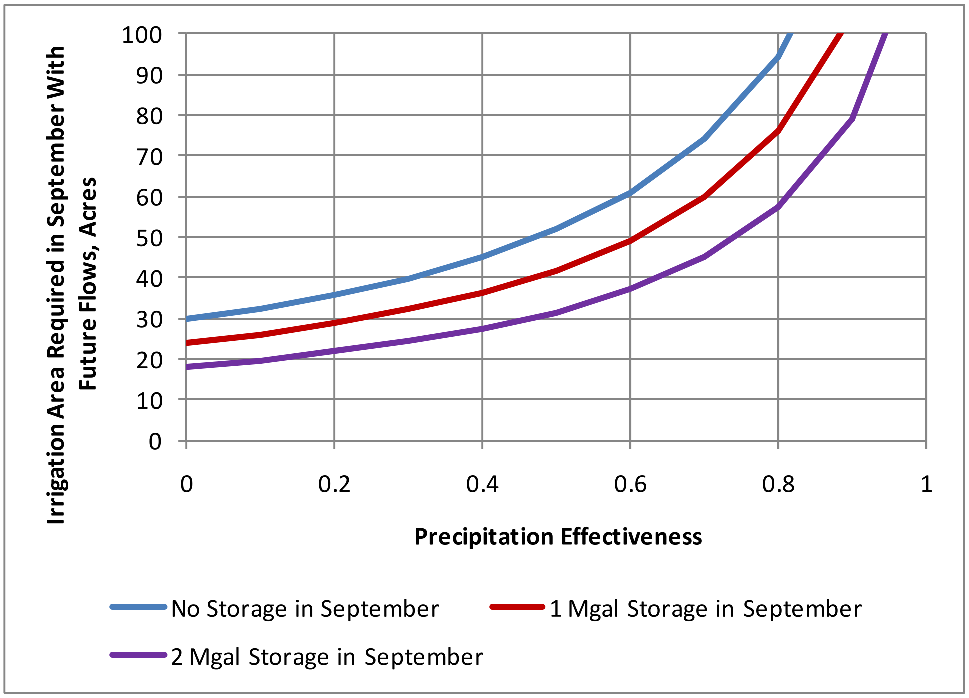


Figure 14-4
Irrigation Area Required in September With Future Flows, Sensitivity to Storage and Precipitation Effectiveness

14.2.2 Impact of Biostimulation Storage on Irrigation Land Requirements

To assess the impact of biostimulation storage on irrigation area requirements with future flows, water balance calculations for the months of July, August and September were completed. In this case, conditions in the three months cannot be considered independently. This is because the biostimulation storage volume will be accumulated before the start of irrigation and will be drawn down as irrigation proceeds. Any water not disposed of by the end of July would be carried over to August and, if necessary, September.

In preparing water balance calculations that span over several months, it is not appropriate to use independent 100-year precipitation values for each month, as that would imply that a 100-year July would be followed by a 100-year August and a 100-year September, which is not reasonable. Based on the 100-year precipitation amounts previously given for July, August, and September, the total obtained by simply summing these values would be 12.25 inches. Instead of doing this, it is necessary to do a separate statistical analysis of the total rainfall for the three months to determine the 100-year frequency for the total. Such a statistical analysis was done with assistance from DWR staff, indicating a 100-year rainfall amount for July through September of 5.96 inches. Therefore, if a 100-year rainfall of 5.43 inches were to be used for September, then the total rainfall in July and August would be 5.96 – 5.43 = 0.53 inches. Similarly, if the 100-year rainfall of 4.43 inches is assumed for July, then the total rainfall for August and September would be 1.53 inches. However, assuming the highest rainfall in

September is most conservative and is the basis of the calculations with 100-year precipitation considered below.

Assessing the impact of biostimulation storage on irrigation land area requirements is complicated by the need to specify the date upon which biostimulation storage would be filled and irrigation could begin. For the same reason that average flows should be used with 100-year return frequency precipitation, average or typical assumptions regarding biostimulation storage should be used in water balance calculations that incorporate 100-year return frequency precipitation. This is because it is presumed that there is no correlation between the timing of snowmelt in the spring, which impacts biostimulation storage, and the occurrence of summertime precipitation. Therefore, coupling conditions that would maximize biostimulation storage requirements together with 100-year return frequency summertime precipitation would result in an event with a lower probability of occurrence than once in 100 years.

In Figures 13-4 and 13-5 in Section 13, two charts are given to estimate when biostimulation storage would be filled and to what total volume. The chart in Figure 13-4 is based on a less conservative assessment of biostimulation storage requirements than the chart in Figure 13-5, as described in Section 13. Using a “typical” (not the most extreme) pattern shown in Figure 13-4, a maximum biostimulation storage volume of about 4.5 Mgal would be accumulated by about June 30. Similarly, using a typical pattern from Figure 13-5, a maximum biostimulation storage volume of about 8 Mgal would be accumulated by about July 15. Using these parameters, separate water balance calculations are shown in Tables 14-4 and 14-5, respectively. For the water balance in Table 14-5, only the second half of July is considered, since it was presumed that effluent would have gone to biostimulation storage until that time.

Table 14-4
Water Balance Calculations Based on Future Flows and Using Less Conservative Biostimulation Storage Assumptions (4.5 Mgal) with 100-Year Return Frequency Precipitation

Parameter	Month			
	July	August	September	Total
Input Data				
Influent Flow, Mgal/d	0.24	0.2	0.17	
Precipitation, Inches	0.265	0.265	5.43	5.96
Precipitation Effectiveness Factor	1	1	0.5	
Grass ET, Inches	8.99	7.75	5.7	22.44
Irrigation Efficiency, Percent	90	90	90	
Irrigation Area, Acres	36.1	36.1	36.1	
Beginning Volume in Storage, Mgal	4.5			
Calculations				
Total Influent Volume for Month, Mgal	7.44	6.2	5.1	
Maximum Possible Irrigation + Precipitation, Inches	9.99	8.61	6.33	24.93
Effective Precipitation, Inches	0.27	0.27	2.72	3.25
Irrigation, Inches	9.72	8.35	3.62	21.69
Effluent Disposal, Mgal	9.52	8.17	3.54	21.24
Net Volume Added to Storage, Mgal	-2.08	-1.97	1.56	
Ending Volume in Storage, Mgal	2.42	0.44	2.00	

Table 14-5
Water Balance Calculations Based on Future Flows and Using More Conservative Biostimulation Storage Assumptions (8 Mgal) with 100-Year Return Frequency Precipitation

Parameter	Month			
	July 16-31	**August**	**September**	**Total**
Input Data				
Influent Flow, Mgal/d	0.24	0.2	0.17	
Precipitation, Inches	0.1325	0.265	5.43	5.8275
Precipitation Effectiveness Factor	1	1	0.5	
Grass ET, Inches	4.495	7.75	5.7	17.945
Irrigation Efficiency, Percent	90	90	90	
Irrigation Area, Acres	46.3	46.3	46.3	
Beginning Volume in Storage, Mgal	8			
Calculations				
Total Influent Volume for Month, Mgal	3.84	6.2	5.1	
Maximum Possible Irrigation + Precipitation, Inches	4.99	8.61	6.33	19.94
Effective Precipitation, Inches	0.13	0.27	2.72	3.11
Irrigation, Inches	4.86	8.35	3.62	16.83
Effluent Disposal, Mgal	6.11	10.49	4.55	21.14
Net Volume Added to Storage, Mgal	-2.27	-4.29	0.55	
Ending Volume in Storage, Mgal	5.73	1.45	2.00	

For the water balance calculations, a precipitation effectiveness factor of 0.5 was used only with 100-year precipitation in September. For months with lesser precipitation, a value of 1.0 was used, indicating that all of the precipitation would be available to the vegetation (none would runoff and escape the runoff recovery system). An irrigation efficiency of 90 percent and September storage of 2 Mgal were used in accordance with previous discussions. It is noted that the 2 Mgal of storage would be in the emergency storage tank and Equalization Storage Tank 2. The biostimulation storage reservoir would be empty at the end of September, as desired to clean the reservoir and prepare for winter. The irrigation land areas indicated in the water balance calculations were determined by trial and error, as needed to yield the ending storage volume of 2 Mgal for September.

As indicated, using the less conservative assumptions on biostimulation storage, the total irrigation area requirement would be 36.1 Acres, while the requirement would be 46.3 Acres for the more conservative assumptions.

A water balance based on the more conservative assumptions regarding biostimulation storage (8 Mgal), which would result in about 11 Mgal stored at the end of July, is shown in Table 14-6. In this case, only August and September are considered in the water balance (all effluent is considered to be stored in July), and average precipitation amounts are used for these months. In this case, the irrigation area requirement is 53.2 Acres. Therefore, this scenario with the more conservative biostimulation storage volume (8 Mgal) and average precipitation is more severe than that with the less conservative biostimulation storage volume (4.5 Mgal) with 100-year precipitation.

Table 14-6
Water Balance Calculations Using the Most Severe Biostimulation Storage Assumptions with Average Precipitation

Parameter	Month		
	August	September	Total
Input Data			
Influent Flow, Mgal/d	0.2	0.17	
Precipitation, Inches	0.19	0.7	0.89
Precipitation Effectiveness Factor	1	1	
Grass ET, Inches	7.75	5.7	13.45
Irrigation Efficiency, Percent	90	90	
Irrigation Area, Acres	53.2	53.2	
Beginning Volume in Storage, Mgal	11		
Calculations			
Total Influent Volume for Month, Mgal	6.2	5.1	
Maximum Possible Irrigation + Precipitation, Inches	8.61	6.33	14.94
Effective Precipitation, Inches	0.19	0.70	0.89
Irrigation, Inches	8.42	5.63	14.05
Effluent Disposal, Mgal	12.16	8.14	20.30
Net Volume Added to Storage, Mgal	-5.96	-3.04	
Ending Volume in Storage, Mgal	5.04	2.00	

Based on the analyses presented above, future irrigation area requirements with biostimulation storage included range from about 36 acres to about 53 acres, depending on the assumptions and level of conservatism. Since it is estimated that approximately 34 acres of effective irrigation area is existing, up to 19 additional acres could be required. As discussed for the calculations in Section 14.3.1, these requirements should be confirmed during design.

14.3 IDENTIFICATION AND EVALUATION OF POTENTIAL IRRIGATION DISPOSAL SITES

Alternative sites considered for expanded effluent irrigation area are described below and shown on Figure 14-5. A field reconnaissance of all of these sites was completed on October 1, 2009. Where property ownership is referenced, it is based on data provided by the Assessor's Office of either Nevada County or Placer County (dependent upon the location of the parcel). References to potential effluent storage are included in certain site discussions below. Discussions of effluent storage options are presented in Section 13.

14.3.1 Irrigation Disposal Site No. 1 - Royal Gorge North Site

Site No. 1 is owned by Royal Gorge LLC and is in the same general location as Effluent Storage Site No. 4 (see Section 13). The 120 acre parcel (APN 047-010-13) is situated in Nevada County, approximately 1 mile southwest of the DSPUD WWTP, and is south of Interstate 80.

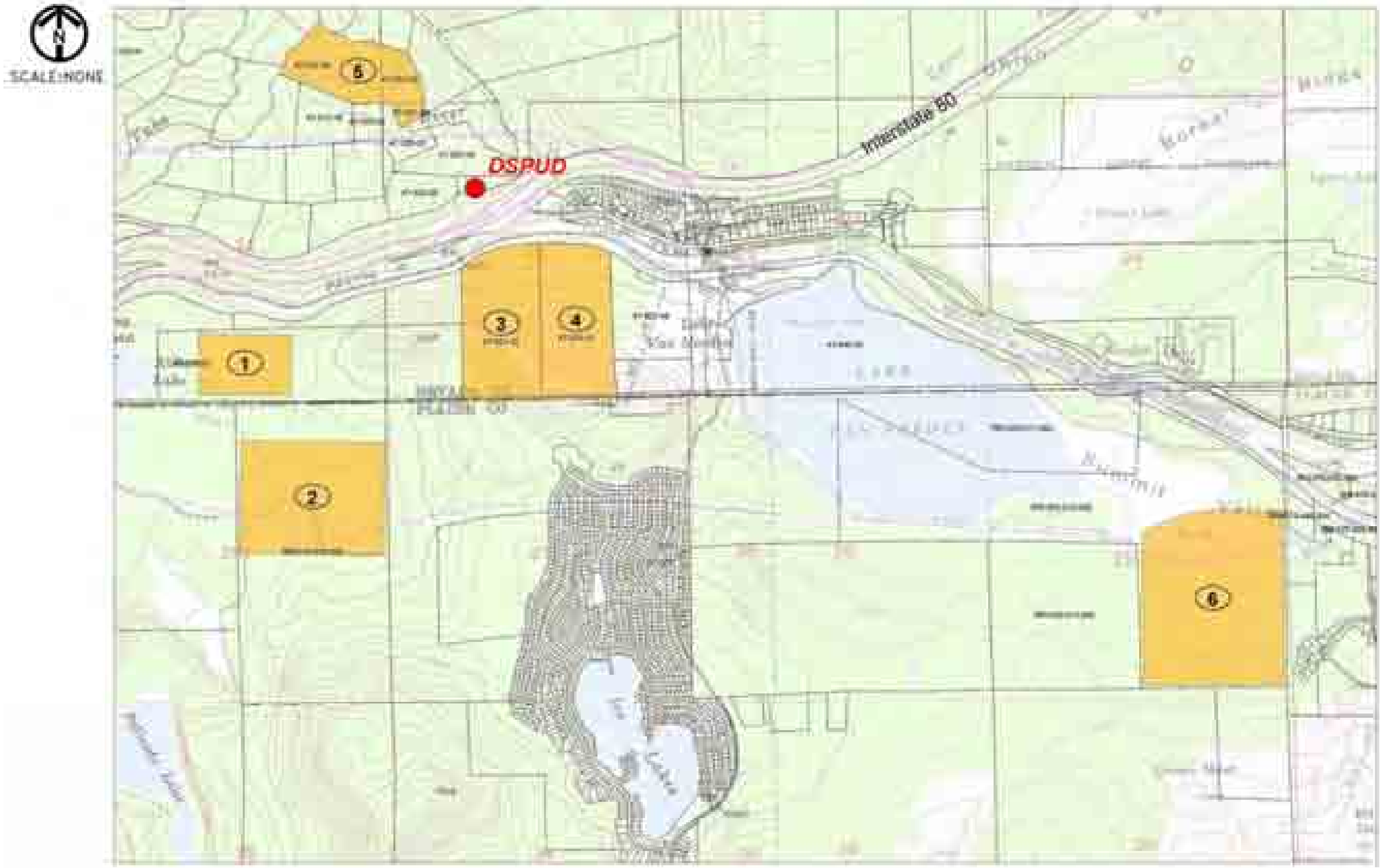


Figure 14-5
Potential Irrigation Disposal Sites Considered

The area of interest on Site No. 1 is the central western portion. This area has relatively flat terrain. The north-central portion of the site includes a gentle topographic depression, or “bowl.” Blackburn Consulting, Inc. (BCI) estimated between five and 10 feet of soil in this area. The bowl may be convertible into an effluent storage reservoir. The beneficial attributes of Site No. 1 are offset by its distance from the irrigation pump station at the DSPUD WWTP, and therefore the need for construction of a new, major effluent force main.

14.3.2 Irrigation Disposal Site No. 2 – Royal Gorge South Site

Site No. 2 is owned by Royal Gorge LLC. The 321 acre site is located on parcel APN 069-010-018 and is located in Placer County, approximately 1.2 miles southwest of the DSPUD WWTP, and south of Interstate 80.

The site is identified by BCI as being mostly hard rock at the surface with a wetland swale and electrical transmission lines through the central portion. From a soils and environmental perspective, this site is not desirable for effluent irrigation disposal.

14.3.3 Irrigation Disposal Site No. 3 – US Forest Service Parcel

Site No. 3 is owned by the US Forest Service. The 80 acre site is located on parcel APN 047-021-052 in Nevada County, approximately 0.2 miles south of the DSPUD WWTP, and is south of Interstate 80 and Donner Pass Road.

This site adjoins irrigation disposal Site No. 4, which is directly east of this parcel. This site is a north facing slope with moderate to very steep terrain. The site is a woodland dominated by red fir and lodgepole pines. It is very similar in character to Site No. 4 (discussed below) with the exception that Site No. 3 contains less wetland area, as mapped on the national wetlands inventory (for further discussion see section 16).

The site soil characteristics appear to resemble closely the existing irrigation disposal site at the Soda Springs Ski Area. The area has not been cleared recently. Site vegetation consists primarily of trees with minimal brush or other understory vegetation. This site would be an acceptable effluent irrigation area, once cleared, based on experience with the existing effluent irrigation area.

Site No. 3, by virtue of adjoining the existing Soda Springs irrigation area, would benefit from the existing irrigation pumping and conveyance infrastructure.

14.3.4 Irrigation Disposal Site No. 4 – Boreal Ridge Corporation Parcel

Site No. 4 is owned by the Boreal Ridge Corporation. The 82 acre site is located on parcel APN 047-021-051 and is located in Nevada County, approximately 0.3 miles south and east of the DSPUD WWTP. It is south of Interstate 80 and Donner Pass Road.

This site adjoins the existing DSPUD irrigation disposal site at Soda Springs, which is directly east of this parcel. This site is a north facing slope with moderate to very steep terrain. The site is a woodland dominated by red fir and lodgepole pines. It is very similar in character to the existing spray irrigation area, with the exception that Site No. 4 has not been cleared for ski/tubing runs. Site No. 4 also contains a wetland area that has been mapped on the national wetlands inventory (for further discussion see Section 16).

Like Site No. 3, Site No. 4 would benefit from the existing irrigation pumping and conveyance infrastructure.

14.3.5 Irrigation Disposal Site No. 5 – DSPUD Parcels

Site No. 5 consists of parcels owned by the District (APN 047-010-024, 047-021-002 and 047-021-003). These properties are located 0.1 to 0.2 miles north and west of the DSPUD WWTP, and are south of the South Yuba River. Portions of these properties were used in the past for effluent disposal via leach fields.

This site has a moderately steep north facing slope. The ground surface is mostly silty sand with scattered cobbles and boulders. The South Yuba River constitutes the northern, western and eastern boundaries of portions of these parcels.

This site appears to be slightly rockier than the existing disposal site or Sites No. 3 and 4. However, because these parcels are much larger than needed, sufficient area may be found to provide the needed disposal capacity, and still leave a generous buffer between the effluent irrigation areas and the South Yuba River.

14.3.6 Irrigation Disposal Site No. 6 – Sugar Bowl Parcel

Site No. 6 is owned by the Sugar Bowl Corporation. The 443 acre site is located on parcel APN 069-020-070 and is located in Placer County, approximately 2.5 miles south and east of the DSPUD WWTP, and is south of Interstate 80 and Donner Pass Road.

The western edge of this parcel includes portions of the old Lake Van Norden lake bed. A large electrical transmission facility bisects the parcel in an east-west direction. The portion of this parcel of interest for irrigation disposal is the portion south of these transmission facilities.

The area south of the transmission lines includes relatively flat to moderately steep terrain, which appears to be suitable for spray irrigation. It is north facing, and dominated by red fir trees in the most southerly portion.

14.3.7 Site Screening

The six potential sites for expanded effluent irrigation disposal were screened to determine if any obvious geotechnical or environmental fatal flaws exist that would preclude their use for the intended purpose.

The sites are all considered to be suitable for effluent irrigation disposal with appropriate clearing and construction of the necessary spray irrigation facilities and effluent runoff containment measures. By virtue of location alone, Site No 4 is considered to be the preferred location for additional sprinkler irrigation. The potential for wetlands on the site is considered a drawback, but appropriate sprinkler system location, operations and wetland mitigation, if needed, should be adequate to make portions of this site acceptable for use. A further advantage with Site No. 4 is that the District reports an agreement is already in place with the parcel owner, allowing for effluent disposal on the property.

Sites No. 3 and No. 5 are ranked closely behind Site No. 4, also by virtue of location. Site No. 3 would benefit from existing irrigation pumping and conveyance facilities. However, Site No. 5 is within properties already owned by the District and transmission of effluent could presumably be facilitated via the existing outfall pipeline, which passes through these properties.

14.3.8 RECOMMENDATION

As developed previously in this section, the need for additional irrigation disposal area is contingent upon the need for biostimulation storage. At this time, DSPUD has not concluded that biostimulation storage is necessary. If it is determined that biostimulation storage and irrigation disposal area expansion are needed, DSPUD should evaluate Site No. 4 further. This includes delineation of potential wetlands and the evaluation of soil characteristics to estimate with more certainty the disposal potential of the site. Evaluation of potential impacts to underlying groundwater will also be required. Given the apparent bedrock underlying the area, it is likely that groundwater will not be impacted by effluent percolation. However, this issue will need to be addressed prior to the Regional Water Board authorizing expanded effluent disposal. The CEQA document for the preferred project resulting from this Facilities Plan will need to address this and related potential impacts. If Site No. 4 is determined to be lacking in disposal capacity after further assessment, the deficit could potentially be solved by utilizing a portion of Site No. 3, also. Alternatively, Site No. 5 could be used solely or as a supplement to Site No. 4. In any case, all sites considered would have to be more extensively studied during preliminary design.

14.4 IRRIGATION FACILITIES IMPROVEMENTS

At the present time, DSPUD has facilities in place to deliver 600 gpm of treated effluent to the irrigation system at the Soda Springs Ski Area. The existing pumping and conveyance system can be used, not only to supply the existing irrigation system, but also the potential expansion area, if the preferred Site No. 4 (or Site No. 3) is confirmed. Based on the worst-case scenario represented by the water balance in Table 14-6, 12.16 Mgal of effluent would have to be pumped for irrigation during the month of August. This would require operation of the existing 600 gpm pumping system on average about 10.5 hours per day, every day, including weekends. Since these facilities are automatically controlled by irrigation timers and related equipment, this should be acceptable. Therefore, no expansion of the pumping and conveyance system is currently anticipated. However, during design, if desired, options for increasing the pumping flow rate and decreasing the operating time can be considered.

The estimated construction cost for preparing the land and installing irrigation and runoff recovery facilities similar to those existing at the Soda Springs Ski Area is $30,000 per acre (including all appropriate markups and a 20 percent contingency allowance). Allowing 25 percent for engineering, administration and environmental review, the total capital cost would be $37,500 per acre. Therefore, the total capital cost for adding 19 acres of irrigation area immediately adjacent to the existing facilities is about $700,000.

Section 15

Biosolids Dewatering and Disposal

Section 15

Biosolids Dewatering and Disposal

In Section 9, various methods for biological treatment of the DSPUD wastewater are investigated. All of these biological treatment methods produce residual solids, typically referred to as sludge or biosolids, which must be processed and hauled away from the wastewater treatment plant. One of the treatment methods investigated in Section 9 (submerged attached growth) includes a chemically-enhanced, high-rate primary clarification process. This process involves the addition of chemicals that form precipitation products and result in the production of chemical sludge, which must be handled together with the biosolids produced during biological treatment.

In this section, the quantities of sludge produced by the various treatment methods are quantified and several options for handling these solids are investigated. First, however, it is helpful to consider the existing residual solids handling facilities and methods.

15.1 EXISTING CONDITIONS

The DSPUD wastewater treatment plant currently includes an integrated fixed-film activated sludge (IFAS) biological treatment system, which produces waste activated sludge (WAS). The WAS includes influent solids that do not get degraded in the process, as well as residual biological solids developed in the mixed liquor and sloughed from the fixed-film support media.

WAS is pumped from the underflows of the two secondary clarifiers to a 600,000 gallon solids storage tank. The tank is minimally aerated to provide limited aerobic digestion of the biosolids and is periodically decanted to reduce the quantity of water held in the tank with the solids. The partially digested liquid sludge is accumulated in the tank during the months when dewatering on sludge drying beds is not possible. In late spring and summer, most of the tank contents are dosed in batches onto sand drying beds.

There are four sand drying beds, each 82 feet long and 31.5 feet wide. The design criteria in the May 1985 construction drawings indicate 10,400 square feet of area at an annual loading rate of 6.4 pounds per square foot. This results in a design annual load of approximately 66,500 pounds of solids to dewater.

The operator's typical practice is to load 7-9 inches of liquid biosolids at a concentration of approximately one percent solids to the beds. Polymer is added to the solids at a rate of 12 pounds per dry ton to increase flocculation and drainage into the sand. Reportedly, dried solids typically can be removed from the beds in as little as 7 to10 days at a dryness of 50 to75 percent solids.

The dewatered solids are loaded into 20 cubic yard drop boxes and picked up by Tahoe Truckee Sierra Disposal at a cost of $500 per trip. Each container reportedly has a net weight of 8.5 tons of wet solids. This results in a hauling and disposal cost of approximately $59 per wet ton. The solids are disposed at the Lockwood Landfill in Sparks, Nevada, owned and operated by Waste Management Company.

For the standard tipping rate, materials accepted at Lockwood must pass the paint filter test, which typically corresponds to at least 15 percent solids content. Since the DSPUD waste solids are typically over 50 percent solids, this is not a problem. The landfill charge rate is currently $16.50 per cubic yard and waste biosolids can be accepted year-round. This assumes a range of 50 to 300 wet tons per year delivered by the District. The District currently delivers approximately 60 wet tons of biosolids per year. The actual dry solids content is not specifically determined.

15.2 FUTURE SOLIDS HANDLING ALTERNATIVES

The following solids handling alternatives are investigated in this section:

- Continued use of the existing solids storage tank (with modifications) and sand drying beds, according to existing practices.
- Construction of a new (smaller) aerated solids holding tank and mechanical dewatering system, making the existing 600,000 gallon solids storage tank available for other uses. The mechanical dewatering alternatives include:
  - Belt Press
  - Centrifuge
  - Screw Press

For the mechanical dewatering alternatives, the proposed size of the aerated solids holding tank is determined to provide a minimum solids retention time of about 20 days, plus emergency storage for residual solids, in the event of a failure of the mechanical dewatering equipment. The sizing of this tank and of the dewatering facilities will depend on the characteristics of the residual solids streams to be handled, which will depend on the biological treatment alternatives being considered. Residual solids quantities before and after solids processing for the various biological treatment alternatives and solids handling options are developed in the subsection below, followed by detailed descriptions and evaluations of the solids handling options.

In all cases, it is assumed that dewatered solids will continue to be disposed of in the Lockwood Landfill (or alternative landfill). If it were desired to beneficially use the biosolids for agricultural purposes, the solids would have to be processed in accordance with the corresponding requirements of the EPA Part 503 Biosolids Rule (40 CFR, Part 503). Under this rule, for agricultural land application, the biosolids would have to be processed to meet at least Class B pathogen destruction. For aerobic digestion, the minimum solids retention times specified in the rule range from 40 days at 20 degrees C to 60 days at 15 degrees C. No specific solids retention time is acceptable for temperatures lower than 15 degrees C, which would prevail for much of the winter at DSPUD. In such cases, adequate pathogen reduction must be demonstrated by testing every load of biosolids hauled from the site to indicate a fecal coliform content of less than 2 million MPN per gram of solids. Additionally, the biosolids must be demonstrated to meet minimum standards for vector attraction reduction, which basically means that the solids cannot contain any volatile solids that are not well-digested and remain significantly putrescible.

It is possible, but it is not suggested as probable, that the solids held through the winter in the 600,000 gallon solids holding tank could meet the requirements for agricultural land application. However, the solids from the small digester being considered under the mechanical dewatering options most certainly would not be suitable for agricultural land application. If agricultural land application should ever be desired, specific steps would have to be taken to assure adequate stabilization. With the small digester and mechanical dewatering options, additional stabilization, perhaps involving lime treatment and heating would be required.

15.3 FUTURE RESIDUAL SOLIDS PRODUCTION

As developed in Section 4, the average annual BOD (and TSS) load in the wastewater treatment plant influent is expected to increase from about 215 lb/d under current conditions to about 285 lb/d under future design conditions, an increase of about 33 percent. Month-by-month average flows and influent BOD loads for the future design condition were estimated based on historical patterns and are shown in Table 15-1. Also shown in Table 15-1 are the month-by-month projections of residual solids expected from the various biological treatment alternatives considered in Section 9 and the reductions in those solids amounts expected under the two aerobic digestion alternatives. Projected monthly sludge volume accumulations in the 600,000 gallon tank, if used, are also shown. Finally, the net residual solids that would be dewatered each month under the various solids handling options considered later in this section are shown. The methods for determining the residual solids amounts and sludge volumes are discussed below.

15.3.1 RESIDUAL SOLIDS FROM PRIMARY AND SECONDARY TREATMENT

For each biological treatment alternative, the amount of BOD entering the secondary treatment system (same as influent BOD, except for the submerged attached growth option) was determined and used to calculate a baseline amount of residual solids, based on typical standards for net sludge yields at the appropriate temperatures and process conditions involved. The estimated amounts of methanol (or alternative carbon source) addition required for each specific process were determined on a month-by-month basis and the incremental sludge yields from methanol addition were estimated.

For the submerged attached growth alternative, sludge production characteristics would be substantially different than the other alternatives, due to the presence of the chemically-enhanced primary treatment system. It is expected that approximately 80 percent of the influent TSS and 50 percent of the influent BOD would be removed during primary treatment. This reduces the loading on the secondary process and, therefore, reduces the amount of residual solids developed in the secondary process. However, the BOD and TSS removed in the primary clarification step would get routed to the aerobic digester to join the secondary sludge.

Table 15-1
Monthly Solids Production and Fate for All Alternatives

Parameter	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Average	Total
Average Influent Flow, Mgal/d	0.15	0.16	0.31	0.33	0.37	0.38	0.39	0.38	0.27	0.24	0.20	0.17	0.28	---
Average Influent BOD Load, lb/d	129	132	434	420	526	401	260	149	208	304	288	169	285	---
Average Sludge Production, lb/d														
MBR (Total, All Biological)	123	177	398	332	390	292	191	112	151	213	201	123	225	---
IFAS (Total, All Biological)	117	167	377	321	379	281	182	105	146	213	201	118	217	---
Subm. Attach. Grwth														---
Biological (Net, Including Digester)	117	167	377	321	379	281	182	105	146	213	201	118	217	---
Chemical	35	36	121	116	146	110	69	37	56	84	80	46	78	---
Total	152	203	497	437	525	391	251	142	201	297	282	164	295	---
Average VSS Destruction, lb/d														
Exist. 600,000 Gallon Tank														
MBR	40	49	77	109	122	126	119	106	131	196	150	82	109	---
IFAS	38	46	73	103	117	122	114	101	125	190	147	80	105	---
Subm. Attach. Grwth	38	46	73	103	117	122	114	101	125	190	141	74	104	---
New Digester														
MBR	54	62	71	62	63	62	59	41	56	79	76	57	62	---
IFAS	51	59	67	59	59	58	56	38	53	77	74	55	59	---
Subm. Attach. Grwth	49	60	67	57	57	55	54	37	51	74	71	50	57	---
End Month Vol. in 600,000 Gal. Tank, Mgal														
MBR	0.12	0.15	0.23	0.29	0.35	0.39	0.40	0.41	0.41	0.30	0.20	0.10	0	---
IFAS	0.12	0.15	0.22	0.28	0.34	0.38	0.39	0.39	0.40	0.29	0.20	0.10	0	---
Subm. Attach. Grwth	0.13	0.17	0.27	0.35	0.44	0.51	0.54	0.55	0.57	0.41	0.26	0.10	0	---
Solids to Dewatering, lb/Month														
Exist. 600,000 Gallon Tank														
MBR	0	0	0	0	0	0	0	0	0	14,079	14,079	14,079	---	42,236
IFAS	0	0	0	0	0	0	0	0	0	13,579	13,579	13,579	---	40,738
Subm. Attach. Grwth	0	0	0	0	0	0	0	0	0	23,170	23,170	23,170	---	69,511
New Digester														
MBR	2,142	3,444	10,129	8,374	9,173	7,143	3,960	2,206	2,873	4,161	3,898	1,970	---	59,473
IFAS	2,033	3,245	9,587	8,111	8,956	6,894	3,774	2,057	2,770	4,200	3,938	1,910	---	57,476
Subm. Attach. Grwth	3,183	4,291	13,325	11,775	13,122	10,411	5,909	3,241	4,501	6,899	6,514	3,440	---	86,611

In the aerobic digester, the BOD and TSS would undergo biological treatment, and it is estimated that the overall net effect would be equal sludge production from influent BOD and TSS, as compared to the integrated fixed-film activated sludge (IFAS) alternative. For the submerged attached growth alternative, however, additional chemical sludge would be derived from the chemically-enhanced primary clarification process. The chemical sludge production was estimated based on an average assumed ferric chloride dose equal to 25 percent of the influent TSS concentration. Chemical sludge quantities were estimated based on the production of phosphate and hydroxide precipitation products (combined with iron) and an allowance for an additional 35 percent chemical sludge due to unspecified mechanisms (typical for such systems).

15.3.2 Residual Solids After Digestion and Dewatering

The amount of solids destroyed during aerobic digestion and the amount of residual solids to be dewatered were determined differently for the two aerated solids holding tank options, as described below.

Continued Use of the Existing 600,000 Gallon Tank

For this option, solids would be accumulated in the tank from the last sludge drying bed use at the end of the summer to the first sludge drying bed use at the beginning of the following summer. However, a substantial portion of the volatile suspended solids (VSS) introduced into the tank would be destroyed through aerobic digestion. The rate of VSS destruction is dependent on time and temperature. The EPA Process Design Manual on Sludge Treatment and Disposal includes a graph that can be used to estimate the fraction of VSS destroyed as a function of the number of degree-C-days (number of days multiplied by the temperature, or the sum of the daily temperatures for the solids retention time). The slope of the VSS destruction curve is very steep initially, indicating a high rate of VSS destruction during the initial days in the digester, but then the curve flattens out as additional VSS destruction becomes slower with extended digestion time. Using this curve as a basis, for each biological treatment option, the fate of solids delivered to the tank and the month-by-month solids inventories and liquid volumes in the tank were determined as follows:

1. At the beginning of October, the tank was assumed to be at a minimum level suitable for mixing and aeration, containing residual digested solids from the previous year.

2. The new solids deposited in the tank in October were tracked to determine incremental VSS destruction amounts on a month-by-month basis, depending on the number of degree-C-days in each month, through the following summer.

3. The new solids deposited in the tank in each of the succeeding months were similarly and separately tracked to determine incremental VSS destruction each month from the date of input to the tank through the following summer.

4. The total solids destruction each month was determined as the summation of the incremental solids destructions resulting from the separate inputs of all of the previous months.

5. The total solids remaining in the tank each month was determined as the total solids deposited in the tank in all preceding months minus the total solids destroyed to that time.

6. The volume of liquid sludge in the tank each month was determined assuming a solids concentration of 1.5 percent after decanting.

7. As additional solids were being added and destroyed, solids were removed from the tank at a constant rate during July, August and September, as needed to return to the beginning tank volume on October 1.

As noted in the procedure described above, it was assumed that solids were sent to the sludge drying beds only during July through September. This is intended to represent a worst-case scenario, resulting in the maximum volume accumulation in the tank at the end of June. If dewatering was assumed to occur in June, then the May volume would be the maximum. In either case, the total amount of solids to be dewatered and hauled away would be essentially the same.

As noted in Table 15-1, the maximum volume accumulated in the solids storage tank was always less than 600,000 gallons, indicating that the existing tank would be adequate for this use.

Use of a New Smaller Digester with Mechanical Dewatering

Under this option, a new, smaller aerated solids holding tank (aerobic digester) would be constructed and would receive the residual solids from wastewater treatment. The solids would be held in the tank for a minimum solids retention time of 20 days, during which substantial volatile solids destruction would occur, depending on the temperature in the tank. As new residual solids are wasted to the tank each month, a corresponding amount of digested solids would have to be removed, dewatered, and hauled away to a landfill for disposal.

The calculations under this option were much simpler and more straightforward as compared to the previous option. Each month, the monthly solids input was determined according to the residual solids production for the biological treatment option being considered. The solids retention time and associated VSS destruction amounts for each month were calculated through an iterative process, leading to the amount of remaining solids that must be dewatered in the month. The required digester volume was determined as the volume that would give the minimum 20-day solids retention time in all months.

15.4 ALTERNATIVE ANALYSIS

Capital, annual, and total present worth costs for the various alternatives are shown in Table 15-2. The bases of the estimates are discussed in the following subsections.

Table 15-2
Alternative Cost Analysis

Biological Treatment Alt. / Digester Alt. / Dewatering Alt.	MBR				IFAS (Upgrade or New)				Submerged Attached Growth			
	Exist Beds	New Belt	New Cent.	New Screw	Exist Beds	New Belt	New Cent.	New Screw	Exist Beds	New Belt	New Cent.	New Screw
Capital Costs												
Modify Existing Solids Tank	232,000				232,000				232,000			
New Digester and Ancillary		285,000	285,000	285,000		285,000	285,000	285,000		315,000	315,000	315,000
Sludge Dewatering and Related Equipment,		425,000	542,500	391,000		425,000	542,500	391,000		425,000	542,500	391,000
Sludge Dewatering Building		230,000	200,000	200,000		230,000	200,000	200,000		230,000	200,000	200,000
Subtotal 1	232,000	940,000	1,027,500	876,000	232,000	940,000	1,027,500	876,000	232,000	970,000	1,057,500	906,000
Sitework @ 5% of Subtotal 1	NA	47,000	51,000	44,000	NA	47,000	51,000	44,000	NA	49,000	53,000	45,000
Site Piping @ 10% of Subtotal 1	NA	94,000	103,000	88,000	NA	94,000	103,000	88,000	NA	97,000	106,000	91,000
Electrical/Instrum. @ 25% of Subtotal 1	58,000	235,000	257,000	219,000	58,000	235,000	257,000	219,000	58,000	243,000	264,000	227,000
Subtotal 2	290,000	1,316,000	1,438,500	1,227,000	290,000	1,316,000	1,438,500	1,227,000	290,000	1,359,000	1,480,500	1,269,000
General Conditions, Overhead and Profit, 20%	58,000	263,000	288,000	245,000	58,000	263,000	288,000	245,000	58,000	272,000	296,000	254,000
Subtotal 3	348,000	1,579,000	1,726,500	1,472,000	348,000	1,579,000	1,726,500	1,472,000	348,000	1,631,000	1,776,500	1,523,000
Contingency, 20%	70,000	316,000	345,000	294,000	70,000	316,000	345,000	294,000	70,000	326,000	355,000	305,000
Total Construction Cost	418,000	1,895,000	2,071,500	1,766,000	418,000	1,895,000	2,071,500	1,766,000	418,000	1,957,000	2,131,500	1,828,000
Engineering, Admin, Environmental, 25%	105,000	474,000	518,000	442,000	105,000	474,000	518,000	442,000	105,000	489,000	533,000	457,000
Total Capital Cost	523,000	2,369,000	2,589,500	2,208,000	523,000	2,369,000	2,589,500	2,208,000	523,000	2,446,000	2,664,500	2,285,000
Annual Costs												
Labor	24,600	14,500	14,700	13,000	24,100	14,400	14,600	13,000	32,700	15,600	16,000	13,400
Power	14,300	5,700	6,300	5,900	13,800	5,500	6,000	5,600	18,400	5,500	6,300	5,800
Polymer	1,300	1,800	3,000	2,200	1,200	1,700	2,900	2,200	2,100	2,600	4,300	3,200
Hauling and Disposal	2,100	9,700	8,800	10,300	2,000	9,400	8,500	10,000	5,100	14,200	12,800	15,000
Maintenance	14,500	65,800	71,900	61,400	14,500	65,800	71,900	61,400	14,500	68,000	74,000	63,500
Total Annual Cost	56,800	97,500	104,700	92,800	55,600	96,800	103,900	92,200	72,800	105,900	113,400	100,900
Present Worth of Annual Costs (b)	845,000	1,451,000	1,558,000	1,381,000	827,000	1,440,000	1,546,000	1,372,000	1,083,000	1,576,000	1,687,000	1,501,000
Total Present Worth Cost	1,368,000	3,820,000	4,147,500	3,589,000	1,350,000	3,809,000	4,135,500	3,580,000	1,606,000	4,022,000	4,351,500	3,786,000

Cost for Indicated Alternative (a), $

(a) In first-quarter 2010 dollars, ENR 20-Cities CCI = 8700.

(b) 20 years at inflation-adjusted discount rate of 3 percent, Present Worth Factor = 14.88.

15.4.1 CONTINUED USE OF THE EXISTING SOLIDS HOLDING TANK AND SAND DRYING BEDS

Under this option, the existing solids holding tank would be retrofitted with a new aeration and mixing system and a new solids excluding decanter. No other improvements would be required.

Based on discussions with the plant manager, the existing sludge drying beds are more than adequate to handle existing solids production from the wastewater treatment plant and should have no problem accommodating the projected future loads for the options that do not include chemical sludge. For the submerged attached growth biological treatment alternative, which does include chemical sludge, it is presumed that the performance of the sludge drying beds would be reduced. Final dewatered sludge solids contents were assumed to be 60 percent for all biological treatment alternatives, except the submerged attached growth alternative, for which 40 percent solids was assumed.

15.4.2 NEW DIGESTER AND BELT PRESS DEWATERING

For this option, a new digester would be required. The volume of the digester would depend on the biological treatment option as indicated in Table 15-3 below:

Table 15-3
Biological Treatment Option Volumes

	MBR	IFAS	Submerged Attached Growth
Active Digestion Volume, gal	52,000	50,000	70,000
Emergency Sludge Storage Volume, gal [(a)]	37,000	36,000	57,000
Total Volume, gal	89,000	86,000	127,000

(a) Two weeks at maximum input.

A covered steel tank with a jet aeration system and solids excluding decanter would be recommended. The pumps and blowers for the jet aeration system would be located in an adjacent sludge dewatering building.

In a belt filter press, sludge is dewatered by two different methods. First, after dosing with polymer, the flocculated sludge is placed on a moving belt upon which gravity drainage occurs (water drains through the belt, while solids are retained on the belt surface). Second, the partly digested sludge is sandwiched between two moving belts that are routed through a series of rollers that apply increasing pressure to squeeze water from the solids. The dewatered solids are then scraped off the belts and conveyed to a bin for hauling away.

An example of a belt filter press installation is shown in Figure 15-1.

Figure 15-1
Belt Press, Courtesy of BDP Industries

For DSPUD, a one meter belt press with a capacity of 200 pounds dry solids per hour is assumed. At one percent feed solids this would be approximately 40 gpm, producing a cake dryness of about 18 percent. The facility would include a press, polymer feed system, feed pump, cake conveyor and building. The press would be operated during normal plant hours and would require operator observation every few hours.

15.4.3 New Digester and Centrifuge Dewatering

For this option, a new digester would be required and would be identical to that indicated for the belt press dewatering option.

In a centrifuge, liquid sludge dosed with polymer is fed into a high-speed rotating vessel called a bowl. Solids are compacted to the outside of the bowl, while the water is extracted from above the sludge cake. The sludge cake is pushed out of the rotating bowl by means of a spiral conveyor, or scroll. A cutaway view of a centrifuge is shown in Figure 15-2.

A centrifuge facility would include a 50 gpm centrifuge, a feed pump, a polymer system, cake conveyor, and building of sufficient size. With the proper controls and alarms, it would be possible to operate unattended overnight. However, most facilities operate only during staffed hours, which is the assumption for DSPUD.

It is estimated that the centrifuge will produce 20 percent cake solids, which is the driest of the mechanical dewatering options. However, the centrifuge will also use the most polymer.

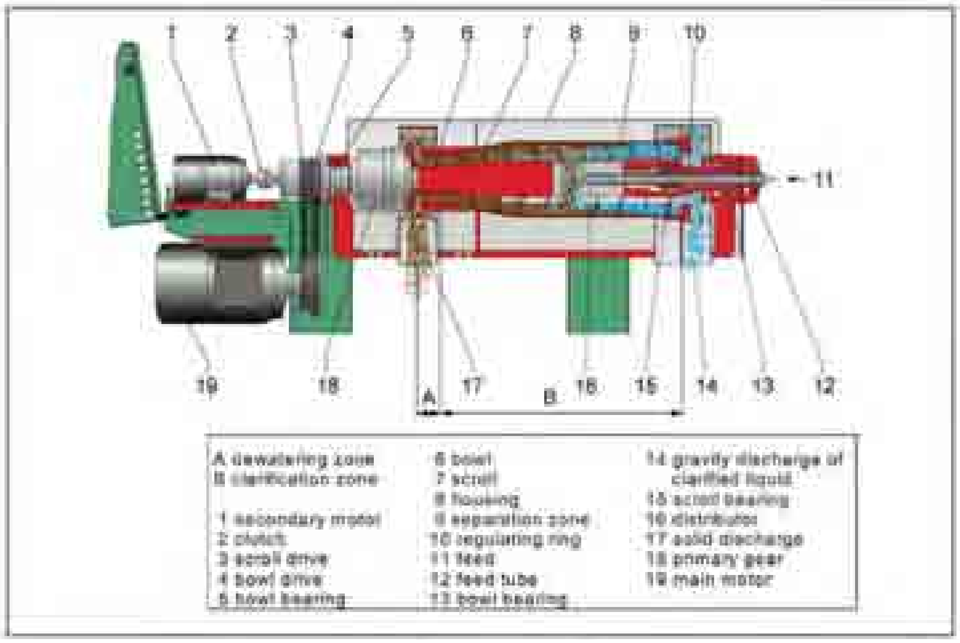


Figure 15-2
Centrifuge Features, Courtesy GEA Westfalia Separator, Inc.

15.4.4 NEW DIGESTER AND SCREW PRESS DEWATERING

For this option, a new digester would be required and would be identical to that indicated for the belt press dewatering option.

A screw press is a dewatering machine wherein a screw rotates extremely slowly in a drainage vessel, or basket. Polymer-dosed sludge is fed into the basket where dewatering occurs by gravity drainage and by compression forces imparted by the screw. The screw presses the sludge into ever decreasing volumes as it moves through the machine to the point of discharge.

A conceptual rendering of a screw press is shown in Figure 15-3.

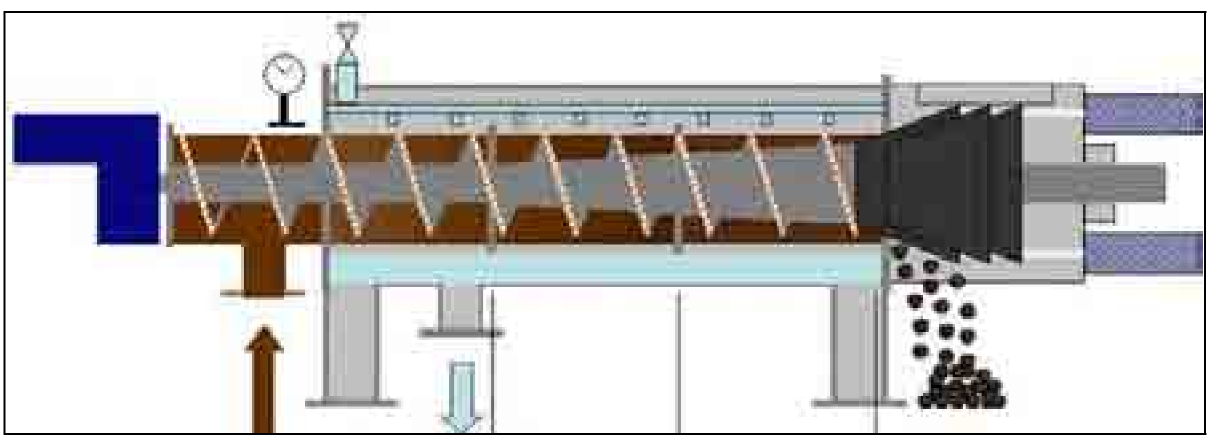


Figure 15-3
Conceptual Rendering of a Screw Press, Courtesy Huber Technology, Inc.

A screw press facility would include a 20 gpm press, a feed pump, a polymer system, cake conveyor, and building of sufficient size. With the proper controls and alarms it is possible to operate unattended overnight. Most screw press facilities operate 24 hours on a dewatering day, which is assumed for DSPUD.

The screw press will produce cake at approximately 17 percent and will use polymer about equal to the belt filter press.

15.5 RECOMMENDED IMPROVEMENTS

As shown in Table 15-2, regardless of which biological treatment and mechanical sludge dewatering options are chosen, the total present worth costs of the sludge handling options with mechanical dewatering are about $2 Million greater than if the existing sludge storage tank and drying beds are continued in use. Of course, with the mechanical dewatering options, the sludge storage tank could be used for another purpose, such as influent flow equalization. However, the net present worth cost benefit of using the existing sludge storage tank for influent equalization instead of building a new tank is only about $500,000 (see Table 8-2 in Section 8). Therefore, it does not make sense to install mechanical sludge dewatering.

Based on the analysis presented above, the District should install a new mixing and aeration system and a new decanter in the existing sludge storage tank and continue using this tank together with the existing sludge drying beds as the preferred future solids handling alternative.

Section 16

Preliminary Environmental Analysis

Section 16

Preliminary Environmental Analysis

ECO:LOGIC's environmental team conducted a preliminary desktop environmental review of each treated effluent storage reservoir site, tank site, and effluent irrigation disposal site alternative to identify possible environmental issues or environmental "fatal flaws" that could threaten the viability of the proposed improvements. For example, wetlands, historical and sensitive cultural resources, and endangered species habitat could represent fatal flaws or time and cost constraints. The preliminary desktop environmental review follows the analyses presented in the Facilities Plan for treated effluent storage, tank, and effluent irrigation disposal alternatives outlined in Section 13 Seasonal Storage to Mitigate Biostimulation in the South Yuba River and Section 14 Effluent Irrigation Disposal.

For all relevant aspects of the preferred project identified in the Facilities Plan, a biologist and cultural resource specialist conducted a reconnaissance-level survey to identify any sensitive environmental resources. All sensitive environmental resources were mapped using sub-meter Trimble GPS and input into a GIS database, where feasible, for the project. In developing the Preliminary Environmental Analysis for the Facilities Plan, ECO:LOGIC's environmental resource specialists worked closely with the project engineers to recommend minor adjustments and solutions, where feasible, that would minimize potential impacts to the environment. Based on the results of the desktop preliminary environmental assessments, including database searches, and site surveys for each alternative, the results of the Preliminary Environmental Analysis for the Facilities Plan was developed below and a determination was made for each project alternative if any environmental "fatal flaw" exists.

If an environmental "fatal flaw" was discovered during the background research and during field surveys, another alternative was selected. However, environmental "fatal flaws" may only determine that the selection of any alternative or site may lead to additional time and costs due to permitting and potential mitigation costs for that alternative and may not be an overriding determination through a cost/benefit analysis that a particular site may still be the preferred alternative due to engineering considerations and/or other cost savings related to design and construction.

16.1 BIOLOGICAL RESOURCES DATABASE SEARCHES

As part of the desktop preliminary environmental assessment, extensive background research was conducted for each project alternative and for each potential site. Background research included evaluating existing information related to public CEQA documents for similar projects in the area and similar projects in the Sierra foothills, evaluating existing topographic and aerial photography data, and considering the results of several database searches.

The following databases were searched for each alternative to determine if any state or federally listed species or protected wetlands are known to occur:

- California Natural Diversity Database (CNDDB, 2009)
- National Wetland Inventory (NWI)
- United States Fish and Wildlife Service (USFWS) list of Threatened and Endangered Species (USFWS, 2009)
- California Native Plant Society (CNPS) electronic inventory of rare plants (CNPS, 2009)
- Soils Surveys for Nevada and Placer Counties (NRCS)

The results of the database searches are detailed below and in several figures documenting known locations of special-status plant and fauna species and wetlands in the areas under consideration in this Wastewater Treatment and Disposal Facilities Plan. Database lists generated from the CNDDB, USFWS, and CNPS are included in Appendix E.

The CNDDB was searched for special-status plant species within 10 miles of the DSPUD WWTP and the results are presented in Figure 16-1. The CNDDB was searched for special-status fauna species within 10 miles of the DSPUD WWTP and the results are presented in Figures 16-2.

Special-status plant and fauna species in proximity to the alternative disposal, tank, and storage sites is documented in Figures 16-3. As opposed to Figure 16-1 and Figure 16-2 which indicate known locations of special-status species in and around Donner Summit, Figure 16-3 indicates known locations of special-status species within and adjacent to the alternative sites. Several alternatives and sites contain mapped wetlands on the NWI (Figure 16-4). The NWI includes mapped locations (from the 1970's) of potential wetlands throughout the United States and can indicate the presence of wetlands such as lakes, streams, wet meadows, freshwater and saltwater marshes, etc.; however, the criteria used by the USFWS to document wetlands within the NWI follows the Cowardin *et al.* (1979) criteria. The Cowardin *et al.* (1979) classification system (based on aerial photography or digital aerial imagery) is not the same criteria used by the U.S. Army Corps of Engineers (Corps) to identify jurisdictional wetlands and other waters of the U.S. Therefore, the presence of wetlands on NWI maps does not automatically indicate that each mapped wetland will meet the criteria set by the Corps, which would be regulated under the Clean Water Act Sections 404 and 401.

The absence of a mapped wetland on the NWI also does not preclude the presence of a wetland meeting the Corps criteria on a given site; therefore, a formal wetland delineation is always recommended once a preferred alternative site is selected to verify the presence of jurisdictional wetlands and to determine whether Clean Water Act Sections 404 and 401 permitting will be required. Mitigation fees for impacts to some types of wetlands regulated by the Corps under the Clean Water Act Section 404 can cost between $150,000 to $400,000 per acre, so working closely with project engineers to try and avoid impacts to jurisdictional wetlands is recommended.

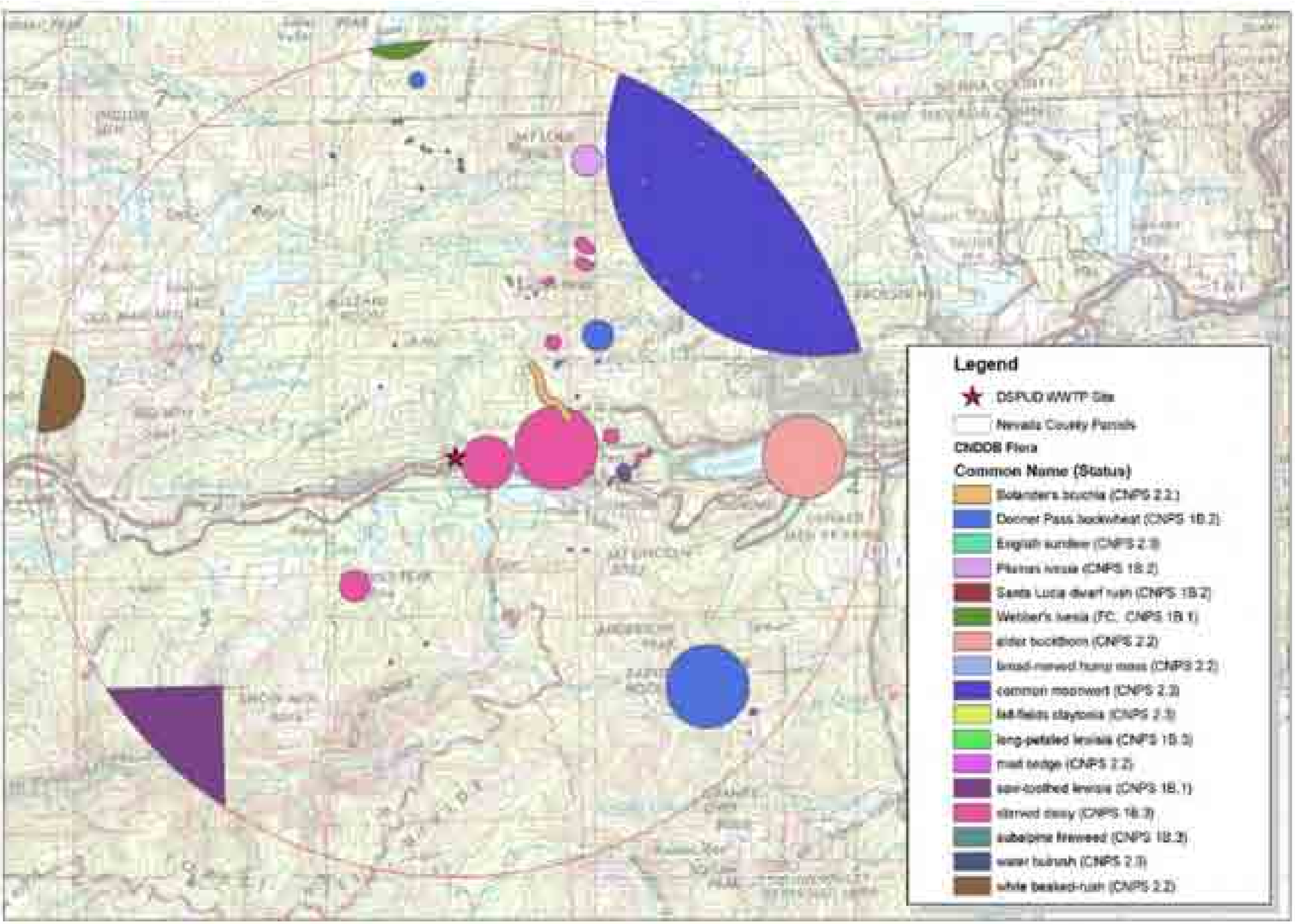


Figure 16-1
Special-Status Plant Species Known to Occur Within Ten Miles of the DSPUD WWTP

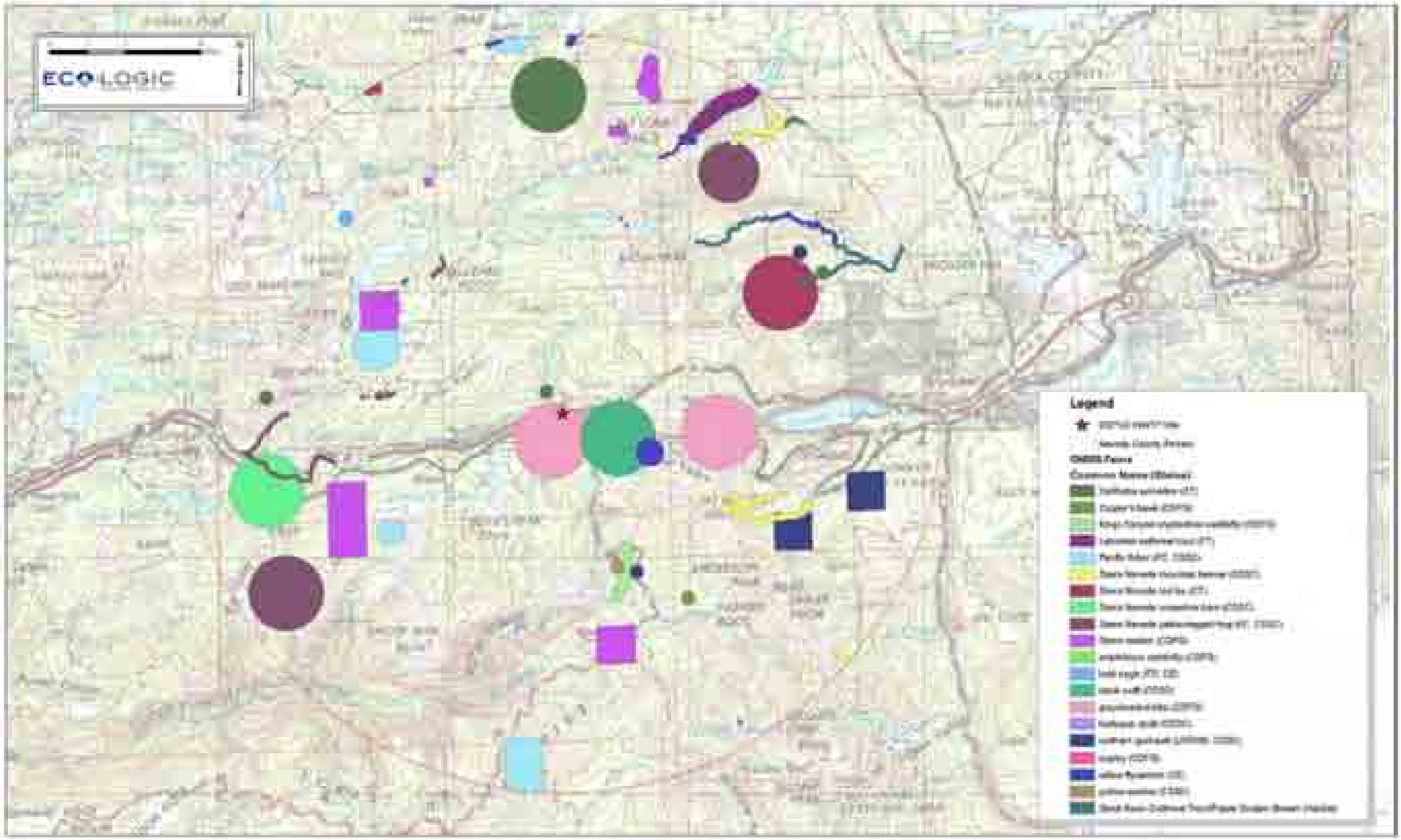


Figure 16-2
Special-Status Fauna Species Known to Occur Within Ten Miles of the DSPUD WWTP

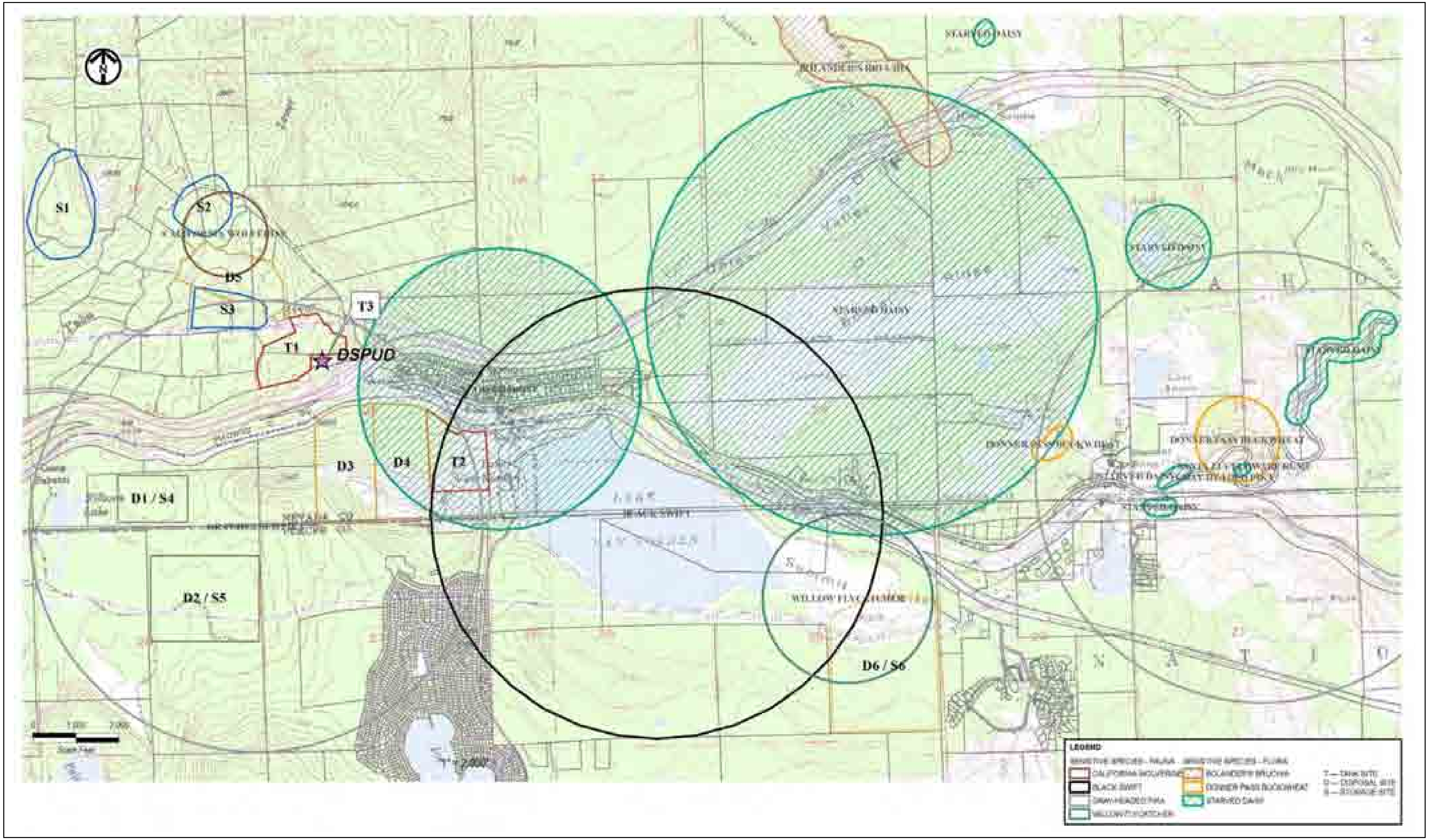


Figure 16-3
Special-Status Plant and Fauna Species Known to Occur in Proximity to the Alternative Sites

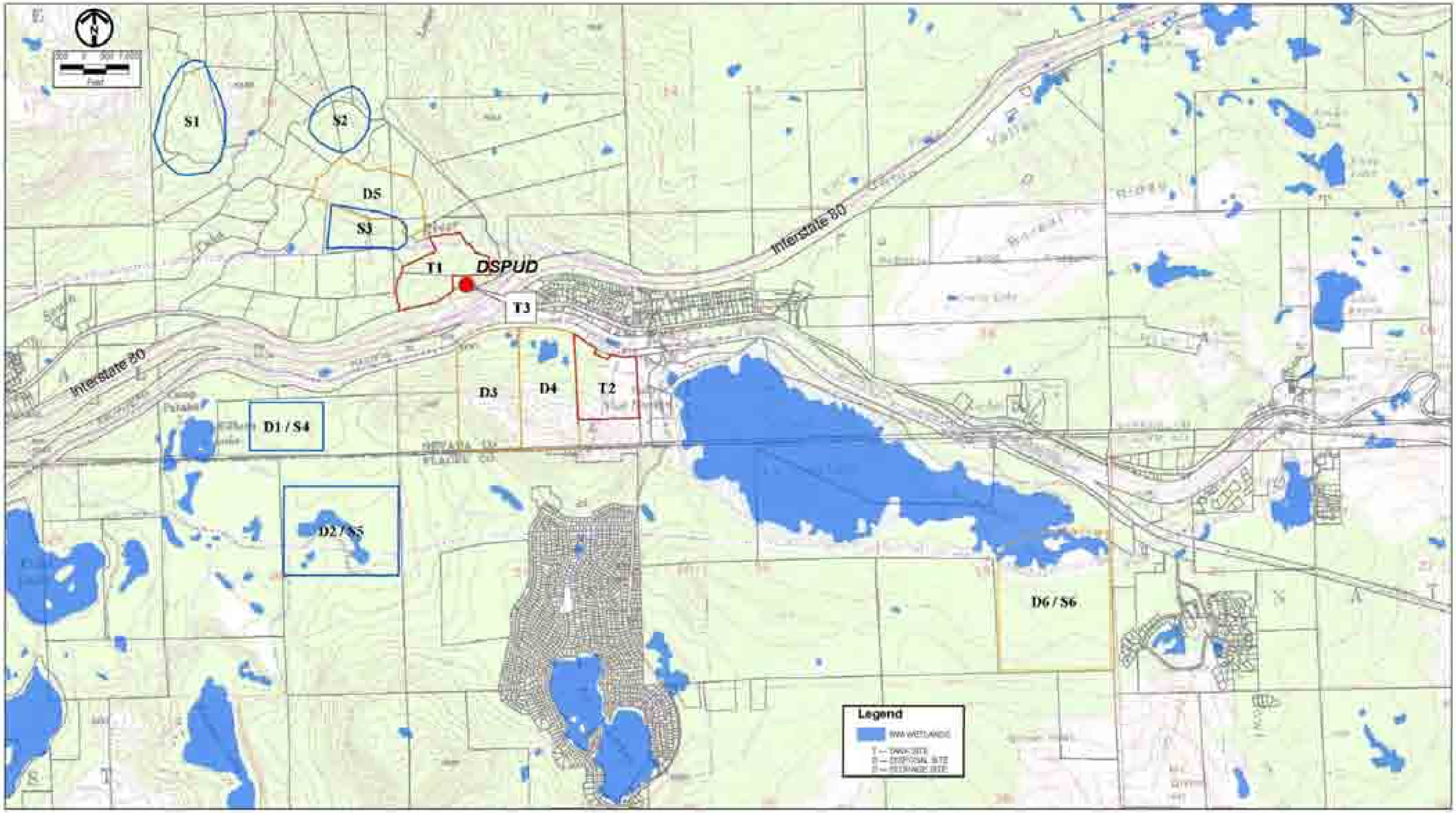


Figure 16-4
Wetlands Mapped on the National Wetland Inventory (NWI) in Proximity to the Alternative Sites

At a minimum, closely working with engineers to make reasonable adjustments to pipeline, tank, or reservoir locations can reduce the level of impacts to these resources and also reduce mitigation costs accordingly.

Soil types were mapped for all parcels being considered for effluent storage or disposal in this Facilities Plan based on the Nevada County and Placer County soil surveys completed by the Natural Resource Conservation Service (NRCS). Most of the soils mapped within the alternative sites are granitic complexes, usually including rocky outcrops with gravelly sandy loams on slopes of 2 percent to 50 percent. Therefore, most of the soils are well drained and are not suitable for wetlands except for the sites with flatter topography, fewer rocky outcrops, less drainage, and soils that contain wet complexes such as irrigation disposal sites D3 and D4 and irrigation disposal/treated effluent storage site D6/S6 (see figures for locations).

The results of the database searches for special-status plant and wildlife species are located in Table 16-1. Table 16-2 includes an overview of the biological communities present within the alternative sites evaluated in this Facilities Plan. For each biological community present in this analysis, the vegetation type, common avian, mammalian, plant, and tree species with the likelihood to occur within each is documented.

16.2 ENVIRONMENTAL REGULATORY OVERVIEW

16.2.1 SECTION 404 OF THE CLEAN WATER ACT (CWA)

The Corps and the Environmental Protection Agency (EPA) regulate the discharge of dredge or fill material into waters of the United States under Section 404 of the CWA ("waters of the United States" include wetlands and lakes, rivers, streams, and their tributaries). Wetlands are defined for regulatory purposes as areas *"…inundated or saturated by surface or ground water at a frequency and duration sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated solid conditions"* (333 CFR 328.3, 40 CFR 230.3). Project proponents must obtain a permit from the Corps for all discharges of fill material into waters of the United States, including wetlands, before proceeding with a proposed action.

Waters of the United States include a range of wet environments such as lakes, rivers, streams (including intermittent streams), mudflats, sandflats, wetlands, sloughs, and wet meadows. Boundaries between jurisdictional waters and uplands are determined in a variety of ways depending on which type of waters is present. Methods for delineating wetlands and non-tidal waters are described below.

Table 16-1
Special-Status Species With a Potential to Occur in the Facilities Plan Study Area

Common Name ***Scientific Name***	**Legal Status[a]**			**Geographic Distribution/ Floristic Province**	**Preferred Habitat**	**Identification Period**	**Level of Potential for Occurrence Within Project Sites**
	Federal	**State**	**CNPS**				
Plants							
Common moonwort *Botrychium lunaria*	–	–	2	1980-3400 meters	Meadows and seeps, subalpine montane coniferous forest, upper montane coniferous forest	August	**Low**. Last reported in Sagehen Creek area north of Truckee with no other occurrences in area (CNDDB, 2010).
Bolander's bruchia *Bruchia bolanderi*	–	–	2	1700-2800 meters	Lower montane coniferous forest, meadows and seeps, upper montane coniferous forest on damp soil	All year	**Low**. Last reported in Castle Valley in Tahoe National Forest with no other occurrences in area (CNDDB, 2010).
Constance's sedge *Carex constanceana*	–	–	1B	2000 meters	Subalpine coniferous forest on shady, mesic soils	August	**Low**. Last reported in Sagehen Creek Experimental Forest in 2008 with no other occurrences in area (CNDDB, 2010).
Mud sedge *Carex limosa*	–	–	2	1200-2700 meters	Bogs and fens, lower montane coniferous forest, meadows and seeps, marshes and swamps, upper montane coniferous forest	June-August	**Low**. Known in the Eagle Lakes area from a 1973 list of ferns and seed plants of Nevada County (CNDDB, 2010).
Fell-fields claytonia *Claytonia megarhiza*	–	–	2	2600-3532 meters	Subalpine coniferous forest on rocky or gravelly soils	July-September	**Low**. Last reported on the north side of Mount Lola Summit in 1975 with no other occurrences in area (CNDDB, 2010).
English sundew *Drosera anglica*	–	–	2	1300-2000 meters	Bogs and fens, meadows and seeps on mesic soils	June-September	**Low**. Last reported near the Sagehen Creek Field Station in 1975 (CNDDB, 2010).
Supalpine fireweed *Epilobium howellii*	–	–	1B	2000-2700 meters	Meadows and seeps, subalpine coniferous forest on mesic soils	July-August	**Low**. Last reported sightings in 2007 at 4 Sierra Pacific locations (CNDDB, 2010).
Starved daisy *Erigeron miser*	–	–	1B	1840-2620 meters	Upper montane coniferous forest on rocky soils	June-October	**Moderate**. Last reported sighting in 2006 near Donner Peak with older reports near DSPUD facilities (CNDDB, 2010).

Common Name *Scientific Name*	Legal Status[a]			Geographic Distribution/ Floristic Province	Preferred Habitat	Identification Period	Level of Potential for Occurrence Within Project Sites
	Federal	State	CNPS				
Plants (*continued*)							
Donner pass buckwheat *Eriogonum umbellatum var. torreyanum*	–	–	1B	1855 – 2620 meters	Meadows and seeps, upper montane coniferous forest on volcanic and rocky soils. Prefers steep slopes and ridge tops usually in bare or sparsely vegetated areas.	July-September	**Low**. Last reported sighting in 2001 on Tahoe National Forest property (CNDDB, 2010).
Plumas ivesia *Ivesia sericoleuca*	–	–	1B	1465-2200 meters	Great Basin scrub, lower montane coniferous forest, meadows and seeps, vernal pools on vernally mesic, usually volcanic soils	May-October	**Low**. Last sighted near Independence Lake in 1946 (CNDDB, 2010).
Webber's ivesia *Ivesia webberi*	C	–	1B	1000-2075 meters	Great Basin scrub, lower montane coniferous forest, pinyon and juniper woodland	May-July	**Low**. Last sighted near Webber Lake in 1875 (CNDDB, 2010).
Santa Lucia dwarf rush *Juncus luciensis*	–	–	1B	300-2040 meters	Chaparral, Great basin scrub, lower montane coniferous forest, meadows and seeps, vernal pools	April-July	**Low**. Last reported sighting in 2006 near Donner Pass on Tahoe National Forest property (CNDDB, 2010).
Long-petaled lewisia *Lewisia longipetala*	–	–	1B	2500-2925 meters	Alpine boulder and rock field, subalpine coniferous forest on mesic rocky or granitic soils	July-August	**Low**. Last reported sightings from 3 locations in 1991 (CNDDB, 2010).
Saw-toothed lewisia *Lewisia serrata*	–	–	1B	900-1435 meters	Broadleaf upland forest, lower montane coniferous forest, riparian forest on mesic, rocky slopes	May-June	**Low**. Last reported sighting in 1980 at a "Sensitive" location in Placer County (CNDDB, 2010).
Broad-nerved hump moss *Meesia uliginosa*	–	–	2	1300-2804 meters	Bogs and fens, meadows and seeps, subalpine coniferous forest, upper montane coniferous forest on damp soils	October	**Low**. Last reported in 2004 at the headwaters of Sagehen Creek (CNDDB, 2010).

Common Name *Scientific Name*	Legal Status[a]			Geographic Distribution/ Floristic Province	Preferred Habitat	Identification Period	Level of Potential for Occurrence Within Project Sites
	Federal	State	CNPS				
Plants (*continued*)							
Stebbins' phacelia *Phacelia stebbinsii*	–	–	1B	610-2010 meters	Cismontane woodland, lower montane coniferous forest, meadows and seeps	May-July	**Low**. Last reported sighting from 1997 (CNDDB, 2010).
White-stemmed pondwed *Potamogeton praelongus*	–	–	2	1800-3000 meters	Marshes and swamps in deep water and lakes	July-August	**Low**. Last reported sighting in 2001 in Catfish Lake (CNDDB, 2010).
Robbins' pondweed *Potamogeton robbinsii*	–	–	2	1530-3300 meters	Marshes and swamps in deep water and lakes	July-August	**Low**. Not known from Placer or Nevada Counties (CNDDB, 2010).
Alder buckhorn *Rhamnus alnifolia*	–	–	2	1370-2130 meters	Lower montane coniferous forest, meadows and seeps, riparian scrub, upper montane coniferous forest	May-July	**Low**. Last reported sighting in 1996 along Little Truckee River (CNDDB, 2010).
White beaked-rush *Rhynchospora alba*	–	–	2	60-2040 meters	Bogs and fens, meadows and seeps, marshes and swamps	July-August	**Low**. Last reported sighting in 1973 at Eagle Lakes with no other occurrences in area (CNDDB, 2010).
Water bulrush *Schoenoplectus subterminalis*	–	–	2	750-2250	Bogs and fens, marshes and swamps on montane lake margins	June-August	**Low**. Last reported sighting in 2007 at Eagle Lakes with no other occurrences in area (CNDDB, 2010).
Munroe's desert mallow *Sphaeralcea munroana*	–	–	2	2000 meters	Great basin scrub	May-June	**Low**. Last reported sighting in 1922 along Squaw Creek with no other occurrences in area (CNDDB, 2010).
Fish							
Lahontan cutthroat trout *Oncorhynchus (= Salmo) clarki henshawi*	T	–	N/A	Endemic to the Physiographic Lahontan basin of northern Nevada, eastern California and Southern Oregon.	Found in a wide variety of cold water habitats including lakes, rivers, and streams. Generally prefer cool flowing water with available cover, well vegetated stable stream banks and relatively silt free waters.	Year-round depending on life stage. Spawns April-July.	**None**. Not found in the Yuba River or within any tributaries to the Yuba River

Common Name *Scientific Name*	Legal Status[a]			Geographic Distribution/ Floristic Province	Preferred Habitat	Identification Period	Level of Potential for Occurrence Within Project Sites
	Federal	State	CNPS				
Amphibians							
Foothill yellow-legged frog *Rana boylii*	–	SSC	N/A	Known in the Sierra Nevada up to 2040 meters to south of Fresno County	Associated with shallow, flowing water in small to moderate sized streams with some cobble-sized substrate.	Year-round depending on life stage	**Low**. Last reported sighting in 1994 along the North Fork of the American River. Found at lower eleveations.
Sierra Nevada yellow-legged frog *Rana sierrae*	C	SSC	N/A	Known in the Sierra Nevada range from 1370 to over 3650 meters from Plumas County to Fresno County.	Associated with streams, lakes, and ponds in montane riparian, lodgepole pine, sub-alpine conifer, and wet meadow habitats. Always found within a few feet of water.	Year-round depending on life stage	**Moderate**. Recently reported sightings within streams and lakes in the project area.
Birds							
Cooper's hawk *Accipter cooperii*	MB	SSC	N/A	Breeding habitat in the Sierra Nevada extends from Shasta County south to northern Kern County.	Nest sites mainly in riparian growths of deciduous trees, including in canyon bottoms on river floodplains and in live oaks.	Year-round	**Moderate**. Project activities may occur within riparian and forested areas; however, this species nests in lower elevations and is not likely to nest within the project area.
Northern goshawk *Accipter gentalis*	MB	SSC	N/A	Permanent resident in the Klamath and Cascade Ranges, in the North Coast Ranges from Del Norte County to Mendocino County, and in the Sierra Nevada south to Kern County. Winters in Modoc, Lassen, Mono, and northern Inyo Counties	Nests and roosts in older stands of red fir, Jeffrey pine, Ponderosa pine, lodgepole pine, Douglas fir, and mixed conifer forests. Usually nests on north slopes and near water	Year-round	**Low**. Project activities will occur within forest areas; however, project sites are within or adjacent to developed and/or disturbed areas making it less likely the species would nest within the project area.
Black swift *Cypseloides niger*	–	SSC	N/A	Breeding habitat in the Sierra Nevada extends from Shasta County south to northern Kern County.	Nest sites mainly in riparian growths of deciduous trees, including in canyon bottoms on river floodplains and in live oaks.	Year-round	**Low**. Project activities may occur within riparian and forested areas; however, this species nests in lower elevations and is not likely to nest within the project area.

Common Name *Scientific Name*	Legal Status[a]			Geographic Distribution/ Floristic Province	Preferred Habitat	Identification Period	Level of Potential for Occurrence Within Project Sites
	Federal	State	CNPS				
Birds (*continued*)							
Yellow warbler *Dendroica petechia brewsteri*	–	SSC	N/A	Western Slope of Sierra Nevada to Kern County and Eastern side from Lake Tahoe south to Inyo. Rare to uncommon in lowlands.	Riparian deciduous habitats in summer. Also breeds in montane shrubbery in open coniferous forests. In migration visits woodland shrub habitats	Neotropical migrant (Summer late spring and early fall)	**Moderate**. Limited riparian areas for cover along Yuba River; however, areas adjacent to Lake Van Norden meadow contain suitable habitat for this species.
Willow flycatcher *Empidomax traillii*	–	E	N/A	Summers along the western Sierra Nevada from El Dorado to Madera Counties and in northern Sierra Nevada in Trinity, Shasta, Tahama, Butte, and Plumas Counties. Found between 2000-8000 feet.	Extensive thickets of low, dense willows on the edge of wet meadows, ponds, or backwaters. Usually found in riparian habitats during migration.	Spring/Summer	**Moderate**. Potential project areas adjacent to Van Norden meadow contain suitable habitat (riparian thickets) for this species; however, there is limited riparian areas for cover along the Yuba River.
American peregrine falcon *Falco peregrinus anatum*	D, MB	E,FP	N/A	Permanent resident along the north and south Coast Ranges. May summer in the Cascade and Klamath Ranges and through the Sierra Nevada to Madera County. Winters in the Central Valley south through the Transverse and Peninsular Ranges and the plains east of the Cascade Range.	Nests and roosts on protected ledges of high cliffs, usually adjacent to lakes, rivers, or marshes that support large prey populations.	Summer	**Low**. Project activities will occur within forest areas; however, project sites are within or adjacent to developed and/or disturbed areas making it less likely the species would nest within the project area.

Common Name *Scientific Name*	Legal Status[a]			Geographic Distribution/ Floristic Province	Preferred Habitat	Identification Period	Level of Potential for Occurrence Within Project Sites
	Federal	State	CNPS				
Birds (*continued*)							
Bald eagle *Haliaeetus leucocephalus*	D, MB	E,FP	N/A	Nests in Siskiyou, Modoc, Trinity, Shasta, Lassen, Plumas, Butte, Tehama, Lake, and Mendocino Counties and in the Lake Tahoe Basin. Winter range includes the rest of California, except the southeastern deserts, very high altitudes in the Sierra Nevada, and east of the Sierra Nevada south of Mono County.	In western North America, nests and roosts in coniferous forests within 1.5 kilometers of a lake, reservoir, stream, or the ocean.	Year-round	**Low**. Project activities will occur within forest areas; however, project sites that are within 1.5 kilometers of a lake or stream lie adjacent to developed and/or disturbed areas making it less likely the species would nest within the project area.
Osprey *Pandion haliaetus*	MB	PR	N/A	Sierra Nevada from Lassen County south to northern Kern County, and in the Transverse, Peninsular and southern coastal mountains	Mature forest with suitable nesting trees. In southern California, occurs in oak and oak-conifer habitats in addition to mature conifer forest	Year-round	**Low**. Recorded along Donner Lake. Project area appears unlikely to nest in the project area since the area lacks suitable foraging habitat for this species.
Great gray owl *Strix nebulosa*	–	E	N/A	Permanent resident of the Sierra Nevada from Plumas County south to the Yosemite area. Occasionally occurs in northwestern California in the winter and the Warner mountains in the summer.	Late successional coniferous forests bordering meadows	Year-round	**Low**. No known records of this species in the project area. Project area appears to lack suitable habitat for this species.
California spotted owl *Strix occidentalis occidentalis*	–	SSC	N/A	Sierra Nevada from Lassen County south to northern Kern County, and in the Transverse, Peninsular and southern coastal mountains	Mature forest with suitable nesting trees. In southern California, occurs in oak and oak-conifer habitats in addition to mature conifer forest	Year-round	**Low**. No known records of this species in the project area. Project area appears to lack suitable habitat for this species.

Common Name *Scientific Name*	Legal Status[a]			Geographic Distribution/ Floristic Province	Preferred Habitat	Identification Period	Level of Potential for Occurrence Within Project Sites
	Federal	State	CNPS				
Mammals							
Sierra Nevada Mountain Beaver *Aplodondia rufa californica*	–	SSC	N/A	Found through out the Cascade, Klamath, and Sierra Nevada Ranges.	Montane riparian habitat preferred. Frequent open and intermediate canopy coverage with dense understory near water. Deep friable soils are required for burrowing.	Year-round (nocturnal)	**Low**. Project area lacks suitable habitat with small deciduous trees and shrubs except for along the Yuba River and Van Norden meadow.
California wolverine *Gulo gulo luteus*	–	T,FP	N/A	Klamath and Cascade Ranges south through the Sierra Nevada to Tulare County	Sighted in a variety of habitats from 480–4,325 meters. Most common in open terrain above timberline and subalpine forests	Year-round (largely nocturnal)	**Low**. Prefer areas with low human disturbance. Uses caves, hollows, cliffs, and logs for cover in denser forests, but may hunt in more open areas. Known from 1973 within project area.
Sierra Nevada snoeshoe hare *Lepus americanus tahoensis*	–	SSC	N/A	Uncommon resident at upper elevations throughout the Sierra Nevada to Mariposa, Mono, and Madera Counties.	Primarily found in montane riparian habits with thickets of alders and willows and in stands of young conifers interspersed with chaparral.	Year round (crepuscular and nocturnal)	**Low**. Rarely found in open spaces. Activity primarily crepuscular and nocturnal. Last reported sighting in area was 1969.
Western white-tailed jackrabbit *Lepus tonsendii*	–	SSC	N/A	An uncommon to rare year round resident of the crest and upper slow of the Sierra Nevada primarily from the Oregon border to Tulare and Inyo Counties. Migrates to higher areas in summer and descends to lower regions in summer.	Areas of scattered shrubs with cover such as thickets of young conifers or low branches of stunted conifers. Feeds in summer in open meadows.	Year Round (nocturnal)	**Low**. Last reported sighting was 1920. Feeds in open meadows during summer; however has nocturnal (sometimes crepuscular) activity patterns. Project activities in the meadow surface will occur in winter.
Pacific fisher *Martes pennanti (pacifica) DPS*	C	SSC	N/A	Coastal mountains from Del Norte County to Sonoma Counties, east through the Cascades to Lassen County, and south in the Sierra Nevada to Kern County	Late successional coniferous forests and montane riparian habitats with a high percentage of canopy cover. Uses cavities, snags, and logs for cover and denning.	Year-round	**Low**. Project site lacks suitable habitat for this species. There is limited late successional forest and riparian habitat with a high percentage of canopy cover in the vicinity of the project.

Common Name *Scientific Name*	Legal Status[a]			Geographic Distribution/ Floristic Province	Preferred Habitat	Identification Period	Level of Potential for Occurrence Within Project Sites
	Federal	State	CNPS				
Mammals *(con't)*							
Gray-headed pika *Ochotona princeps schisticeps*	–	–	N/A	Uncommon resident at upper elevations throughout the Sierra Nevada to Mariposa, Mono, and Madera Counties.	Primarily found in montane riparian habits with thickets of alders and willows and in stands of young conifers interspersed with chaparral.	Year round (crepuscular and nocturnal)	**Low**. Last reported sighting project area was 1937.
American badger *Taxidea taxus*	–	SSC	N/A	Cascade Ranges in Siskiyou County, the Sierra Nevada from Lassen County south to Tulare County	Most abundant in drier open stages of most shrub, forest, and herbaceous habitats with friable soils.	Year-round	**Low**. Could visit open forested, riparian, and meadow areas within the project area; however, there are no documented sightings of this species in the area.
Sierra Nevada red fox *Vulpes vulpes necator*	–	T	N/A	Cascade Ranges in Siskiyou County, the Sierra Nevada from Lassen County south to Tulare County	Coniferous forests, generally from 1,500–2,560 meters. Often associated with mountain meadows	Year-round	**Low**. Could visit forested and meadow areas; however, documented sightings of this species are few in the area.

Federal
- T = listed as threatened under the federal Endangered Species Act.
- D = delisted under the federal Endangered Species Act
- PD = proposed for delisting
- C = candidate to become a proposed species
- MB = Migratory Bird Treaty Act
- – = no listing.

State
- E = listed as endangered under the California Endangered Species Act.
- T = listed as threatened under the California Endangered Species Act.
- R = listed as rare under the California Native Plant Protection Act. This category is no longer used for newly listed plants, but some plants previously listed as rare retain this designation.
- CE = candidate species for listing as endangered under the California Endangered Species Act
- FP = fully protected species
- SSC = species of special concern in California
- PR = Protected Raptor Species
- – = no listing.

California Native Plant Society (CNPS)
- 1B = List 1B species: rare, threatened, or endangered in California and elsewhere.
- 2 = List 2 species: rare, threatened, or endangered in California but more common elsewhere.
- 3 = List 3 species: plants about which more information is needed to determine their status.
- 4 = List 4 species: plants of limited distribution.

Table 16-2
Biological Communities Found Within the Preliminary Environmental Analysis Areas

Biological Communities	Location	Vegetation Type	Common Wildlife	Common Vegetation
Mixed Coniferous Forest	**Sites S1, S2, S3, D1/S4, D2/S5, D5, D6/S6 adjacent to Lake Van Norden Meadow, and T1 adjacent to the DSPUD Facilities**	Eastside pine and mixed conifer	**Avian species:** western tanager (*Piranga ludoviciana*), western wood peewee (*Contopus sordidulus*), hairy woodpecker (*Picoides villosus*), mountain chickadee (*Poecile gambeli*), white-breasted nuthatch (*Sitta carolinensis*), brown-headed cowbird (*Molothrus ater*), chipping sparrow (*Spizella passerina*), Oregon junco (*Junco hyemalis thurberi*), yellow-rumped warbler *Dendroica coronata*), northern flicker (*Colaptes auratus*), and Steller's jay (*Cyanocitta telleri*). **Mammalian species:** lodgepole chipmunk (*Tamias speciosus*), mule deer (*Odocoileus hemionus*), montane vole (*Microtus montanus*), fisher (*Martes pennanti*), California vole (*Microtus californicus*), black bear (*Ursus americanus*), raccoon (*Procyon lotor*), mountain lion (*Felis concolor*), and western gray squirrel (*Sciurus griseus*).	**Major vegetation types:** 1. Eastside pine, 2. Lodgepole pine, 3. Mixed conifer, 4. Subalpine conifer, 5. White fir. **Tree species:** Jeffrey pine (*Pinus jeffreyi*), white fir (*Abies concolor*), sugar pine (*Pinus lambertiana*), ponderosa pine (*Pinus ponderosa*), lodgepole pine (*Pinus contorta* ssp. *murrayana*), and western white pine (*Pinus monticola*). **Plant species:** Indian paintbrush (*Castilleja pinetorum*), snowberry (*Symphoricarpos mollis*), mule ears (*Wyethia mollis*), Sierra currant (*Ribes nevadense*), and mountain pride (*Penstemon newberryi*)
Red Fir Forest	**Effluent Irrigation Disposal Sites D3 and D4, and parts of Site D1/S4**	Red fir trees	See above discussion as species are similar for both biological communities.	These habitats within the project areas are characterized by **dense stands of red fir** (*Abies magnifica*). Because the canopy associated with this habitat is extremely dense and relatively impermeable to sunlight, the understory supports sparse vegetation.

Biological Communities	Location	Vegetation Type	Common Wildlife	Common Vegetation
Montane Meadow	**Lake Van Norden Meadow, D6/S6, and the adjacent to the Yuba River**	Mixed meadow plants	**Species include:** American robin, mountain chickadee, cliff swallow (*Petrochelidon pyrrhonota*), killdeer (*Charadrius vociferus*), mourning dove, northern flicker, California mule deer, western bluebird (*Sialia mexicana*), and green-tailed towhee (*Pipilo chlorurus*)	**Major vegetation types:** 1. Annual grass/forbs, 2. Wet meadow, 3. Perennial grass, and 4. Mixed meadow. **Shrubs:** various willows (*Salix* spp.), **Grasses and forbs Species:** meadow barley (*Hordeum brachyantherum*), common monkeyflower (*Mimulus guttatus*), clover (*Trifolium* spp.), Indian paintbrush, mint (*Mentha* sp.), shooting star (*Dodecatheon jeffreyi*), and yarrow (*Achillea millefolium*) **Herbaceous species:** fireweed (*Epilobium angustifolium*.), cinquefoil (*Potentilla* sp.), and primrose (*Primula* sp.).
Riparian Scrub	**Along the Yuba River and adjacent to Lake Van Norden Meadow**	Willow and quaking aspen	**Species include:** raccoon, western gray squirrel, California mule deer, northern flicker, mountain chickadee, and lodgepole chipmunk.	**Vegetation types:** 1. Willow, 2. Quaking aspen, 3. Willow-aspen. **Species include:** willow (*Salix* sp.), alder (*Alnus tenuifolia*), cottonwood (*Populus* sp.), and quaking aspen (*Populus tremuloides*)

- Wetlands are defined as “those areas that are inundated or saturated by surface or groundwater at a frequency and duration sufficient to support and under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions” (33 CFR Section 328.3[b]). Presently, to be a wetland, a site must exhibit three wetland criteria: hydrophytic vegetation, hydric soils, and wetland hydrology existing under the “normal circumstances” for the site.
- The lateral extent of non-tidal waters is determined by delineating the ordinary high water mark (OHWM) (33 CFR Section 328.4[c][1]). The OHWM is defined by the Corps as “the line on the shore established by the fluctuations of water and indicated by physical character of the soil, destruction of terrestrial vegetation, the presence of litter and debris, or other appropriate means that consider the characteristics of the surrounding areas” (33 CFR Section 328.3[e]).

Under the Clean Water Act Section 404, wastewater treatment ponds and associated wastewater drainage systems that are not considered tributaries to waters of the U.S. are exempt from U.S. Army Corps of Engineer Jurisdiction (40 CFR Part 230.3 Subpart A [t]).

16.2.2 MIGRATORY BIRD TREATY ACT AND BALD AND GOLD EAGLE PROTECTION ACT

The Migratory Bird Treaty Act (MBTA, 16 United States Code Section 703-711) and the Bald and Golden Eagle Protection Act (16 USC Section 668) protect certain species of birds from direct take. The MBTA protects migrant bird species from take through setting hunting limits and seasons and protecting occupied nests and eggs. Additionally, there are California Department of Fish and Game (CDFG) Codes (3503, 3503.5 and 3800), which further protect nesting birds and their parts (see State Regulations sections below). The Bald and Gold Eagle Protection act prohibits the take or commerce of any part of these species. The USFWS administers both Acts and reviews federal agency actions that may affect species protected by the Acts.

Typically, it is recommended that all vegetation removal be conducted outside of the nesting season, which generally falls between February 1 and August 30, however this may vary from year to year depending on various environmental conditions. If vegetation must be removed during the breeding season, a qualified biologist should conduct a nest survey of the entire project site immediately prior to the removal of vegetation.

16.2.3 FEDERAL ENDANGERED SPECIES ACT (FESA)

The U.S. Fish and Wildlife Service (USFWS) has jurisdiction over species listed as threatened or endangered under Section 9 of the ESA. The act protects listed species from harm or *take* which is broadly defined as “…the action of harassing, harming, perusing, hunting, shooting, wounding, killing, trapping, capturing, or collecting, or attempting to engage in any such conduct.” For any project involving a federal agency (including the issuance of a permit) in which a listed species could be affected, the federal agency must consult with the USFWS in accordance with Section 7 of the FESA. The USFWS issues a biological opinion and, if the project does not jeopardize the continued existence of the listed species, issues an incidental-take permit.

16.2.4 CALIFORNIA ENDANGERED SPECIES ACT (CESA)

The California Department of Fish and Game (CDFG) has jurisdiction over species listed as threatened or endangered under section 2080 of the California Fish and Game Code. The California Endangered Species Act (CESA) prohibits take of state-listed threatened and endangered species. The state Act differs from the federal Act in that it does not include habitat destruction in its definition of *take*. The California Fish and Game Code defines *take* as "hunt, pursue, catch, capture, or kill, or attempt to hunt, pursue, catch, capture, or kill." The CDFG may authorize *take* under the CESA through Sections 2081 agreements. If the results of a biological survey indicate that a state-listed species would be affected by the project, the CDFG would issue an Agreement under Section 2081 of the CDFG Code and would establish a Memorandum of Understanding for the protection of state-listed species. CDFG maintains lists for Candidate-Endangered Species and Candidate-Threatened Species. California candidate species are afforded the same level of protection as listed species.

16.2.5 CDFG SPECIES OF CONCERN

In addition to formal listing under FESA and CESA, species receive additional consideration by CDFG and lead agencies during the CEQA process. Species that may be considered for review are included on a list of "Species of Special Concern", developed by these resource agencies. It tracks species in California whose numbers, reproductive success, or habitat may be in decline. Species of Special Concern, which are species of limited distribution, declining populations, diminishing habitat, or unusual scientific, recreational or educational values. These species do not have the same legal protection as listed species, but may be added to official lists in the future.

16.2.6 STREAMBED ALTERATION AGREEMENTS: CDFG CODE SECTION 1600 ET. SEQ.

CDFG has jurisdictional authority over wetland resources associated with rivers, streams, and lakes under Sections 1600–1616. CDFG has the authority to regulate all work within the State of California that would substantially divert, obstruct, or change the natural flow of a river, stream, or lake; substantially change the bed, channel, or bank of a river, stream, or lake; or use material from a streambed.

In practice, CDFG marks its jurisdictional limit at the top of the stream or lake bank or the outer edge of the riparian vegetation, where present, and sometimes extends its jurisdiction to the edge of the 100-year floodplain. Because riparian habitats do not always support wetland hydrology or hydric soils, wetland boundaries, as defined by CWA Section 404, sometimes include only portions of the riparian habitat adjacent to a river, stream, or lake. Therefore, jurisdictional boundaries under Section 1600 may encompass a greater area than those regulated under CWA Section 404.

CDFG enters into a streambed alteration agreement with an applicant and can impose conditions on the agreement to ensure that no net loss of wetland values or acreage will be incurred. The streambed or lakebed alteration agreement is not a permit, but a mutual agreement between CDFG and the applicant.

16.2.7 CALIFORNIA NATIVE PLANT SOCIETY – NATIVE PLANT SPECIES LIST

The California Native Plant Society (CNPS) maintains a list of plant species, native to California, that have low numbers, limited distribution, or are otherwise threatened with extinction. This information is published in the *Inventory of Rare and Endangered Vascular Plants of California* (Skinner and Pavlik, 1994). Potential impacts to populations of CNPS-listed plants receive consideration under CEQA review. The following identifies the definitions of the CNPS listings:

- List 1A: Plants believed extinct
- List 1B: Plants rare, threatened, or endangered in California and elsewhere
- List 2: Plants rare, threatened, or endangered in California, but more numerous elsewhere
- List 3: Plants about which we need more information - a review list
- List 4: Plants of limited distribution - a watch list

16.2.8 NESTING MIGRATORY BIRDS AND RAPTORS: CDFG CODE SECTIONS 3503, 3503.5, AND 3800

Sections 3503, 3503.5, and 3800 of the CDFG Code prohibit the take, possession, or destruction of birds, their nests or eggs. Implementation of the take provisions requires that project-related disturbance at active nesting territories be reduced or eliminated during critical phases of the nesting cycle (March 1 - August 15, annually). Disturbance that causes nest abandonment and/or loss of reproductive effort (e.g., killing or abandonment of eggs or young) or the loss of habitat upon which the birds depend is considered "taking" and is potentially punishable by fines and/or imprisonment. Such taking would also violate federal law protecting migratory birds (e.g., MBTA above).

16.2.9 CEQA GUIDELINES SECTION 15380

CEQA *Guidelines* Section 15380(b) provides that a species not listed on the federal or state list of protected species may be considered rare or endangered if the species can be shown to meet certain specific criteria. This section was included in the guidelines primarily to deal with situations in which a public agency is reviewing a project that may have a significant effect on, for example “candidate species” that have not yet been listed by the USFWS or CDFG. CEQA, therefore, enables an agency to protect a species from significant project impacts until the respective government agencies have an opportunity to list the species as protected, if warranted.

In general, plants appearing on the CNPS List 1 (plants believed to be extinct and rare, threatened or endangered in California) and List 2 (rare, threatened, or endangered plants in California but more numerous elsewhere) are considered to meet CEQA’s Section 15380 criteria. Impacts to these species would, therefore, be considered “significant” requiring mitigation.

16.2.10 CEQA OAK WOODLANDS CONSERVATION LAW

Effective January 1, 2005, Senate Bill 1334 (Kuehl) established Public Resources Code (PRC) Section 21083.4, the state's first oak woodlands conservation standards for the CEQA processes. This new code requires counties to determine whether or not a project may cause a significant effect or conversion of oak woodlands during the CEQA process whenever a County is a responsible agency for a project. In addition, if the counties determine a project will significantly affect oak woodlands, the project proponent must employ one or more of the following CEQA Oak Woodlands Mitigation Alternatives:

- Conserve oak woodlands through the use of conservation easements.
- Plant an appropriate number of trees, including maintaining plantings and replacing dead or diseased trees.
- Contribute funds to the Oak Woodlands Conservation Fund, as established under subdivision (a) of Section 1363 of the Fish and Game Code, for the purpose of purchasing oak woodlands conservation easements.
- Other mitigation measures developed by the County.

This law states that county actions resulting in the loss of oak trees five inches or more in diameter at breast height (dbh or 4.5 ft.) will be subject to compensatory mitigation measures. Oaks less than 5 inches dbh will still be subject to conservation measures contained in county ordinances or general plans. Placer County does have a specific tree ordinance that supersedes this law; however Nevada County does not have a specific tree ordinance. Both Nevada and Placer counties have specific policies and goals within their General Plans that refer to tree and other environmental protections (Appendix F).

16.3 CULTURAL RESOURCES REGULATIONS

CEQA, PRC Section 21083.2, and CEQA Guidelines 15064.5 include provisions for significance criteria related to archaeological and historical resources. A significant archaeological or historical resource is defined as one that meets the criteria of the California Register of Historical Resources (CRHR), is included in a local register of historical resources, or is determined by the lead agency to be historically significant. A significant impact is characterized as a "substantial adverse change in the significance of a historical resource."

PRC Section 5024.1 authorizes the establishment of the CRHR. Any identified cultural resource must therefore be evaluated against the CRHR criteria. In order to be determined eligible for listing in the CRHR, a property must be significant at the local, state, or national level under one or more of the four significance criteria modeled on the NRHP criteria.

16.3.1 CALIFORNIA REGISTER OF HISTORICAL RESOURCES

In order to be determined eligible for listing in the California Register of Historical Resources (CRHR), a property must be significant at the local, state, or national level under one or more of the following four criteria as defined in Public Resources Code 5024.1 and CEQA Guideline 15064.5(a).

1. It is associated with events or patterns of events that have made a significant contribution to the broad patterns of the history and cultural heritage of California and the United States.
2. It is associated with the lives of persons important to the nation or to California's past.

3. It embodies the distinctive characteristics of a type, period, region, or method of construction, or represents the work of an important creative individual, or possesses high artistic values.
4. It has yielded, or may be likely to yield, information important to the prehistory or history of the state and the nation.

In addition to meeting one or more of the above criteria, a significant property must also retain integrity. Properties eligible for listing in the CRHR must retain enough of their historic character to convey the reason(s) for their significance. Integrity is judged in relation to location, design, setting, materials, workmanship, feeling, and association.

16.3.2 PUBLIC RESOURCES CODE

PRC Section 21083.2 governs the treatment of unique archaeological resources, defined as "an archaeological artifact, object, or site about which it can be clearly demonstrated" as meeting any of the following criteria:

1. Contains information needed to answer important scientific research questions and that there is a demonstrable public interest in that information.
2. Has a special and particular quality such as being the oldest of its type or the best example of its type.
3. Is directly associated with a scientifically recognized important prehistoric or historic event or person.

If it can be demonstrated that a project will cause damage to a unique archaeological resource, appropriate mitigation measures shall be required to preserve the resource in place and in an undisturbed state. Mitigation measures may include, but are not limited to, 1) planning construction to avoid the site, 2) deeding conservation easements, or 3) capping the site prior to construction. If a resource is determined to be a "non-unique archaeological resource", no further consideration of the resource by the lead agency is necessary.

The preferred alternatives subject to DSPUD Board of Supervisors approval (projects that go to design intended to be constructed and covered under CEQA) will undergo a thorough investigation of cultural and historical resources with proper mitigation included to avoid and minimize impacts to these resources. During CEQA compliance and regulatory permitting for any of the project alternatives that are approved for design and construction, at a minimum the following activities will be required:

- A records search will be conducted by an archeologist with professional qualifications in conjunction with the North Central Information Center at the California State University, Sacramento
- A detailed cultural and historical resources survey will be conducted for all areas under consideration in any project alternative
- Consultations with the California Native American Heritage Commission will be conducted

- A formal Cultural Resources Study and report will be developed for CEQA and Section 106 consultations if necessary

16.3 PROJECT SITE SURVEY RESULTS

ECO:LOGIC biologists conducted field surveys to document potential environmental constraints and "fatal flaws" on October 1, 2009. ECO:LOGIC biologists visited each of the treated effluent storage reservoir, tank, and effluent irrigation disposal alternative sites to document general habitats and biological communities present at each, and to identify the potential presence of wetlands and habitats for special-status plant and fauna species. The surveys were reconnaissance in nature and did not include in-depth analyses of botanical species, soils types (including presence of hydric soils), or jurisdictional wetlands on any of the sites. The biological community of each alternative site along with the common vegetation and wildlife spotted in each was noted in a field journal. Based on the results of the database searches as described in Section 16.1, ECO:LOGIC biologists noted whether a site contained potential habitats for any special-status plant or fauna species and whether potential waters of the U.S., including wetlands were present.

Biological communities within the alternative sites, including a list of general tree and wildlife species, are described in Table 16-2. The most common biological community in the alternative sites is Mixed Coniferous Forest, which contains up to 5 dominant species of trees. This biological community is dominant in the following sites: S1, S2, S3, D5, T1, D1/S4, D2/S5, and D6/S6 adjacent to the Lake Van Norden Meadow. Site references in this section refer to previous sections within the Facilities Plan. Each site is described in Sections 13 and 14 herein referred to as S for treated effluent storage reservoir sites, T for tank sites, and D for effluent irrigation disposal sites.

The lower, steeper slopes on the south side of Interstate 80 contain the biological community called Red Fir Forest. The effluent irrigation disposal sites D3 and D4 are dominated by this community. Red Fir is the dominant species in this biological community and due to its dense canopy contains minimal understory vegetation.

The large Lake Van Norden meadow (site D6/S6 included) and small, isolated areas adjacent to the South Yuba River (near sites D5 and S2) contain the biological community named Montane Meadow. In most cases, a Montane Meadow will meet the criteria of the Corps and will therefore be considered a jurisdictional wetland and will be subject to the Clean Water Act Section 404 permitting requirements, if impacted.

Most sites contain stands of mature native trees (mostly hardwood species) that are either protected by Placer County (General Plan policies and tree ordinance) or Nevada County (General Plan policies) depending on which County they are located in. The Placer County Tree Ordinance requires that all native trees with a diameter at breast height (dbh) of greater than 6 inches (10 inches for multiple trunked trees) be mitigated if they are removed or impacted significantly during construction. All native trees in Placer County besides the Grey Pine (*Pinus sabiniana*) are included and protected and a tree permit is required prior to removing any protected trees in Placer County. The Nevada County General Plan protects native oak trees with either a dbh of 36 inches or greater or any grove of native hardwood trees with a 33 percent canopy closure.

Lastly, Riparian Scrub is a biological community that associates with rivers and streams. Riparian Scrub was documented to occur along the South Yuba River (near sites D5 and S2) and also associates with small areas along the southern end of the Lake Van Norden meadow (site D6/S6 included). Riparian Scrub can sometimes meet the criteria of the Corps and therefore can be subject to the Clean Water Act Section 404 permitting requirements. Riparian Scrub is generally regulated by the CDFG and impacts to this biological community are normally permitted under the Lake or Streambed Alteration Agreement program (CDFG Code 1600 *et seq*.).

Environmental site constraints and potential "fatal flaws" are detailed below. Sites that are not considered the preferred alternative sites are detailed first, while the preferred alternative sites are detailed at the end of Section 16.3. Treated effluent storage reservoir and tank sites are numbered as presented in Section 13 of this Facilities Plan, while the effluent irrigation disposal sites are numbered according to Section 14. The numbering below refers to Sections 13 and 14 and are represented in Figure 16-3 and Figure 16-4.

South Yuba River Diffuser and Gaging Station

Section 11 within the Facilities Plan details issues related to effluent disinfection. An aspect of effluent disinfection is the production of disinfection byproducts from the use of chlorine during the disinfection process, including dichlorobromomethane. In order for DSPUD to obtain dilution credits for continued use of chlorine, DSPUD should consider the installation of a river diffuser and river gaging station (as detailed in Section 11 within the Facilities Plan). The installation of a river diffuser that crosses the South Yuba River channel in the general area of the existing WWTP outfall would include direct impacts to riparian vegetation along the banks of the river and to the river bed itself. However, if constructed during the time of year when flows within the river are minimal, impacts to aquatic habitats within the South Yuba River can be minimized to a level of less than significant.

Since the South Yuba River is considered a "waters of the U.S." by the Corps and is also regulated by CDFG (for impacts to the river's bed and bank and riparian zone), the installation of a river diffuser and river gaging station would require Section 404 and 401 permits, and a CDFG Streambed Alteration Agreement (CDFG Code 1600 *et seq*.). Depending on the results of plant and fauna surveys in the river diffuser and gaging station areas, consultations with USFWS and CDFG could be required for special-status species. Each required permit would contain mitigation for direct and indirect impacts to the South Yuba River. However, working closely with project engineers, regulatory agencies, and DSPUD, the project can be designed to minimize these impacts.

Potential Treated Effluent Storage (Reservoir) Sites

Reservoir Site Alternative - S1 – Myers Site.. Located about 1.5 mi NW of the DSPUD WWTP, north of the South Yuba River and along a steep, southwest facing slope. Granitic rock and/or boulders are exposed along most of the surface. Access is difficult along steep/narrow roads. The National Wetland Inventory (NWI) maps do not show any wetlands on this site; however, the site contains several drainages of various sizes (small to large) that apparently flow from the site, from above the site, and through the site to the South Yuba River, making them potentially jurisdictional waters of the U.S. The potential for special-status species is low to moderate with the most likely special-status species for this site being nesting raptors since the site contains several large trees. Most of the trees on this side of I-80 include almost pure stands of lodgepole pine with some intermittent Jeffrey pine, white fir, and mountain hemlock. This site was eliminated from further consideration given engineering concerns with construction of a pipeline from the WWTP to the site. This site is further from the WWTP than several other sites, and it would require pumping from the WWTP, both of which would contribute to higher overall environmental impact (greater disturbance to natural resources and greater energy needs).

Reservoir Site Alternative – S2 – Outfall Site. Located about 0.8 mi NW of the DSPUD WWTP on the north side of the South Yuba River. Terrain is mostly gentle, rising to the north. The surface is mostly silty sand with scattered cobbles and boulders. Through that portion of the site where the South Yuba River runs through it, the floodplain is not very wide given the surrounding topography; however, areas of riparian trees and vegetation (mostly willows) line both sides of the South Yuba River. This site was observed and evaluated from existing DSPUD parcels (site D5) and through an analysis of aerial photography of the site. This site contains lodgepole pine with some intermittent Jeffrey pine, white fir, and mountain hemlock. The NWI maps do not show any wetlands on this site; however, the site contains several drainages (generally small in this area) that pass through the site to the South Yuba River, making them potentially jurisdictional waters of the U.S. Several small drainages run through the site and one of the drainages is located in an area determined to be a potential reservoir site of up to 5 acres. This site also contains dense woodland that would most likely require a moderate level of mitigation for trees removed under Nevada County policies and CEQA.

The potential for special-status species is moderate with the most likely special-status species for this site being nesting raptors since the site contains a large number of large trees. A current search of the California Natural Diversity Data Base (CNDDB) shows several special-status fauna species in the vicinity of this site, including the CA wolverine (CA threatened species), gray-headed pika (CA species of concern), black swift (CA species of concern), and willow flycatcher (CA endangered). The starved daisy (CNPS listed 1B species) is the only known special-status plant species in the vicinity of this site.

This site's major environmental constraint is the South Yuba River. Crossing the South Yuba River with a pipeline would require a Section 404 (Corps), Section 401 (RWQCB), and Section 1602 Streambed Alteration Agreement (CDFG) permits, which are a constraint and would include both mitigation and monitoring requirements. If possible, travel to and from this reservoir site should be conducted by crossing an existing bridge; however, if any work, including establishing a temporary crossing, within the South Yuba River should be done when the river is dry to minimize impacts to fish and other aquatic species. The lack of a suitable existing bridge could require construction of a river crossing, which could lead to further impacts to the riparian corridor and the bed and bank of the South Yuba River itself.

Reservoir Site Alternative – S3 – Franz Site. This site is assessed below as a preferred alternative.

Reservoir Site and Irrigation Disposal Alternative – Site S4/D1 – Royal Gorge North Site. Located about 1 mile southwest of the DSPUD WWTP, south of I-80, UPRR, and the South Yuba River on Royal Gorge owned property. Generally on this side of the South Yuba River, red fir trees are very plentiful; however, lodgepole pines, mountain hemlock, and western white pine trees are also intermixed. The north-central portion of Site No. 4 is within a gentle topographic "bowl" with potential for a reservoir site. The NWI maps do not show any wetlands on this site; however, the site contains several small drainages that most likely drain to the South Yuba River or nearby lakes, making them potentially jurisdictional waters of the U.S. The site contains many trees, mostly lodgepole pines in the area where the "bowl" is located. There are also some red firs and western white pines on the site. See attached photos of this site.

The potential for special-status species is moderate with the most likely special-status species for this site being nesting raptors since the site contains a number of large trees. The density of trees on this site is very high, with a dense stand of lodgepole pines in and adjacent to the "bowl" area. The CNDDB shows several special-status wildlife species in the vicinity of this site, including the CA wolverine (CA threatened species), gray-headed pika (CA species of concern), black swift (CA species of concern), and willow flycatcher (CA endangered). The starved daisy (CNPS listed 1B species) is the only known special-status plant species in the vicinity of this site. Other documented sensitive species on the southern side of the South Yuba River include the Pacific fisher (CA species of concern, Federal Candidate), Sierra marten (CA species of concern), northern goshawk (CA species of concern), yellow warbler (CA species of concern), and Sierra Nevada mountain beaver (CA species of concern). Nesting raptors and birds would be the most likely sensitive species to be found on Site S4/D1.

Reservoir and Irrigation Disposal Site Alternative – Site D2/S5 – Royal Gorge South Site. Located about 1.2 mi SW of the DSPUD WWTP, on Royal Gorge land. This site was evaluated via a windshield survey while passing through this parcel with a Royal Gorge employee. This site was not walked. However, based on aerials of the site and the drive through the site, the entire site appears to be crossed by a large drainage that contains several obvious wetlands, including one very large wetland in the middle of the site (documented on the NWI Figure 16-3). Removal of trees on this site would require a Placer County tree permit and mitigation for the trees and woodland under CEQA since it is located in Placer County. This site is the least desirable of storage sites S1 through S5 due to the large drainage and wetland complexes within this site. The site is eliminated from further consideration.

Reservoir Site Alternative – S6 – Sugar Bowl Site. Located about 2.5 mi SE of the DSPUD WWTP, on Sugar Bowl property (which borders Royal Gorge on its south and west sides). This site is at the southeast end of old Lake Van Norden (now mostly a large meadow). The meadow is highly visible and contains a large wetland complex and likely shallow groundwater. The meadow area would be a major environmental constraint in regards to permitting, mitigation fees, and local opposition to this site is expected to develop during the CEQA and permitting process. Lake Van Norden and the adjacent meadow are listed on the NWI as wetlands and it would most likely fit the criteria of a jurisdictional wetland by the Corps.

The CNDDB shows several special-status wildlife species in the vicinity of this site, including the gray-headed pika (CA species of concern), black swift (CA species of concern), willow flycatcher (CA endangered), and northern goshawk (CA species of concern). The yellow warbler (CA species of concern) would have a high likelihood of occurring in the willow thickets surrounding the large meadow. The starved daisy (CNPS listed 1B species) is the only known special-status plant species in the vicinity of this site. Other documented sensitive species on this side of the South Yuba River include the Pacific fisher (CA species of concern, Federal Candidate), Sierra marten (CA species of concern), and Sierra Nevada mountain beaver (CA species of concern).

Areas of higher ground exist to the south at this site (south of electrical transmission lines crossing the site), which appears to be suitable for either an earthen fill reservoir, or for spray irrigation facilities and would lie outside the meadow/wetland complex. This site would create the longest route for a new effluent pipeline. The meadow and adjacent wetlands would need to be avoided to keep public opposition and mitigation fees for the project to a minimum. The southern side of the meadow/wetland complex is dominated by red fir trees and nesting raptors and birds would be the most likely sensitive species to be found in this area of Site D6/S6. Removal of trees on this site (in the meadow area and south of the meadow) would require a Placer County tree permit and mitigation for native trees removed with a dbh of greater than 6 inches since it is located in Placer County and woodland impacts would need to be assessed under CEQA.

Effluent Irrigation Disposal Alternatives

Irrigation Disposal Alternative – Site D1 – Royal Gorge North Site. This site is assessed above with site S4 as a treated effluent reservoir site alternative.

Irrigation Disposal Alternative – Site D2 – Royal Gorge South Site. This site is assessed above with site S5 as a treated effluent reservoir site alternative.

Irrigation Disposal Alternative – US Forest Service Parcel – D3. This spray site is located adjacent to site D4. This site would be preferred to site D4 based on environmental constraints alone since this site contains a much smaller area of drainages and wetlands that are mapped on the NWI and were confirmed to exist during the survey. This site also contains less dense woodland (mostly red fir with some lodgepole pines) than site D4. The drainages and wetlands can be avoided by spraying above the areas mapped as containing wetlands. However, to reach this parcel with spray irrigation equipment, site D4 would most likely need to be crossed and a run off collection system would need to be constructed to collect excess water that doesn't percolate into the soils. This would cause additional impacts to the dense woodlands and potentially to jurisdictional wetlands and drainages. The additional impacts due to crossing site D4 would most likely lead to this site D4 being the preferred site.

Site D4 could also contain any of the sensitive species listed on sites S4, S5, and S6. This site also contains dense woodland on the lower, northern end of the parcel and less dense woodland higher up the north facing slope.

Irrigation Disposal Alternative – Boreal Ridge Corporation Parcel – D4. This site is assessed below as a preferred alternative.

Irrigation Disposal Alternative – DSPUD Parcels – D5. These spray sites are located between Reservoir Site S3 and Reservoir Site S2 on property owned by DSPUD. Terrain is mostly gentle, rising to the north. The surface is mostly silty sand with scattered cobbles and boulders. The South Yuba River runs through these parcels, which would be a major constraint if the river could not be avoided. As such, were effluent disposal proposed here, significant buffers would be included to reduce potential impacts the effluent irrigation might have on the river. The South Yuba River floodplain is not very wide given the surrounding topography; however, areas of riparian trees and vegetation (mostly willows) line both sides of the South Yuba River (Riparian Scrub biological community). This site contains lodgepole pine with some intermittent Jeffrey pine, white fir, and mountain hemlock. The NWI maps do not show any wetlands within these parcels; however, the parcels contain several drainages (generally small in this area) that pass through to the South Yuba River, making them potentially jurisdictional waters of the U.S.

The potential for special-status species is moderate with the most likely special-status species for this site being nesting raptors since the site contains a number of large trees. A current search of the CNDDB shows several special-status fauna species in the vicinity of this site, including the CA wolverine (CA threatened species), gray-headed pika (CA species of concern), black swift (CA species of concern), and willow flycatcher (CA endangered). The starved daisy (CNPS listed 1B species) is the only known special-status plant species in the vicinity of this site.

Irrigation Disposal and Reservoir Site Alternative – Site D6/S6 – Sugar Bowl Site. This site is assessed above with site S6 as a treated effluent reservoir site alternative.

Tank Site Alternatives

Tank Site – T1. Located less than 0.25 mi NW of treatment plant, south of PG&E transmission lines and south of South Yuba River. Some rocky tributary channels exist between the transmission lines and river. The area appears to contain rocky channels that would be difficult to excavate and require diversion of surface flows. Though the NWI maps do not show any wetlands on this site, the tributary channels and drainages that pass through the site could be jurisdictional waters of the U.S. and would require permitting if crossed or impacted; however, a formal wetland delineation should be conducted to determine the extent and location of any waters of the U.S., including wetlands within this tank site. Potential special-status species on this site would be the same as outlined for Reservoir Alternative S3 located to the NW of this site. This site also contains dense woodland that would most likely require a moderate level of mitigation for trees and woodlands removed under Nevada County policies and CEQA.

Tank Site – T2 – Existing Boreal Ridge Corporation Ski Area. This potential tank site is located within the existing parcel that is currently being used as a spray field and it is owned by Boreal Ridge Corporation. This site does contain several constraints, including very little room at the base of the existing ski run to place a tank. The placement of a tank on this site could be an aesthetic issue and could impact tubing operations depending on the tank size and placement location. This site does not contain the dense woodland of red fir and lodgepole pines that are located on sites D3 and D4. This site could also contain any of the sensitive species listed on Sites S4, S5, and S6; however, since the site does not contain woodland and is currently operated as a ski resort, the potential of these species to be located within this site are considered low.

Preferred Alternative Sites

DSPUD WWTP Tank Site – T3. Siting of a tank within the existing DSPUD WWTP site (site T3 on Figures 16-3 and 16-4) would limit environmental impacts if placed in a developed area and if pipelines to and from the storage tank did not cross any sensitive habitat. The NWI maps do not show any wetlands within or adjacent to the WWTP site. A current search of the California Natural Diversity Data Base (CNDDB) shows several special-status fauna species in the vicinity of the WWTP site, including the CA wolverine (CA threatened species), gray-headed pika (CA species of concern), black swift (CA species of concern), and willow flycatcher (CA endangered). However, the starved daisy (CNPS listed 1B species) is the only known special-status plant species in the direct vicinity of the WWTP site. If a tank is placed within the developed areas on the existing WWTP site, these special-status species would not be impacted by its development.

Reservoir Site Alternative – S3 – Franz Site. Located about 0.5 mi NW of the DSPUD WWTP, north of PG&E transmission lines and south of South Yuba River. Some rocky tributary channels exist between the transmission lines and river. In their preliminary report (October 1, 2009) Blackburn Consulting, Inc. states that the "rocky channels would be difficult to excavate and require diversion of surface flows." Though the NWI maps do not show any wetlands on this site, the tributary channels and drainages that pass through the site could be jurisdictional waters of the U.S. and would require permitting if crossed or impacted. Potential special-status species on this site would be the same as outlined for Reservoir Alternative S2. This site also contains dense woodland that would most likely require a moderate level of mitigation for trees removed under Nevada County policies and CEQA. This site is considered a preferred reservoir site.

Irrigation Disposal Alternative – Boreal Ridge Corporation Parcel – D4. This spray site is located adjacent to the existing parcel that is currently being used as a spray field (owned by Boreal Ridge Corporation); therefore, it would be straight forward to extend the spray irrigation system and collection system to this site. This site does contain 2 constraints, but they are small enough to warrant this site as a preferred alternative since it is located adjacent to an operational spray irrigation system. The site constraints include the following: (1) this site contains several drainages, and (2) a large wetland mapped on the NWI was confirmed to exist during the survey. However, the drainages and wetlands can be avoided by spraying above and to the sides of the areas mapped as containing wetlands. The NWI wetland contains almost a pure stand of mountain alder contained in a drainage area. The area of wetland on this site that would actually be considered jurisdictional by Corps criteria is most likely smaller than what was mapped by NWI; however, a formal wetland delineation would need to be conducted to determine the jurisdictional limits of this feature.

This site also contains dense woodland of red fir and lodgepole pines that would most likely require some mitigation for trees under Nevada County policies and CEQA. This site could also contain any of the sensitive species listed on Sites S4, S5, and S6.

16.4 PRELIMINARY ENVIRONMENTAL ANALYSIS CONCLUSIONS

In regards to overall environmental constraints, the following treated effluent storage (reservoir sites), tank, and effluent irrigation disposal alternatives would be the preferred alternatives based on the environmental analysis conducted in Sections 16.1 and 16.2:

- DSPUD WWTP Tank Site – T3
- Reservoir Site Alternative – S3 – Franz Site
- Irrigation Disposal Alternative – Boreal Ridge Corporation Parcel – D4

As described in Sections 16.1 and 16.2, these sites were selected based on being sites least likely to contain environmental “fatal flaws” as well as meeting the overall goal of the Facilities Plan.

The other sites were not selected as the preferred alternatives in regards to environmental constraints for several reasons. Site D6/S6 (includes an alternative to develop a treated effluent storage reservoir and/or develop the site for effluent irrigation disposal) is the only site that has a clear “fatal flaw.” Site D6/S6 presents the site with the greatest potential to cause both impacts to the environment (wetlands, special-status species, etc.) and create public scrutiny of the project, especially if any infrastructure connecting the site with DSPUD existing facilities impact Lake Van Norden and its associated highly valued wetlands (mountain meadow).

Reservoir Site S3 looks most promising for locating a potential effluent storage reservoir. Reservoir Site S3 would be preferred since it is located on the south side of the South Yuba River. Reservoir Site S2 has the South Yuba River as a constraint if a pipeline is to cross the river or a crossing is constructed to access the site. Reservoir Site S1 is logistically not feasible and Reservoir Site S4, S5, and S6 have a significant number of wetland and permitting issues that should make them less feasible unless a pipeline and reservoir could be developed outside of these sensitive areas. Out of these additional reservoir alternatives, Reservoir Site S4 would be most preferable to the other two on the south side of Interstate 80. Reservoir Site S6 contains a “fatal flaw” since the location could impact the Lake Van Norden Meadow, which is not only protected by the Corps, but impacts to the meadow would be highly scrutinized during the CEQA process.

The preferred effluent irrigation disposal alternative would include site D4 as opposed to site D3. Since the parcel is located adjacent to the existing parcel that is currently being used as a spray field (T2 - owned by Boreal Ridge Corporation), the least environmentally damaging alternative would include extending the existing spray irrigation system on T2 to D4. Additional drainage could be collected and returned to the existing (or slightly modified) pond at the base of the existing ski slope operations or the pond could be expanded if necessary to take on additional holding capacity as needed. Though this preferred effluent irrigation disposal alternative contains an area of wetlands mapped by NWI and would most likely be considered jurisdictional by Corps criteria, this feature could most likely be avoided during the expansion of effluent irrigation disposal capacity within this parcel.

The preferred tank site alternative would include site T3 as opposed to sites T1 and T2. Since the T3 site is located within the existing DSPUD WWTP facilities, the least environmentally damaging alternative for tank sites would include the construction of an additional tank or tanks at this site. There would be less ground disturbance to native biological communities and it is less likely that a tank at this site would require the level of scrutiny and mitigation as the other two alternative sites.

The installation of a river diffuser and river gaging station is detailed in Section 11 within the Facilities Plan. The installation of a river diffuser that crosses the South Yuba River channel in the location of the existing outfall would include direct impacts to riparian vegetation along the banks of the river and to the river bed itself. However, if constructed during the time of year when there are no flows within the river or flows are minimal, impacts to the South Yuba River can be minimized to a level of less than significant.

Section 17

Selection and Description of the Apparent Best Project

Section 17

Selection and Description of the Apparent Best Project

In the previous sections of this report, every major component of the DSPUD wastewater treatment and disposal system has been investigated to determine any improvements needed to attain regulatory compliance, while handling the design flows and loads established for the proposed project. In many cases, alternative analyses were completed to analyze several options and identify the most cost effective means for accomplishing the design and operational goals for the plant components in question. In some cases, however, a selection of the apparent best alternative for a particular part of the plant could not be made based solely on analysis of that part, because of interdependencies with other plant components. In this section, an overall alternative analysis is presented to assist in selection of the apparent best combination of components, considering all the interdependencies involved. Subsequently, the complete apparent best project is described, including alternatives for certain undecided aspects that remain to be determined by DSPUD after review of this document. A flow diagram, conceptual site layout and an implementation schedule are presented.

17.1 OVERALL ALTERNATIVE ANALYSIS

As developed in the previous sections of this report, the following interdependencies between plant components exist:

1. The type of equalization storage and headworks system required will depend on the biological treatment alternative selected. In particular, the MBR alternative would require new finer screens in the plant headworks.
2. UV and ozone disinfection facilities would be different for the MBR alternative than for the other biological treatment alternatives (however, ozone was eliminated from further consideration).
3. There are differences in the types and amounts of residual solids (sludge) produced by the various biological treatment alternatives, which impacts solids handling facilities costs.

In addition to the items listed above, the choice of biological treatment and disinfection alternatives will have an impact on the layout and cost of additional shop and office space needed at the wastewater treatment plant. With the MBR alternative, the existing filtration system can be removed, making approximately 1250 square feet available for expansion of the shop area in the existing Advanced Treatment Building. Similarly, if UV disinfection is selected, approximately 430 square feet currently devoted to chlorine disinfection in the same building can be made available for additional operations office space. According to District staff, these additional areas are needed and, if existing space cannot be repurposed, new building space would be required.

The exact size and layout of any new facilities would have to be determined during preliminary design. For the purpose of this study, it is adequate to assume the areas indicated above. Accordingly, costs for removing existing equipment and remodeling existing building spaces or building new spaces of equivalent size, as appropriate for the various biological treatment and disinfection options, are included in the alternative cost analysis discussed below. Like all capital costs in the cost analysis discussed below, the building costs are complete capital costs, including electrical, sitework, general conditions, overhead, profit, contingencies, engineering, administration and environmental studies costs.

17.1.1 COSTS OF COMBINED ALTERNATIVES

In Table 17-1, an overall alternative cost analysis is presented to show the relative costs of the various biological treatment alternatives, when coupled with either chlorine or UV disinfection. In each case, the costs for equalization and headworks facilities, solids handling facilities, and shop/office space corresponding to the option in question are shown. Capital, annual, and present worth costs are given.

As indicated in the footnotes to Table 17-1, the costs for chlorine disinfection are based on chlorination without ammonia present (not chloramination). Accordingly, the estimated costs of studies, a diffuser and a flow gaging station in the South Yuba River needed to obtain dilution credits for disinfection byproducts are included. The reader is referred to Section 17.2 for further consideration of project costs based on chloramination.

17.1.2 OVERALL COMPARISON OF COMBINED ALTERNATIVES

In Table 17-2, the various alternative combinations are rated with respect to several key economic and non-economic criteria, each of which has been assigned an importance weighting factor. Table 17-2 was developed with the input and review of DSPUD staff and the Joint Wastewater Facilities Committee formed by DSPUD and SLCWD in an effort to assure that the criteria included in the table and the relative weighting factors appropriately reflect the interests and concerns of DSPUD and SLCWD.

The criteria, weighting factors and ratings are discussed briefly below.

Capital and Annual Costs

Capital and annual costs are the first two criteria by which the alternatives are rated and the ratings reflect the costs indicated in Table 17-1. Capital cost is assigned a rating factor of 25 percent, meaning, in effect, that 25 percent of the overall decision on which alternative to select is based on capital cost. Annual cost is assigned a weighting factor of 10 percent, with the net result being that costs (capital and annual combined) account for 35 percent of the overall decision. The relative weighting for capital cost versus annual cost reflects the fact that capital costs are generally two or more times the present worth of annual costs for all alternatives.

Table 17-1
Overall Alternative Cost Analysis

Biological Treatment Alternative: Disinfection Alternative:	Cost for Indicated Combination of Alternatives (a), $							
	Upgrade Existing IFAS		New IFAS		MBR		Submerged Attached Growth	
	Chlorine	UV	Chlorine	UV	Chlorine	UV	Chlorine	UV
Capital Cost								
Equalization Storage / Headworks (b)	2,250,000	2,250,000	2,250,000	2,250,000	3,730,000	3,730,000	2,250,000	2,250,000
Biological Treatment	6,230,000	6,230,000	7,355,000	7,355,000	10,140,000	10,140,000	16,590,000	16,590,000
Filtration (c)	201,000	201,000	201,000	201,000	0	0	700,000	700,000
Disinfection (d)	1,199,000	2,628,000	1,199,000	2,628,000	1,199,000	1,753,000	1,199,000	2,628,000
Solids Handling (e)	523,000	523,000	523,000	523,000	523,000	523,000	523,000	523,000
Reconfigure Existing Space for Shop/Office	0	25,000	0	25,000	50,000	75,000	0	25,000
New Shop/Office Space	475,000	385,000	475,000	385,000	195,000	105,000	475,000	385,000
Total	10,878,000	12,242,000	12,003,000	13,367,000	15,837,000	16,326,000	21,737,000	23,101,000
Annual Cost								
Equalization Storage / Headworks (b)	47,000	47,000	47,000	47,000	48,000	48,000	47,000	47,000
Biological Treatment	227,000	227,000	233,000	233,000	251,000	251,000	293,000	293,000
Filtration (c)	11,950	11,950	11,950	11,950	0	0	14,340	14,340
Disinfection (d)	20,400	35,740	20,400	35,740	20,400	37,140	20,400	35,740
Solids Handling (e)	43,400	43,400	43,400	43,400	44,600	44,600	60,600	60,600
Total	349,750	365,090	355,750	371,090	364,000	380,740	435,340	450,680
Present Worth Cost								
Present Worth of Annual Costs (f)	5,204,000	5,433,000	5,294,000	5,522,000	5,416,000	5,665,000	6,478,000	6,706,000
Total Present Worth	16,082,000	17,675,000	17,297,000	18,889,000	21,253,000	21,991,000	28,215,000	29,807,000

(a) First quarter 2010 cost level, ENR 20-Cities CCI = 8700.
(b) Based on Equalization Concept 1.
(c) New coagulation and flocculation assumed to be required ahead of the filters for the submerged attached growth option.
(d) Chlorine cost based on free chlorine, not chloramination. Costs include studies and facilities needed to obtain dilution credits for disinfection byproducts. UV disinfection for MBR based on closed vessel system.
(e) Based on continued use of existing solids storage tank and sludge drying beds.
(f) 20 years at inflation-adjusted discount rate of 3 percent. Present Worth Factor = 14.88.

Table 17-2
Alternative Ratings and Ranking

Criterion	Weighting Factor %	Ratings For Indicated Alternative Combination (a)							
		Upgrade Existing IFAS		New IFAS		MBR		Submerged Attached Growth	
		Chlorine	UV	Chlorine	UV	Chlorine	UV	Chlorine	UV
Capital Cost	25	10.0	8.9	9.1	8.1	6.8	6.6	5.0	4.7
Annual Cost	10	10.0	9.6	9.8	9.4	9.6	9.2	8.0	7.8
Confidence In Design and Technology	25	4	4	8	8	10	10	7	7
Robustness and Reliability	5	8	8	8	8	10	10	8	8
Misc. Compliance Improvements, Exi	5	6	7	6	7	9	10	6	7
Adaptability to Future Permits	5	6	8	6	8	10	8	6	8
Ease of Future Expansion	5	9	9	9	9	10	10	9	9
Plant Footprint	5	8	8	8	8	10	10	8	8
Construction Impacts in River (d)	3	5	10	5	10	5	10	5	10
Power Use	3	9	8	9	8	8	7	10	9
Chemical Use	3	9	10	9	10	9	10	8	9
Residuals Produced	3	10	10	10	10	10	10	8	8
Hazardous Gas Exposure Risk	3	3	10	3	10	3	10	3	10
Overall Weighted Score (b)	100	7.43	7.63	8.19	8.41	8.66	8.88	6.67	7.09
Rank (c)		6	5	4	3	2	1	8	7

(a) The highest rated alternative is assigned a score of 10. Other alternatives are scored lower, according to the relative concern compared to the highest rated alternativ
(b) Summation of individual ratings multiplied by the corresponding weighting factors.
(c) The alternative with the highest overall weighted score is ranked "1". Other alternatives are ranked "2" through "8", according to overall score.
(d) Construction in the river would be associated with continuing chlorine disinfection, based on installing a diffuser to obtain dilution credits for disinfection byproducts.

Confidence in Design and Technology

Confidence in design and technology was assigned a weighting factor of 25 percent to reflect the high importance of this criterion. The MBR alternative was given a rating of 10 because it is an established technology with probably thousands of plants by various manufacturers throughout the world. Also, the activated sludge biological process that is employed in the MBR is well developed and understood, being covered in many text books and research papers and being described by extensively reviewed and accepted mechanistic design models and process simulation software. The New IFAS alternative was assigned a rating of 8 because much of the design is based on proprietary empirical models developed by the manufacturers of the associated equipment, and these models are not available for peer review and independent confirmation. However, this technology is well established with hundreds of plants worldwide (includes moving bed bioreactors as well as IFAS systems). The submerged attached growth option is considered to be somewhat less established and defined than the New IFAS alternative. The alternative of upgrading the existing IFAS system using structured sheet media was given a low confidence rating because it is a new technology available from only one manufacturer with only three existing installations, two of which have existed for only one year and none of which are required to denitrify. Design procedures for this process are based on the manufacturer's research and have not been adequately validated by full scale operations.

Robustness and Reliability

Robustness and reliability was assigned a weighting factor of 5 percent. Robustness and reliability represent the degree to which the process is resilient and can perform consistently well, even in problematic conditions, such as influent flow or load spikes, extreme weather, or other challenging biological process conditions. Because the membranes provide an absolute barrier to the escape of particulate matter from the biological treatment system, very consistent performance can be assured. With a biological treatment system that relies on sludge settling in a clarifier (such as IFAS), there can be much more variability in effluent quality, which would lead to a higher probability (although still low if properly designed and operated) of potential permit violations. The point is that the MBR is more resilient and can more readily accommodate challenging conditions, including potential operator error, without compromising effluent quality. In the specific case of the DSPUD wastewater treatment plant, the existing process basins are more than adequate for the reactor requirements of a MBR, whereas they are just marginally okay for the IFAS alternatives. This adds to the relative robustness of the MBR design, as compared to the IFAS design at DSPUD. This robustness is partly evidenced by the ability to maintain a much higher biomass inventory in the MBR (see Table 9-5 in Section 9). Based on this discussion, the MBR would be considered more robust and reliable than the IFAS alternatives. The submerged attached growth alternative is judged to be of similar robustness and reliability as the IFAS alternatives.

Miscellaneous Compliance Improvements for Existing Regulated Constituents

This criterion, with a weighting factor of 5 percent, is included to reflect the additional level of treatment available through the MBR, as compared to the other alternatives, which would be helpful in meeting permit requirements for some existing regulated constituents. Because of the small pore size of the membranes, the MBR will remove small particulate constituents that none of the other alternatives can remove. This is partly evidenced by the fact that a typical effluent turbidity for the MBR is less than 0.2 NTU, while for all other alternatives, the corresponding effluent turbidity is 2.0 NTU. To the extent that various regulated constituents exist in particulate form, membrane filtration would provide additional removals. Membrane filtration can even provide incremental removals of constituents that are currently measured as “dissolved”. This is because dissolved constituents are actually determined as those that would pass through a 0.45 micron filter. The pore size used in MBR membranes can be substantially smaller than 0.45 micron, depending on the manufacturer of the membranes. The potential benefit of membrane filtration in removing existing regulated compounds has not yet been quantified because plant effluent samples have not been tested with and without membrane filtration. However, at the time of writing this document (May 2010) a program is currently underway to do such membrane filtration testing in conjunction with routine monitoring for several constituents (aluminum, silver, zinc, copper, and manganese).

In addition to possible incremental removals of constituents already in the wastewater, MBR treatment would eliminate the need to add certain chemicals that could otherwise exacerbate permit compliance. In particular, with granular media filtration that would exist with biological processes other than the MBR, it is frequently necessary to add aluminum-based coagulants. These coagulants would not be necessary with a MBR and, therefore, aluminum compliance could be improved with the MBR. Similarly, if chlorine disinfection is continued, lower chlorine doses would be needed and this would result in lower sulfur dioxide doses for dechlorination. Lowering the additions of all these chemicals will reduce the salinity of the final effluent. Additionally, lowering the chlorine dose should result in less production of disinfection byproducts.

Adaptability to Future Permit Requirements

Just as developed above for existing regulated constituents, membrane filtration could provide incremental removals of any future regulated constituents that exists partly in particulate form. Additionally, MBR treatment conditions the effluent for subsequent disinfection. The benefit of this, as developed in Section 11, is to allow much more economical UV and ozone disinfection and lower chlorine doses for chlorine disinfection. The benefits related to UV and chlorine disinfection are already reflected in other rating criteria discussed above. However, the benefits with regard to ozonation are not reflected in other rating criteria.

It is known that ozonation is an effective treatment for mitigating emerging contaminants of concern that are not yet regulated, such as pharmaceuticals and personal care products, pesticides, and others, many of which disrupt endocrine systems in exposed organisms. Throughout the United States and the industrialized world, regulators, environmental interests, wastewater professionals and, in many cases, the public have become very concerned over the presence of these substances in wastewater effluent. Substantial feminization of male fish and other aquatic life abnormalities have been seen in many locations.

Because of the concerns indicated above, and because feminization of male fish has been documented in receiving waters containing effluent from wastewater treatment plants in the Las Vegas area, the Clark County Water Reclamation District (Clark County, Nevada), working together with the Southern Nevada Water Authority, has extensively investigated the presence of emerging contaminants of concern in the Clark County wastewater effluent and has tested and proven the benefits of ozonation to mitigate these constituents. The Clark County Water Reclamation District has already installed ozonation treatment.

In California, the State Water Resources Control Board has formed a Science Advisory Panel on Chemicals of Emerging Concern in Recycled Water to guide future actions relating to the monitoring of chemicals of emerging concern for recycled water projects. Monitoring for emerging contaminants of concern in wastewater effluents is already being required in the Santa Ana region.

Based on the above, it is believed to be only a matter of time before emerging contaminants must be monitored and possibly removed at many wastewater treatment plants, including DSPUD. Therefore, DSPUD should consider that in future years, it may be necessary to implement ozonation and/or other treatments for these constituents.

MBR biological treatment is considered to be more adaptable to potential future permitting requirements for emerging contaminants of concern because of the higher level of treatment provided, which conditions the effluent for subsequent additional treatment. If DSPUD continues with chlorination (including possible chloramination) in the near-term future and then implements ozonation in future years, the MBR biological treatment alternative would have the advantage that ozone alone would be adequate for disinfection and for treatment of emerging contaminants, while for the other biological treatment alternatives, UV disinfection would be needed as a supplement to ozone. However, if DSPUD decides to install UV disinfection in the near-term and then ozonation in future years, this incremental benefit of the MBR would be eliminated (but, MBR would still have the benefits of additional removals of future regulated constituents associated with particulates).

Ease of Future Expansion

Ease of future expansion (weighting factor of 5 percent) is intended to represent how easily additional process basins and equipment could be added to increase capacity. The MBR was rated slightly higher than the IFAS options because it is considered easier to add a membrane basin and the associated equipment than a new clarifier and RAS pumping system. The MBR was rated somewhat higher than the submerged attached growth alternative because submerged attached growth would require additions to the primary treatment system as well as the secondary treatment system. Additionally, site availability for future expansion is of less concern for the MBR alternative than for the others, because of its small footprint.

Plant Footprint

The MBR option would have by far the smallest footprint, resulting in the least disturbance of the natural landscape. This was assigned a weighting factor of 5 percent.

Construction Impacts in the South Yuba River

This criterion (weighting factor of 3 percent) was included to reflect the need for construction of a cross-stream diffuser and a flow gaging station in the South Yuba River to obtain dilution credits for disinfection byproducts with chlorine disinfection. Such construction activities are likely to be opposed by environmental interests and would require additional permitting and environmental review.

The disadvantages associated with chlorine disinfection under this criterion could be eliminated if DSPUD is able to successfully test and then implement chloramination to completely mitigate disinfection byproducts, eliminating the need for dilution credits.

Power Use, Chemical Use and Residuals Produced

These three criteria indicated are intended to represent initial and future impacts on resources and the environment. These criteria do not include the cost impacts associated with power, chemicals and residuals, which are considered under other criteria. Each of these criteria was assigned a weighting factor of 3 percent.

Hazardous Gas Exposure

This criterion reflects the use (or lack thereof) of hazardous gases within the treatment system. All alternatives include the continued use of ammonia gas. Use of chlorine and sulfur dioxide gases would be eliminated with UV disinfection.

Results of Overall Comparison of Combined Alternatives

As shown in Table 17-2 the combined project alternative with the highest overall score is the MBR biological treatment alternative, coupled with UV disinfection. The second ranked alternative is MBR coupled with chlorine disinfection. Some of the reasons why chlorine disinfection was rated lower than UV disinfection could be eliminated if chloramination could be tested and proven effective as a method for mitigation of disinfection byproducts, without the need for dilution credits. This topic is discussed further below.

17.2 SELECTION OF THE APPARENT BEST PROJECT

After consideration of a draft of this Facilities Plan and participation in the development of Table 17-2, the Joint Wastewater Facilities Committee formed by DSPUD and SLCWD determined that MBR biological treatment is preferred, but a firm decision on the disinfection process cannot be made at this time. Without consideration of possible future regulations for emerging contaminants of concern, UV disinfection would be preferred. However, if ozonation is likely to be needed in future years for treating emerging contaminants of concern, then the prudence of an initial investment in UV becomes questionable, since UV would not be needed with ozonation.

The alternative of installing ozonation immediately is not desirable because of the high costs involved and the fact that ozonation is not needed to meet existing permit requirements. Furthermore, any need to implement ozonation could be more than 10 years in the future. By that time, it is hoped that ozonation in wastewater treatment might be more widespread and that the technology would become more cost-effective. Also, at that time, the specific ozone design criteria needed to meet actual permit requirements would be known, perhaps leading to a design that would be substantially different than one that would be implemented in advance of such knowledge. Finally, ozonation can produce its own byproducts that could require additional treatment, which is not reflected in the costs for ozone developed in Section 9.

In consideration of the issues above, a reasonable approach might be to continue using chlorine disinfection as long as possible and hopefully until there is more clarity regarding emerging contaminants of concern and the potential need for ozonation. Continued use of chlorine, however, may not be practical, because it may not be possible to obtain dilution credits for disinfection byproducts. Furthermore, DSPUD would like to avoid the cost and environmental concerns of working in the South Yuba River to install a cross-stream diffuser and gaging station, if these were confirmed as prerequisites to dilution credits. Also, the costs associated with continued use of chlorine, like the costs for building UV disinfection, are troublesome if ozonation later replaces chlorination. Some of the concerns associated with chlorination could be alleviated if chloramination could be tested and proven as an effective means of mitigating disinfection byproducts, without the need for dilution credits.

Based on all of the above, the Joint Wastewater Facilities Committee determined that both UV disinfection and chloramination should be carried forward for further consideration. Based on informal discussions with Regional Water Quality Control Board staff and the official rejection of dilution credits as part of the existing NPDES Permit governing the DSPUD wastewater treatment plant (adopted in April, 2009), the committee determined that pursuing dilution credits for disinfection byproducts would be undesirable. Therefore, chlorination (as opposed to chloramination) should not be considered further at this time.

Another matter considered by the Joint Wastewater Facilities Committee was whether biostimulation storage and the spray irrigation system expansion that would be triggered by such storage should be included in the recommended project. Since the causes and contributing factors that produced the algal bloom in the South Yuba River in June 2008 are not known and since no such bloom occurred in 2009 nor is known to have occurred in years prior to 2008, the need for spending millions of dollars on a biostimulation storage reservoir and associated expansion of the spray irrigation disposal system cannot be firmly established at this time. Further studies of biostimulation in the South Yuba River are ongoing and planned. Accordingly, the committee determined that project costs with and without biostimulation storage and related facilities should be indicated in the Facilities Plan.

It is anticipated that DSPUD, working together with SLCWD, will decide whether to pursue chloramination or UV disinfection and whether or not to include biostimulation storage in the proposed project after review of this document and consideration of other factors that are relevant to the two Districts.

17.2.1 Summary Description of Project Components

In this section, the apparent best project is summarized, to the degree that it can be defined at this time. Two disinfection alternatives are considered, as noted above. A flow diagram and a conceptual site plan for the recommended improvements (with alternatives) are presented in Figures 17-1 and 17-2, respectively. The various improvements are discussed briefly below.

Influent Flow Equalization and Headworks

The new 550,000 gallon Equalization Storage Tank 2 (EST2) and the associated Equalization Return Pump Station would be located generally behind the existing Operations Building (original firehouse), with a new access road from the east side of the building, all as shown in Figure 17-2. EST2 would be somewhat taller than existing Equalization Storage Tank 1 (EST1). The most cost-effective combination of diameter and height to give the desired volume will be determined in preliminary design. It is proposed that the top water surface elevation in EST2 be slightly lower than in EST1 to allow gravity filling of EST2 from an overflow from EST1, if it is desired to fill the tanks in that order. Alternatively, both tanks could be filled simultaneously through interconnecting piping or independently if either tank is taken out of service.

Just like the two tanks can be filled simultaneously, they can be drained simultaneously with flow going back from EST2 to EST1 before being metered into the treatment system. However, the lower portion of EST2 that cannot be drained to EST1 or to the headworks by gravity would have to be pumped through the proposed new Equalization Return Pump Station at EST2.

Both EST1 and EST2 would be fitted with new jet aeration systems that allow independent control of mixing and aeration. Additionally, both tanks would have overflows that would be routed to the Emergency Storage Tank.

A new headworks facility with fine screens needed to protect the MBR process would be constructed just to the west of the existing Operations Building. This will require relocating the existing propane tanks that are currently in that location. Due to excessive head loss concerns, it will probably be necessary to abandon the existing headworks and use only the new headworks. However, during preliminary design, the possible use of the existing headworks screen as a coarse screen in front of the proposed new fine screens should be considered.

Although not specifically shown in the flow diagram of Figure 17-1, the return flow from the Emergency Storage Tank can be connected to EST2 as well as EST1 to allow EST2 to be used for additional emergency storage capacity, instead of being used for equalization at such times as that may be desired. For example, in the spring, both the Emergency Storage Tank and EST2 could be used to provide some biostimulation storage capacity, particularly if a new biostimulation storage reservoir is not included in the project.

Biological Treatment System and Related Supplemental Heating and Chemical Feed Systems

The proposed new MBR system would be configured using existing Plant 1 and Plant 2 basins for reactor basins as described in Section 9 and shown in Figure 9-17.

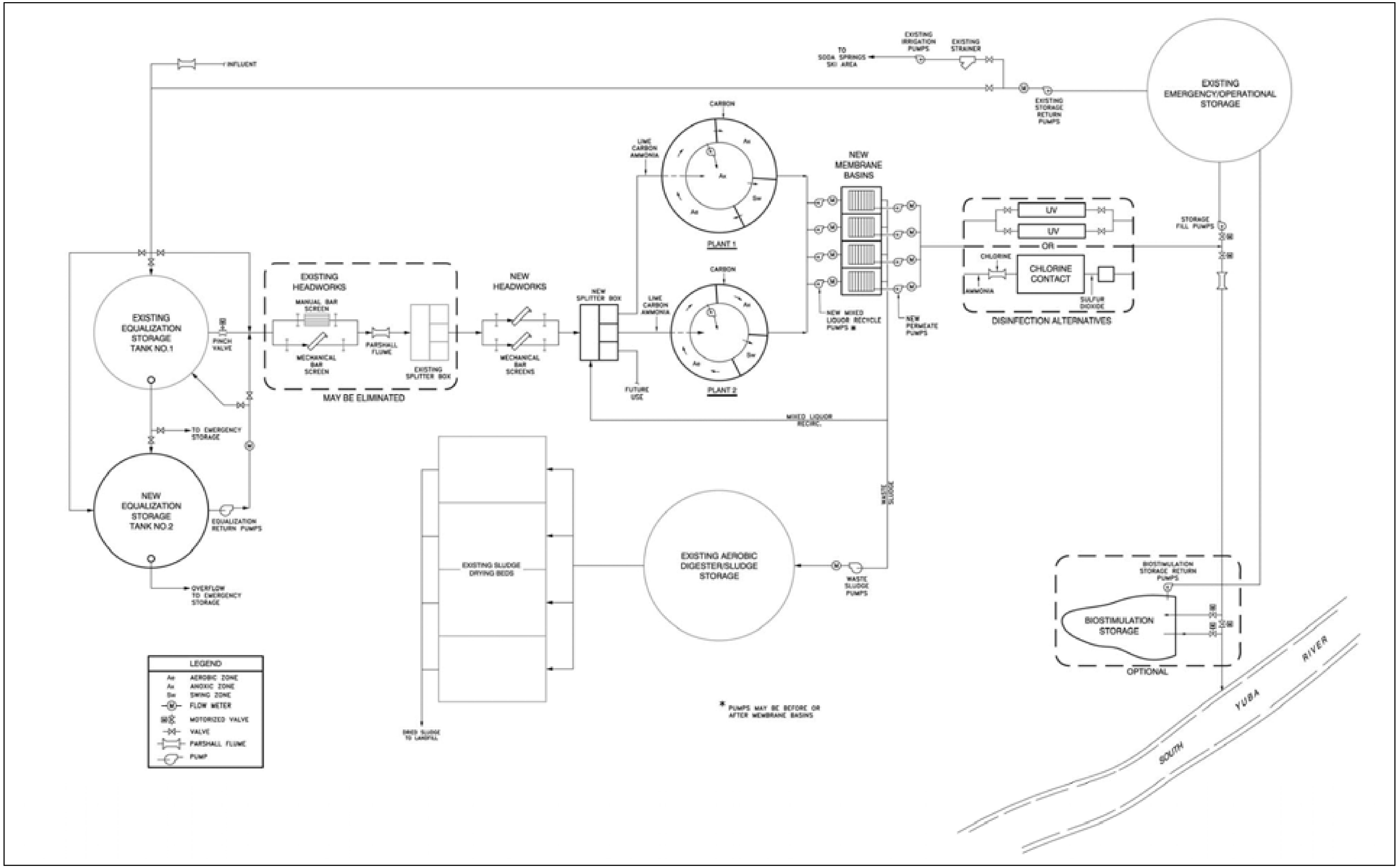


Figure 17-1
Flow Diagram

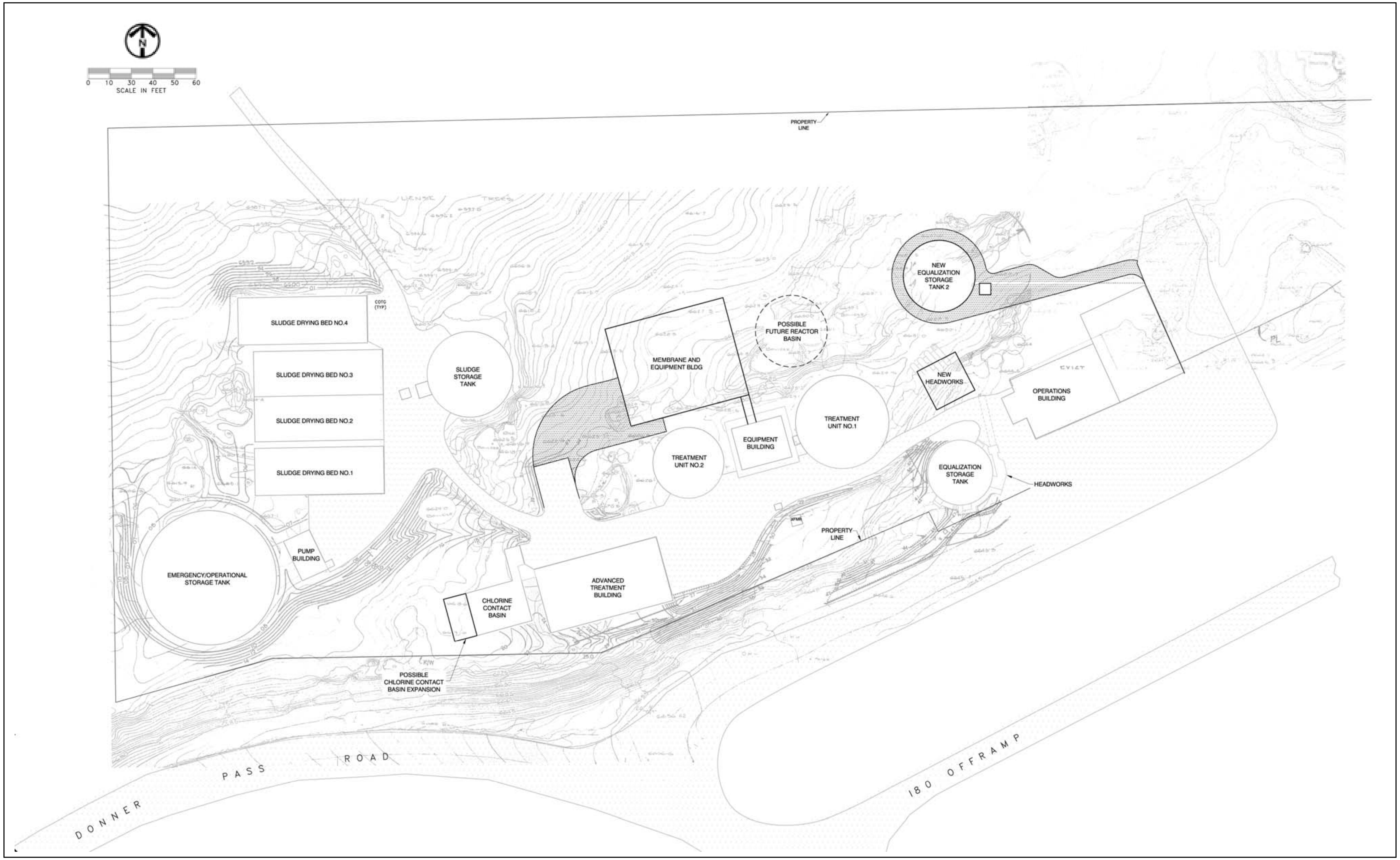


Figure 17-2
Conceptual Layout

New membrane basins and associated pumps, blowers, electrical and other equipment would be located in a new Membrane and Equipment Building as shown conceptually in Figure 17-2. The specific layout of the building would depend on the requirements of the membrane equipment supplier to be selected for this project.

In addition to the equipment specifically associated with the MBR system, the new Membrane and Equipment Building would be used to house the boiler and heat exchanger system recommended to maintain a minimum temperature of 7 °C in the reactor basins. Recirculation pumps would be used to circulate mixed liquor through the heat exchangers. As indicated in Section 9, two 1.0 million Btu/hr systems are proposed.

Depending on what chemical is selected as a carbon source to support denitrification, the chemical feed pumps could also be located in the same building, and potential bulk storage tanks could be located outside the building. If methanol is selected for use, the methanol storage tanks and feed pumps would be in a separate facility that is not shown in Figure 17-2, and would be as described in Section 9.

Additional project features that could be located in the new Membrane and Equipment Building include the ammonia feed facilities (described in Section 9) and the closed vessel UV system, if used (described in Section 11). The final size and layout of the Membrane and Equipment Building would have to be determined during preliminary design when more specific details and required project components are known.

Disinfection

Two disinfection alternatives are still in consideration for this project: chloramination and UV disinfection. Before a decision could be made to implement chloramination, the effectiveness of this process to adequately mitigate disinfection byproducts at DSPUD would have to be tested and proven as previously noted.

If chloramination is selected for implementation, the new ammonia feed facilities associated with this process could be located in the existing Advanced Treatment Building, probably in the room currently occupied by the effluent filtration system, which is to be removed. The chlorine contact basin would have to be expanded as noted in Section 11 and as shown in Figure 17-2.

If UV disinfection is chosen for implementation, the proposed closed cell UV system could be located in the new Membrane and Equipment Building as discussed above.

Emergency/Operational Storage

As discussed in Section 12, the existing Emergency/Operational Storage Tank and ancillary facilities are adequate for the proposed project. No additional volume or other modifications are needed.

Solids Handling

As developed in Section 15, the existing sludge storage tank would be modified with a new mixing and aeration system and a new decanter. It is possible that the pumps and blowers associated with this system could be located in the new Membrane and Equipment Building. Alternatively, a small building could be provided adjacent to the sludge storage tank. The most cost-effective configuration would be developed during preliminary design.

Office and Shop Space

With implementation of the MBR biological treatment process, approximately 1250 square feet of building space currently occupied by the effluent filtration system would be converted to shop space. The filtration system would have to be removed and improved ventilation, lighting, and other features provided in accordance with desired functionality to be determined during preliminary design.

If UV disinfection is chosen, the approximate 430 square feet currently occupied by chlorine and sulfur dioxide storage and feed equipment would be converted to office space. Again, demolition of existing facilities would be required, followed by upgrades needed to create the office space desired.

If chloramination is selected for implementation, the additional office space desired could be provided in the proposed new Membrane and Equipment Building. Alternatively, the existing Advanced Treatment Building could be expanded on the east side.

Biostimulation Storage, Irrigation and Related Facilities

As developed previously in this Section, it is not known whether a biostimulation storage reservoir and related facilities are to be included in this project. If biostimulation storage is implemented, a spray irrigation disposal system expansion would also be needed to dispose of the stored effluent. The biostimulation storage reservoir and irrigation improvements, if desired, would be as described in Sections 13 and 14, respectively.

17.2.2 PROJECT COST ESTIMATE

Overall project capital cost estimates for four alternative projects are presented in Table 17-3. The four alternatives cover both chloramination and UV disinfection, with and without biostimulation storage and irrigation facilities. All of the base costs (before markups) indicated in Table 17-3 are taken directly from the Sections of this report dealing with the specific improvements involved, with two exceptions: 1) the costs for office and shop space are not covered in another section of this report, and 2) the cost for chloramination is based on a modification of the costs indicated in Section 11 (Table 11-4). For chloramination, the costs associated with the river diffuser, gaging station and special studies needed to attain dilution credits were eliminated and the cost for the ammonia feed system needed for chloramination was added.

Table 17-3
Alternative Project Cost Estimates

Item	Cost (a), $			
	With Biostimulation Storage and Irrigation		Without Biostimulation Storage and Irrigation	
	MBR, UV	MBR, Chloram.	MBR, UV	MBR, Chloram.
Wastewater Treatment Plant				
Existing Equalization Facilities Modifications	180,000	180,000	180,000	180,000
New Equalization Storage Tank and Ancillary Facilities	770,000	770,000	770,000	770,000
New Headworks / Fine Screens	600,000	600,000	600,000	600,000
Modify Plants 1 and 2 Basins	100,000	100,000	100,000	100,000
New Membrane Basins	330,000	330,000	330,000	330,000
New MBR System Equipment, Installed	1,900,000	1,900,000	1,900,000	1,900,000
Building for MBR and Related Equipment	950,000	950,000	950,000	950,000
Secondary Process Equipment Not Included in MBR Pkg.	440,000	440,000	440,000	440,000
MBR Internal Process Piping	300,000	300,000	300,000	300,000
Secondary Process Supplemental Heat System	739,000	739,000	739,000	739,000
Ammonia Feed System Modifications	175,000	175,000	175,000	175,000
Methanol Storage and Feed System	225,000	225,000	225,000	225,000
Soda Ash Feed System Modifications	20,000	20,000	20,000	20,000
Chlorine and Sulfur Dioxide System Modifications, Chloram.		312,000		312,000
Expand Chlorine Contact Basin		60,000		60,000
UV Disinfection Structures	160,000		160,000	
UV Disinfection Equipment, Installed	540,000		540,000	
Modify Existing Sludge Storage Tank	232,000	232,000	232,000	232,000
Shop/Office Space	75,000	140,000	75,000	140,000
New Standby Power System in Building	300,000	300,000	300,000	300,000
Subtotal 1, Wastewater Treatment Plant	8,036,000	7,773,000	8,036,000	7,773,000
Electrical and Instrumentation at 25% of Subtotal 1	2,010,000	1,940,000	2,010,000	1,940,000
Sitework @ 5% of Subtotal 1	400,000	390,000	400,000	390,000
Site Piping @ 10% of Subtotal 1	800,000	780,000	800,000	780,000
Subtotal 2, Wastewater Treatment Plant	11,246,000	10,883,000	11,246,000	10,883,000
Remote Facilities				
Biostimulation Storage and Ancillary Facilities	2,626,000	2,626,000		
Expand Spray Irrigation Disposal System	475,000	475,000		
Subtotal 3, Remote Facilities	3,101,000	3,101,000	0	0
Subtotal 4, Wastewater Treatment Plant and Remote Facilities	14,347,000	13,984,000	11,246,000	10,883,000
General Conditions, Overhead and Profit @ 20% of Subtotal 4	2,250,000	2,180,000	2,250,000	2,180,000
Subtotal 5	16,597,000	16,164,000	13,496,000	13,063,000
Contingencies @ 20% of Subtotal 5	3,320,000	3,230,000	2,700,000	2,610,000
Total Construction Cost	19,917,000	19,394,000	16,196,000	15,673,000
Engineering, Administration and Environmental @ 25%	4,980,000	4,850,000	4,050,000	3,920,000
Total Project Cost	24,897,000	24,244,000	20,246,000	19,593,000
Escalated Total Project Cost (b)	26,420,000	25,730,000	21,490,000	20,790,000

(a) First-quarter 2010 cost level, ENR 20-Cities CCI = 8700, except as noted below.
(b) Escalated construction cost based on assumed inflation rate of 2% per year for three years to the estimated mid-point of construction, ENR 20-Cities CCI = 9233.

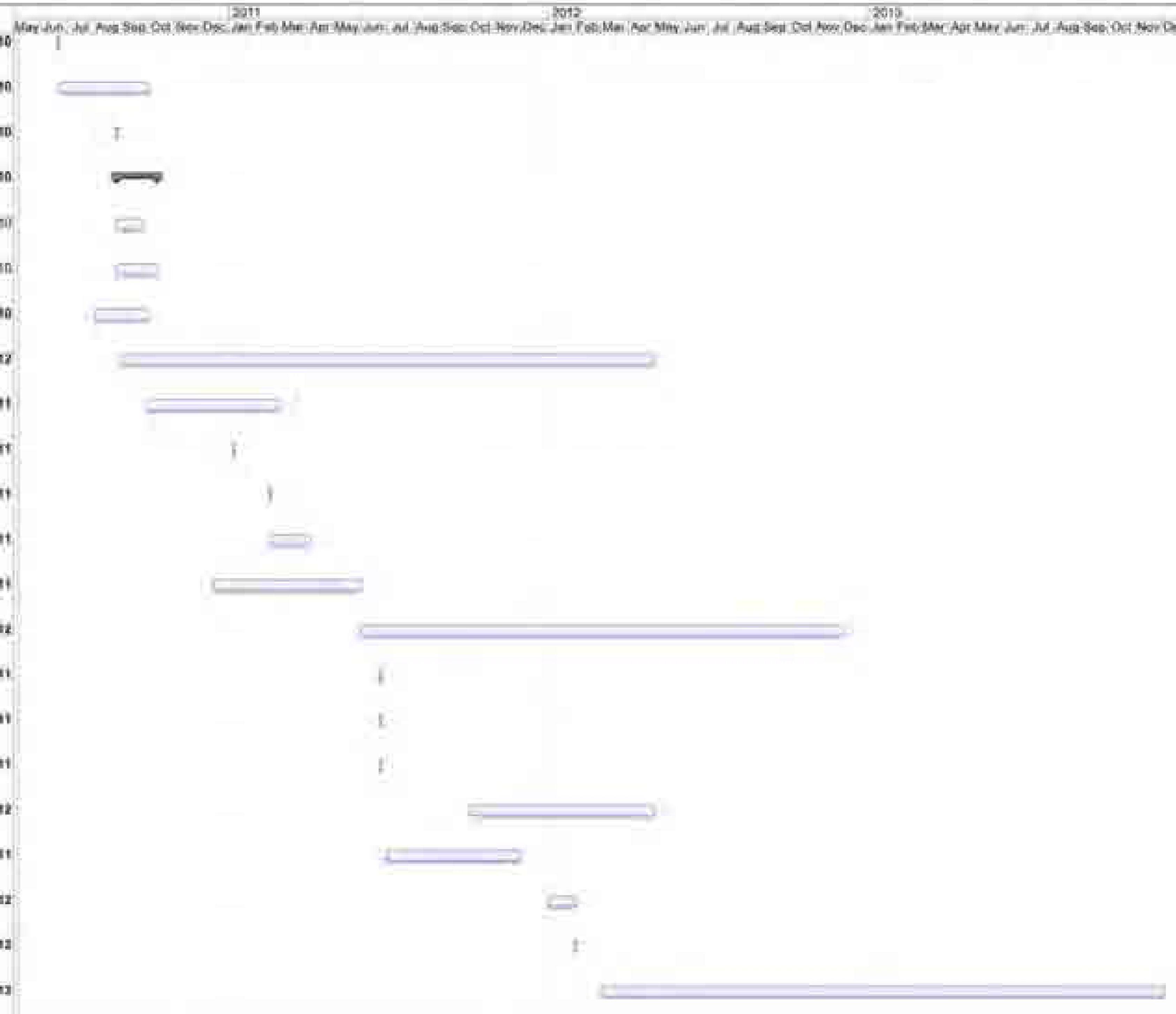
The costs developed in this study and shown in the upper portion of Table 17-3 are based on the cost level in the first quarter of 2010. In the bottom line of the table, the total cost is escalated to the estimated cost level at the mid-point of construction, which is currently anticipated to be early in the year 2013. Of course, the estimated inflation rate of 2 percent per year is subject to much uncertainty.

As indicated in the Table 17-3, the additional cost of UV over chloramination is about $700,000. However, this cost difference would be reduced by the cost of testing and proving the effectiveness of chloramination. The cost difference resulting from the addition of biostimulation storage and an irrigation system expansion is about $4.9 million. These cost differences are based on the escalated costs indicated in Table 17-3.

17.3 IMPLEMENTATION SCHEDULE

An implementation schedule for the proposed project is shown in Figure 17-3.

ID	Task Name	Start	Finish
1	Consultants Received NTP from DSPUD	Mon 6/21/10	Mon 6/21/10
2	Topographic Survey	Wed 6/23/10	Fri 10/1/10
3	CEQA Project Description, NOP and NEPA NOI	Fri 8/27/10	Fri 8/27/10
4	CEQA/NEPA Public Noticing Periods	Fri 8/27/10	Mon 10/11/10
5	CEQA NOP 30-day Public Review	Fri 8/27/10	Sun 9/26/10
6	NEPA NOI 45-day Public Review	Fri 8/27/10	Mon 10/11/10
7	CEQA/NEPA Field Surveys and Studies	Mon 8/2/10	Fri 10/1/10
8	Environmental Permitting	Wed 9/1/10	Tue 5/1/12
9	Dilution/Mixing Study	Fri 10/1/10	Tue 3/1/11
10	Development of Administrative Draft EIR/EIS	Fri 1/7/11	Fri 1/7/11
11	Development of Public Draft EIR/EIS	Fri 2/18/11	Fri 2/18/11
12	CEQA/NEPA EIR/EIS 45-day Public Review Periods	Fri 2/18/11	Mon 4/4/11
13	Pre-Design - Pre-Design and Equipment Pre-Selection Complete	Wed 12/15/10	Wed 6/1/11
14	Revenue and Financing Program (Prop 218 process complete)	Wed 6/1/11	Mon 12/3/12
15	Develop Final EIR/EIS, Mitigation Monitoring and Reporting Plan, Findings Statement, Notice of Determination	Fri 6/24/11	Fri 6/24/11
16	Develop State Board Division of Financial Assistance SRF ERU Environmental Compliance Support Services (if needed)	Fri 6/24/11	Fri 6/24/11
17	USDA Environmental Compliance Reporting and Consultations (if needed)	Fri 6/24/11	Fri 6/24/11
18	WER Studies (as needed)	Mon 10/3/11	Tue 5/1/12
19	Design	Fri 7/1/11	Thu 12/1/11
20	Bid Period	Mon 1/2/12	Thu 2/2/12
21	Contractor Selection and Notice to Proceed	Wed 2/1/12	Wed 2/1/12
22	Construction	Thu 3/1/12	Wed 12/4/13



Project: Facilities Plan Schedule for to
Date: Thu 5/6/10

Task | Progress | Summary | External Tasks | Split
Split | Milestone | Project Summary | External MileTask

Figure 17-3
Proposed Project Schedule

Appendix A

TM. No. 1 - Design Flows and Loads

ECO:LOGIC
ENGINEERS · CONSULTANTS

Donner Summit Public Utility District WWTP
Technical Memorandum No. 1

Design Flows and Loads

Prepared By: Jeffrey R. Hauser, P.E.

Reviewed By: Akram Botrous, Ph.D., P.E.
David Price, P.E.

Date: February 3, 2008 with updates through November 11, 2009

1.1 PURPOSE

The purpose of this technical memorandum is to analyze historical data and develop design flows and loads for the Donner Summit Public Utility District (DSPUD) Wastewater Treatment Plant (WWTP). The remainder of this technical memorandum is organized into the following major sections:

- Historical Plant Data and Modifications for this Analysis
- Analysis of Historical Flows
- Analysis of Historical Constituent Concentrations and Loads
- Estimate of Future Users Flows and Loads
- Summary of Existing and Future Flows and Loads

1.2 HISTORICAL PLANT DATA AND MODIFICATIONS FOR THIS ANALYSIS

In general, five separate plant data sources were used for this analysis as set forth below:

- Monthly Self Monitoring Reports received as electronic (Excel) files for the months of January 2002 through September 2007. These were used for influent flows, influent biochemical oxygen demand (BOD_5 or simply BOD), and for influent total suspended solids (TSS). The BOD and TSS data from these reports were for composite samples collected from Monday morning through Tuesday morning and from Wednesday morning through Thursday morning each week.

- Daily Wastewater Treatment Plant Lab Data Sheets for the months of January 2007 through September 2007. These were used for influent ammonia and alkalinity data, based on grab samples from the equalization basin effluent flow taken on Tuesday and Thursday morning each week.

- Commercial laboratory reports for special influent monitoring completed for and subsequent to the DSPUD/SLCWD Joint Engineering Study on Wastewater Flows and Loads, dated June 10, 2004, by ECO:LOGIC Engineering in cooperation with Dewante and Stowell. The special monitoring was conducted on weekends between August 2003 and March 2005, and on weekdays on August 8, 2003 and between January 2005 and April 2005. Parameters monitored for include BOD, chemical oxygen demand (COD), TSS, total Kjeldahl nitrogen (TKN), and alkalinity.

- The electronic file (Excel) tabulation of plant data from October 2002 through July 2006 completed by Dr. Curtis McDowell of Brentwood Industries. This was used only for influent ammonia data for the period indicated. The influent ammonia data was based on grab samples of the equalization basin effluent taken each morning. Dr. Curtis obtained these data from Daily Wastewater Treatment Plant Lab Data Sheets, which were provided to him.

- Data from special monitoring conducted for this analysis in January and February 2008, including on-site testing results, as well as commercial laboratory results.

In evaluating the plant data, it is critical to know what procedures were used for tabulating data on various dates. In general, plant influent flows, BOD, and TSS data for the self monitoring reports were handled as follows:

- Daily influent flows were calculated based on influent flow meter totalizer readings taken at about 8 AM every morning. The flow reported on a specific date was calculated as the totalizer reading taken the morning after that date minus the totalizer reading on that date. Therefore, the flow reported on any given date is actually the volume of wastewater received from 8 AM on that date to 8 AM on the next day.

- Influent BOD and TSS data for the self monitoring reports are based on 24-hour composite samples of influent equalization basin outflow collected (as noted above) from Monday morning through Tuesday morning and from Wednesday morning through Thursday morning, each week. From January 2002 through January 2004, these data were reported on the date the composite samples were completed, i.e., Tuesday and Thursday. After January 2004, the data were reported on the date the composite sample was started, i.e., Monday and Wednesday. Adjustments to the reported dates for this analysis are discussed below.

It is important that wastewater constituent data be properly coordinated with flow data. For example, it is important to know that BOD results from a sample collected over a certain period of time are associated with a flow for the same period of time. Otherwise, the total mass or load of BOD entering the plant on that day will be calculated incorrectly. Because of this issue, all of the BOD and TSS data included in the Self Monitoring Reports from January 2002 through January 2004 were shifted back one day for this analysis. That is, a value recorded on a given date was moved to the previous date. Additionally, for the same reason, all of the ammonia data from the Wastewater Treatment Plant Lab Data Sheets, including those tabulated by Dr. McDowell, were shifted back one day. This is because ammonia samples taken from the equalization basin effluent in the morning are most representative of the flow reported through that same morning and logged on the previous day. Similarly, all of the special influent monitoring data was associated with plant flows logged the day before the composite sample was collected and taken to the laboratory for analysis.

1.3 ANALYSIS OF HISTORICAL FLOWS

Historical daily influent flows for the period from January 2002 through September 2007 are presented in Figure 1-1. In addition to the daily data, which are shown as discrete data markers, rolling 7-day and 30-day average flows are shown as colored lines. The rolling averages are trailing averages, meaning the value shown on a specific date is the average of the data for the indicated number of days preceding and including that date.

In addition to the influent flow data, precipitation data are shown in Figure 1-1 to illustrate the dependency of flow on precipitation. Separate curves are shown for daily precipitation, 30-day average precipitation, and cumulative precipitation beginning in September of each year. The precipitation data are for the Truckee Ranger Station and were obtained from the National Climatic Data Center (NCDC, part of the United States Department of Commerce, National Oceanic and Atmospheric Administration). Each daily precipitation amount provided by NCDC represents the precipitation from 8 AM on the previous date to 8 AM on the date indicated. Therefore, to correspond to the convention used for dating flow data at the DSPUD WWTP, all of the precipitation data were shifted back one day (shifted to one day earlier).

As indicated in the Figure, the influent flow to the DSPUD WWTP is highly seasonal, with peak flows occurring in the winter and spring and the lowest flows occurring in the late fall. The winter peak flows that typically occur in the period from December through February and sometimes March are largely the result of high occupancies and large numbers of day visitors to the ski resorts in the service area during the ski season. The high flows that occur generally in May and June are the result of elevated infiltration and inflow (I/I) into the sewage collection system during the spring snowmelt.

Ending dates are shown in Figure 1-1 for the highest 7-day (weekly) average flows occurring in the ski season. As can be noted from the dates, the highest weekly average flows in the ski season usually occur around the period from Christmas to New Years Day. Because of the peak flows that occur during this period and because average flows leading up to this time are substantially lower, so the peak is a “shock” condition, the Christmas / New Year’s period at DSPUD represents one of the most challenging conditions for WWTP design and operation. Additional weekly average flow peaks occur around the time of spring break from schools (late March or early April) and around the time of the Presidents’ Day weekend (late February).

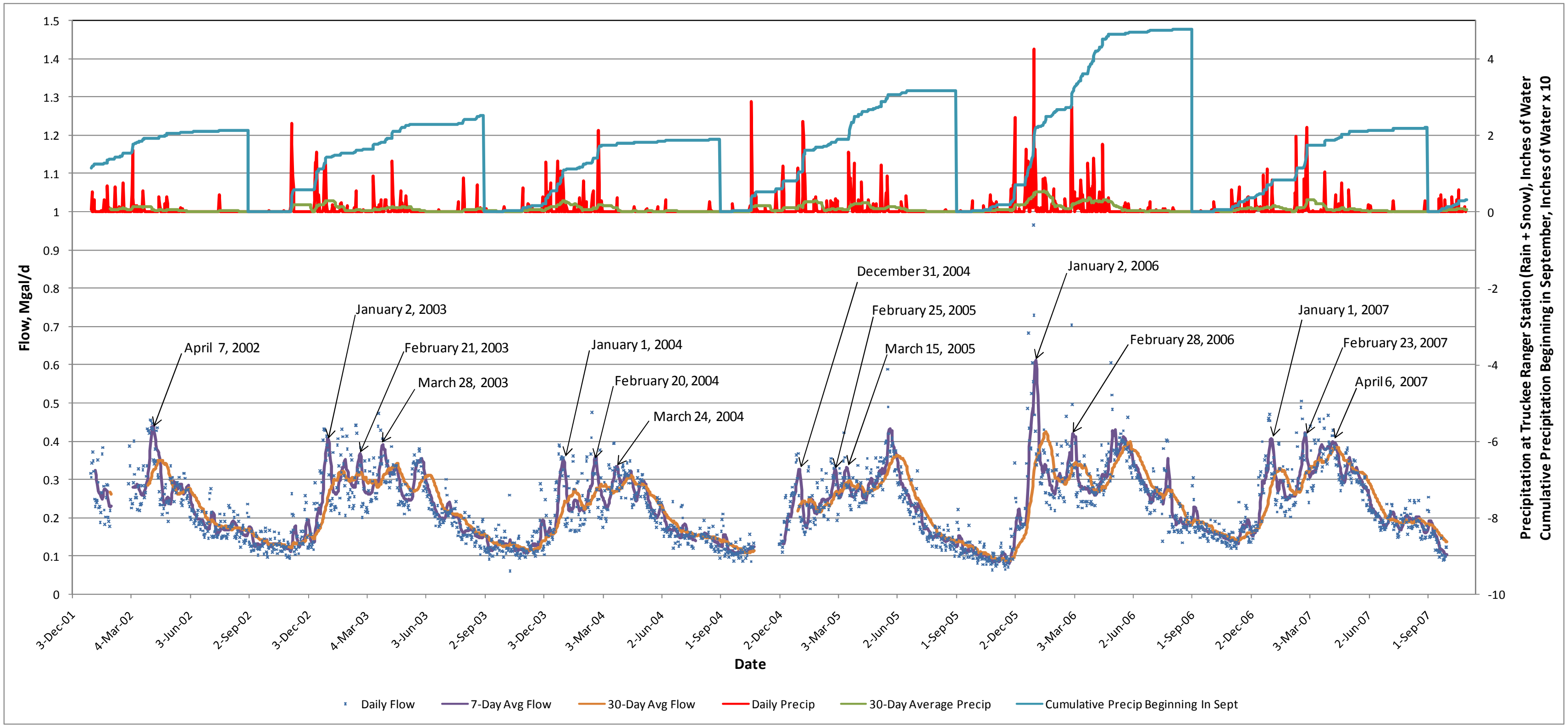


Figure 1-1
Influent Flows and Precipitation 2002 - 2007

The impact of precipitation on plant flows is clearly illustrated in Figure 1-1. The highest weekly average flow of 0.61 Mgal/d ending on January 2, 2006 was apparently the result of heavy precipitation in the previous days, almost all of which fell as rain, not snow. There was a total of 14.4 inches of precipitation in a period of 16 days through January 1, 2006, including 4.25 inches recorded on December 30, 2005, and 1.64 inches recorded on January 1, 2006 (NCDC dates shifted back one day as previously noted). The maximum daily wastewater treatment plant influent flow measured during 2002 through 2007 was 0.967 Mgal/d, recorded on December 30, 2005, on the same day as the peak day precipitation of 4.25 inches. Based on depth-duration-frequency data available from the California Data Exchange Center (CDEC), the daily rainfall amount of 4.25 inches was an 18-year return frequency event. The 16-day rainfall total of 14.4 inches was about an 11-year return frequency event.

1.3.1 MONTHLY AVERAGE FLOWS AND VARIATIONS FROM YEAR TO YEAR

To further illustrate the seasonal flow variations, to indicate year-to-year variability and to provide data needed for water balance calculations to be used for evaluation of dry season land disposal requirements, a trailing 30-day average flow (essentially a monthly average flow for the month in question) was calculated for the last day of each month. The results are shown in Table 1-1 and in Figure 1-2.

From the data shown in Figure 1-2, it is clear that the flows can vary substantially from year to year, undoubtedly as the result of differing weather conditions and the impacts of those conditions on transient populations within the service area and on infiltration and inflow. For example, the flows for the spring and early summer of 2004 were low compared to corresponding flows for other years, while the flows for the spring, summer, and fall of 2006 were high. These flow variations seem to correlate with the cumulative precipitation occurring since September shown in Figure 1-1. The annual precipitation total of 47.7 inches that fell from September 1, 2005 through August 31, 2006 was an 18-year return frequency event.

The Regional Water Quality Control Board typically requires that 100-year return frequency precipitation be used in the design of land disposal systems. To estimate the impact of 100-year return frequency precipitation on wastewater flows during the summer months when land disposal would be practiced, the average flow during July through September was plotted against the annual precipitation total beginning in the previous September, as shown in Figure 1-3. A linear least squares trendline was used to allow extrapolation to the 100-year return frequency annual precipitation of 58.6 inches. Based on the trendline, it is estimated that the influent flows during the summer months with 100-year return frequency precipitation could be about 5 percent higher than the actual flows in the summer of 2006. It is recognized that this estimate is based on limited data and is only approximate. It is also recognized that the actual flows that occur in any given summer are the result of many factors, and annual precipitation since the previous September is only one of the factors. Other factors would include the status of the collection system as regards infiltration and inflow mitigation measures that might be taken by DSPUD or SLCWD, average precipitation over several years, the type (rain or snow) and timing of precipitation and the conditions during the spring snowmelt. Despite all the uncertainties involved, it is suggested that using 105 percent of actual 2006 summer flows provides a reasonable estimate of summer flows that might occur with 100-year return frequency annual precipitation with the existing level of development at Donner Summit. Future flow estimates would have to include an allowance for future growth.

Table 1-1
Monthly Average Flows

Month	30-Day Average Flow Calculated on the Last Day of Each Month, Mgal/d					
	2002	2003	2004	2005	2006	2007
Jan	0.264	0.302	0.235	0.216	0.313	0.277
Feb		0.301		0.269	0.336	0.321
Mar	0.291	0.314	0.279	0.282	0.280	0.365
Apr	0.325	0.297	0.289	0.280	0.325	0.349
May	0.278	0.306	0.235	0.364	0.379	0.298
Jun	0.197	0.227	0.169	0.256	0.286	0.200
Jul	0.176	0.199	0.157	0.176	0.272	0.193
Aug	0.170	0.160	0.142	0.139	0.184	0.183
Sep	0.138	0.127	0.120	0.122	0.180	0.138
Oct	0.126	0.122		0.094	0.149	
Nov	0.151	0.141		0.099	0.162	
Dec	0.234	0.220	0.228	0.337	0.278	
Avg	0.214	0.226	0.206	0.219	0.262	0.258

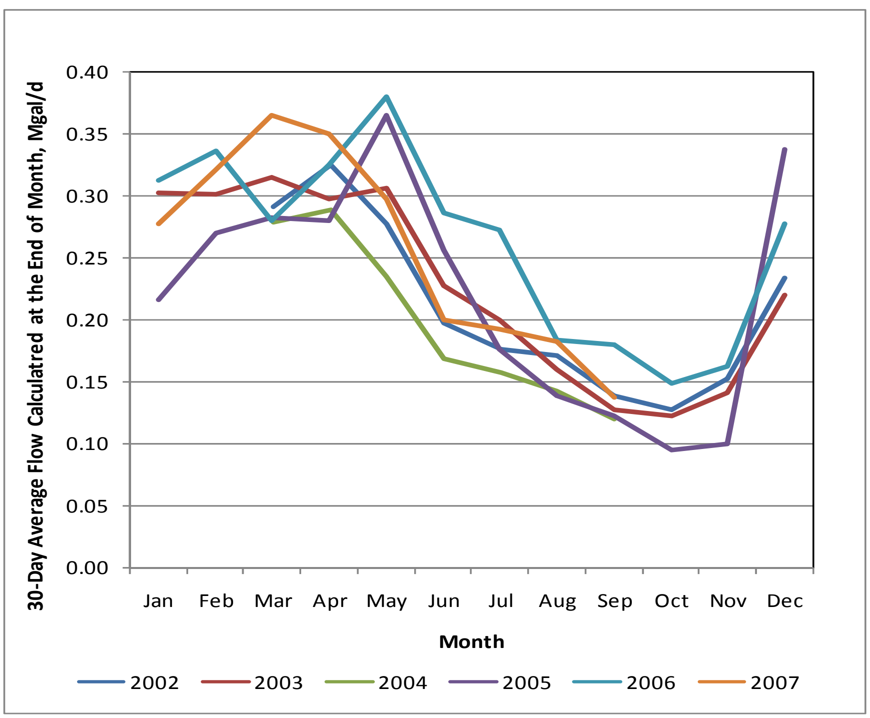


Figure 1-2
Monthly Average Flows By Year

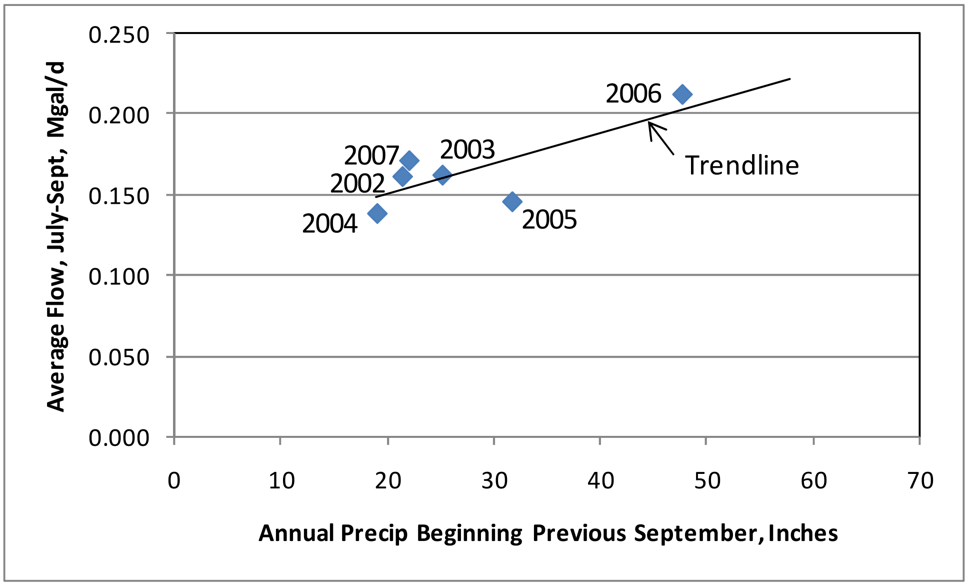


Figure 1-3
July-September Flow versus Annual Precipitation

1.3.2 Flow Variations By Day of the Week

Since Donner Summit is a resort community, it is typical to have higher occupancies and transient populations on weekends than on weekdays. This phenomenon was evaluated by analyzing median flows for each day of the week over the entire period of record. Additionally, because the ski season is of particular importance for DSPUD, median flows for each day of the week for the months of December through February (the prime ski season) were also calculated. The results are shown in Figure 1-4. As indicated, in the ski season, weekend flows are typically around 150 percent of midweek flows, while for the year as a whole, weekend flows are typically around 125 percent of midweek flows. This phenomenon is very important when considering an influent monitoring program to correctly represent the DSPUD wastewater characteristics. This topic is discussed further later in this memorandum.

1.3.3 Biological Process Design Flows for Existing Conditions

For the design of biological treatment processes at DSPUD, it is most important to assess the flows that would occur at times of peak occupancy during the ski season. Although flows during the spring snowmelt can be as high or higher than those in the peak of the ski season, the springtime flows would generally be very dilute due to the combined effects of high infiltration and inflow into the sewage collection system and very low occupancies and transient populations at this time of the year. As a result of the low loading conditions, the springtime flows are less critical for biological process design.

In addition to quantifying the wintertime peak flows, it is important to assess the variability in flows leading up to the peak and after the peak. As mentioned previously, the Christmas / New Years holiday period is a particularly challenging design condition. Similar peak flow events occurring around other ski season holiday periods are also important.

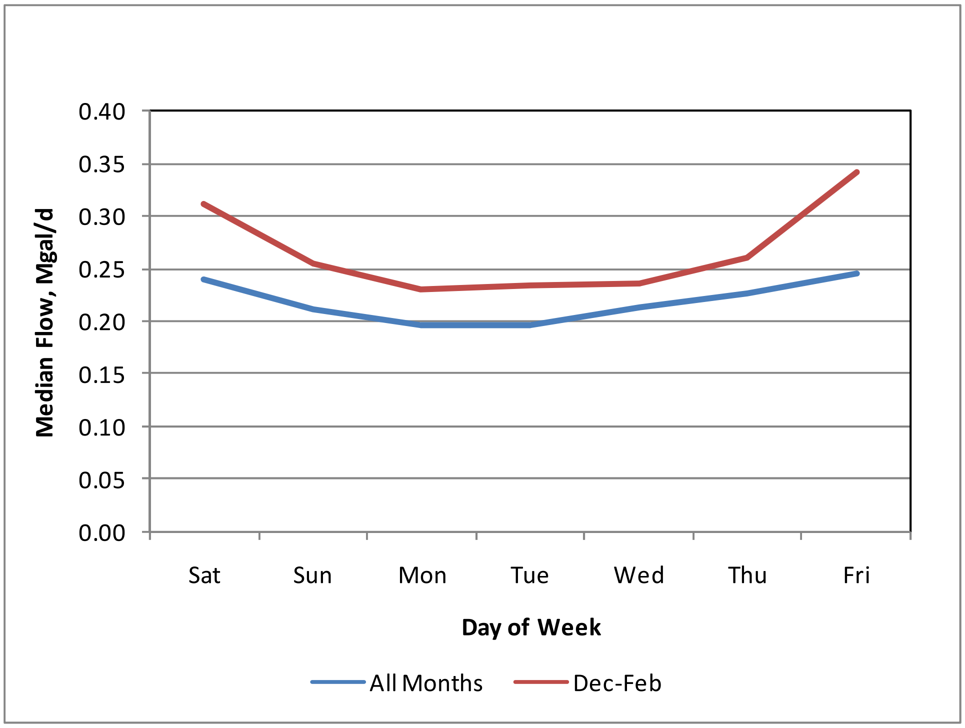


Figure 1-4
Median Flow by Day of Week for 2002 – 2007

In communities with more stable flows and loads than at Donner Summit, the "nameplate capacity" of a wastewater treatment plant is typically based on average dry weather flows and biological process basin sizing is typically based average day maximum monthly conditions. At Donner Summit, however, it is more appropriate to use a critical winter peak condition to establish the nameplate capacity of the plant and it is necessary to consider shorter-term (compared to monthly) peak flow and load conditions for biological process basin sizing. In the design of the 1985 improvements, the peak three-day flow and load condition during the ski season was used to characterize plant design requirements. Based on biological growth kinetics, it is now believed that a weekly average might be more appropriate. In reality, however, it is of little importance whether the peak week or peak three days is used as a basis of assigning a nameplate capacity for the plant. Regardless of the duration used to assign a nameplate capacity, the plant design must take all flows and loads into consideration. In fact, Dynamic simulations of the variable flows and loads occurring over several weeks or months will be needed for proper plant design analyses.

Based on the data shown in Figure 1-1 and the discussions above, the following are believed to represent appropriate biological process design flows for current conditions (not including any allowances for future growth) at DSPUD:

Typical Average Annual Flow (AAF) (This is the average flow for all of 2002-2006)	0.23 Mgal/d
Average Day Maximum Monthly Flow (ADMMF) (See discussion below)	0.35 to 0.43 Mgal/d
Average Day Maximum Weekly Flow (ADMWF) (See discussion below)	0.43 to 0.61 Mgal/d
Peak Day Flow (PDF)	0.97 Mgal/d

In the listing above, a range of flows is given for the ADMMF and the ADMWF. In each case, the lower value represents data from several events over the period studied. The higher value represents data from the rather unusual event occurring as the result of abnormally high precipitation in late December 2005 and early January 2006. In reviewing the influent BOD load data presented later in this document, it can be noted that the BOD loads in the 2005/2006 peak flow events were similar to those in other peak load events. Since the flows were higher in the peak events of 2005/2006, however, the concentrations were lower. Therefore, the range of flows is important and it will be necessary for the plant design to accommodate both the higher flows at a lower concentration and the lower flows at a higher concentration (both with about the same load).

By review of Figure 1-1, it can be seen that peak monthly average and peak weekly average flows in the spring snowmelt period are similar to those in the winter ski season to be used for biological process design, as listed above.

1.3.4 MAXIMUM PLANT HYDRAULIC DESIGN FLOWS FOR EXISTING CONDITIONS

In general, influent flows to the DSPUD WWTP are taken in to an equalization basin and are released to downstream treatment units at a controlled rate. Thus, in the case of current conditions, flows in excess of the peak day flow of 0.97 Mgal/d should essentially never be passed through the treatment plant. However, it is prudent for the plant design to provide for the ability to pass through the plant any flow that could reasonably be expected through the influent sewer. Thus, in the event that the equalization basin was prematurely filled before the peak flow occurred, the peak flow could be passed through the plant to receive some treatment without overflowing basins and spilling onto the ground.

To determine historical peak influent flows, DSPUD staff researched plant records from 2002 through 2006, noting days with unusually high flows. Influent flow recordings were then recovered and reviewed to determine the maximum influent flows on those days. The durations of the peak flows were not specifically evaluated, but they are referred to herein as peak hourly flows. The ten highest peak hourly flows noted in this way are summarized below:

December 30, 2005	1.66 Mgal/d
December 31, 2005	1.64 Mgal/d
December 22, 2005	1.34 Mgal/d
December 28, 2005	1.04 Mgal/d
February 5, 2006	1.04 Mgal/d
December 23, 2005	1.00 Mgal/d
February 4, 2002	1.00 Mgal/d
December 29, 2005	0.98 Mgal/d
January 2, 2006	0.96 Mgal/d
December 27, 2003	0.95 Mgal/d

As indicated in the list above, the peak hourly flows that occurred near the end of December 2005 were unusually high. As previously noted, 4.25 inches of precipitation were recorded in Truckee on December 30, 2005 (date adjusted to match plant records convention), and this was an 18 year return-frequency event. This is believed to be a reasonable basis for assessing design peak hourly flows. Accordingly, after rounding, it is considered appropriate to use 1.7 Mgal/d as the design peak hourly flow for existing conditions. It is interesting to note that 1.7 Mgal/d is the design hydraulic capacity for the existing plant, as noted in the design drawings completed in 1985.

1.4 ANALYSIS OF HISTORICAL CONSTITUENT CONCENTRATIONS AND LOADS

For wastewater treatment plant design, it is essential to have a good understanding of the constituent loadings that the plant will experience. The term load or loading refers to the total mass or weight of a constituent entering the wastewater treatment plant over a specific period of time. Loadings are normally expressed in units of pounds per day. A constituent load for a given time period is determined by multiplying the average flow times the flow-weighted average constituent concentration during that period, and then applying a conversion factor to get the desired units (pounds per day).

The main constituents of concern are the biochemical oxygen demand (BOD_5, or simply BOD) and total Kjeldahl nitrogen (TKN). TKN includes organic nitrogen (typically about 1/3 of the TKN) and ammonia nitrogen (typically about 2/3 of the TKN). Since there is normally no nitrite or nitrate nitrogen in the plant influent, TKN usually comprises the total influent nitrogen. Other influent parameters that are also important include total suspended solids (TSS) and alkalinity. Chemical oxygen demand (COD) can be used instead of or as a supplement to BOD.

As discussed for flows, constituent loadings must be determined for the critical peak winter flow and load events, because it is the loadings during these events that will be most important for evaluation and sizing of wastewater treatment facilities. Of particular concern are the average and peak loadings occurring throughout a sustained peak week condition such as would occur in the Christmas / New Year's period and sometimes around Presidents' Day and spring break from school. Also as discussed for flows, it is important to determine patterns in constituent loadings in the weeks leading up to and after the peak events.

Determination of critical design influent constituent loadings from the plant records at DSPUD is difficult for several reasons:

1. Influent samples are typically collected twice weekly, which is not adequate, for example, to define loading patterns throughout critical peak week conditions.
2. Influent samples have always been collected on weekdays, not on weekends. Just as is the case for flows, it is believed that weekend loads can be much higher than weekday loads.
3. Influent samples are collected from the equalization storage basin, which can contain plant recycle streams, such as filter backwash water, as well as raw sewage.

In the paragraphs below, historical plant data for BOD, TSS, ammonia nitrogen, TKN and alkalinity are discussed.

1.4.1 BIOCHEMICAL OXYGEN DEMAND (BOD)

Mid-week plant influent (actually equalization basin effluent) BOD concentrations and loadings recorded from January 2002 through September 2007 are shown in Figures 1-5 and 1-6, respectively. As shown in Figure 1-5, mid-week influent BOD concentrations vary widely during the year, from lows under 10 mg/L during the spring snowmelt to highs around 400 mg/L in peak occupancy periods. As was done for the flows in Figure 1-1, dates are indicated in Figure 1-6 for key peaks of the weekly average influent BOD loads. As can be noted, the peak loads frequently occur during the Christmas / New Year's period and around the Presidents' Day weekend. In general, as would be expected, influent BOD loads are highest throughout the winter ski season. Intermediate loads occur in the summer and the lowest loads of the year occur in the spring and fall.

As previously stated, the BOD loads shown in Figure 1-6 are based on mid-week samples and do not reflect the effects of peak weekend loads. To assess the degree to which weekend loads might be higher than weekday loads, the BOD results for the weekend monitoring that was done for and subsequent to the 2004 Joint Engineering Study were compared to the corresponding mid-week values from the self monitoring reports, as shown in Table 1-2. It is noted that the last two lines in Table 1-2 are for Saturday and Sunday of the same weekend. As indicated in the table, when both previous and following weekday results are considered, the weekend load ranged from 1.06 to 5.90 to the average weekday load. However, the main concern here is to compare weekend loads to the maximum mid-week loads shown in Figure 1-6. When the weekend BOD load is compared to the maximum weekday average, whether preceding or following the weekend, the result is the minimum ratio of weekend to weekday values shown in the last column of Table 1-2. As indicated, the weekend BOD load ranged from 1.06 to 2.40 times the corresponding maximum weekday average values. However, the low value of 1.06 occurred when there was no sample on the Wednesday, so the weekend value was compared only to the Monday value. Excluding this datum, the ratio of the weekend to maximum weekday BOD load range is 1.21 to 2.40. Even when the Monday included in the weekday day was a holiday (indicated by "**" in the table), the BOD load on that day was typically substantially less than the preceding Saturday or Sunday. As previously noted, the Monday sample was collected from Monday morning through Tuesday morning.

If the average weekday values in Table 1-2 were assumed to represent the average for all five weekdays (a weighting factor of 5) and the weekend value were assumed to represent the average for two weekend days (a weighting factor of 2), then the weekend to weekday BOD load ratio range of 1.21 to 2.4 would correspond to a weekly average BOD load to weekday average BOD load ratio range of 1.06 to 1.4 (based on a weighted average of the weekend and weekday values). Therefore, it is estimated that the true peak week (including weekends and weekdays) average BOD loads may have been 1.06 to 1.4 times the one-week average of mid-week values shown in Figure 1-6. Accordingly, a good estimate of an existing peak week BOD load might be about 780 lb/d (determined as 1.3 x 600 lb/d or about 1.4 x 560 lb/d).

It is believed the discussion above provides the best engineering estimate of peak week BOD loads, based on limited data. If substantially more weekend data were available, a more accurate assessment of critical peak week BOD loads could be made. From this assessment, the importance of sampling on weekends during peak occupancy periods is evident. Accordingly, it is recommended that the weekend sampling during peak occupancy periods should be a regular practice of the District.

1.4.2 TOTAL SUSPENDED SOLIDS (TSS)

Mid-week plant influent (actually equalization basin effluent) TSS concentrations recorded from January 2002 through September 2007 are shown in Figure 1-7. As indicated, the TSS concentrations follow the same general trends as BOD, but the range is even somewhat greater.

The most important aspect of influent TSS concentrations is their values relative to influent BOD concentrations. The ratio of influent TSS to BOD is an important parameter in predicting the sludge yield from a biological treatment process, and the sludge yield is critical for sizing reactor basins. If the ratio is near 1.0 (+/- about 0.1), then typical empirical models for sludge yields in municipal wastewater treatment can be used. If the TSS/BOD ratio is substantially above the normal range, then higher sludge production per unit of BOD removed can be expected, as compared to typical systems.

A chart showing the influent TSS/BOD ratio versus the influent BOD concentration for the period of record analyzed herein (2002 through September 2007) is shown in Figure 1-8. As indicated in the figure, when the BOD concentration is low (such as occurs during high I/I events), the TSS/BOD ratio can be substantially greater than 1. However, when the BOD is higher, the TSS to BOD ratio tends to be near 1. Since it is the high concentrations that occur during peak loading events that govern wastewater treatment process design, it can be considered that the TSS/BOD ratio is near 1 for process design purposes.

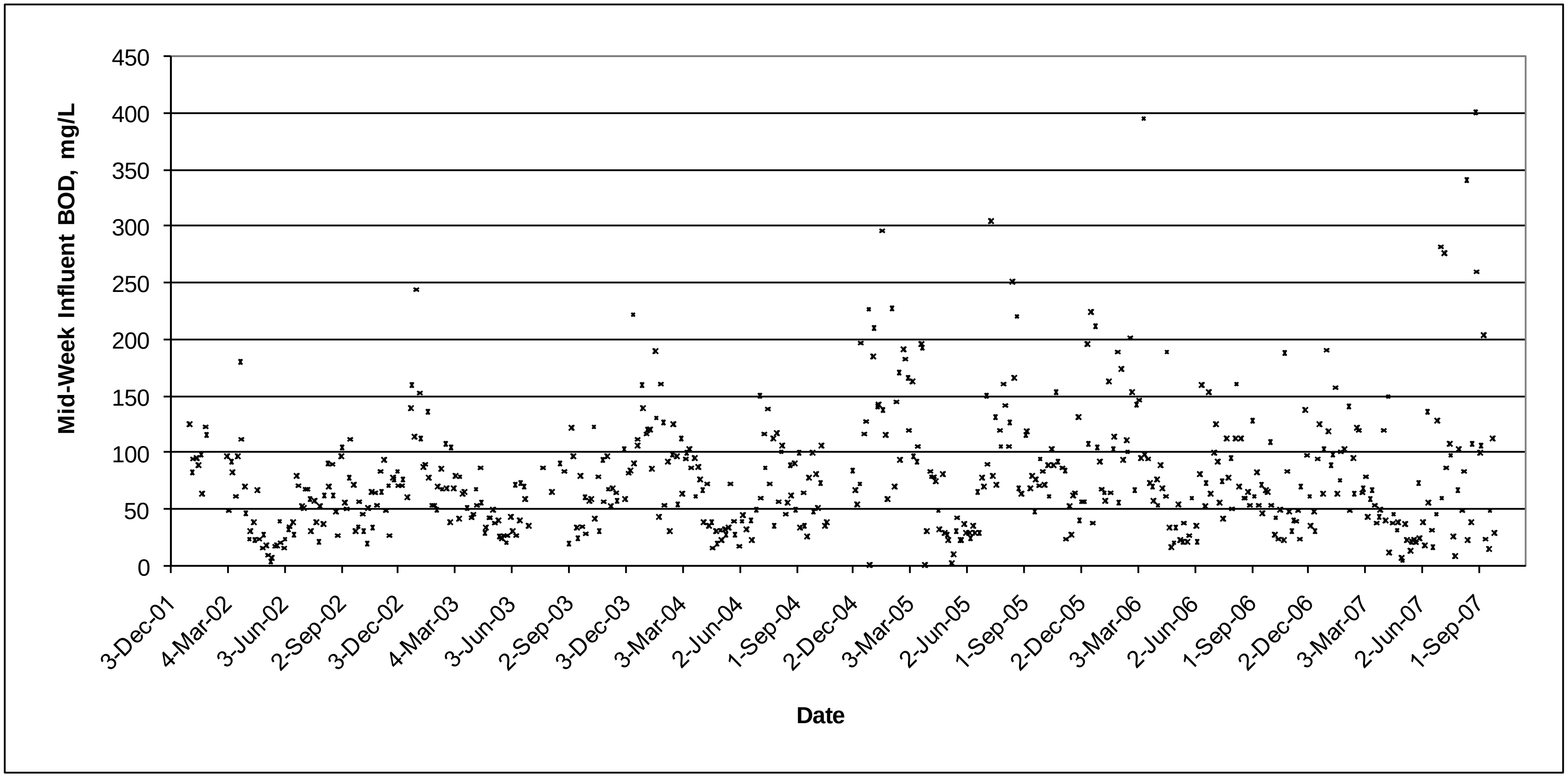


Figure 1-5
Mid-Week Influent BOD Concentrations

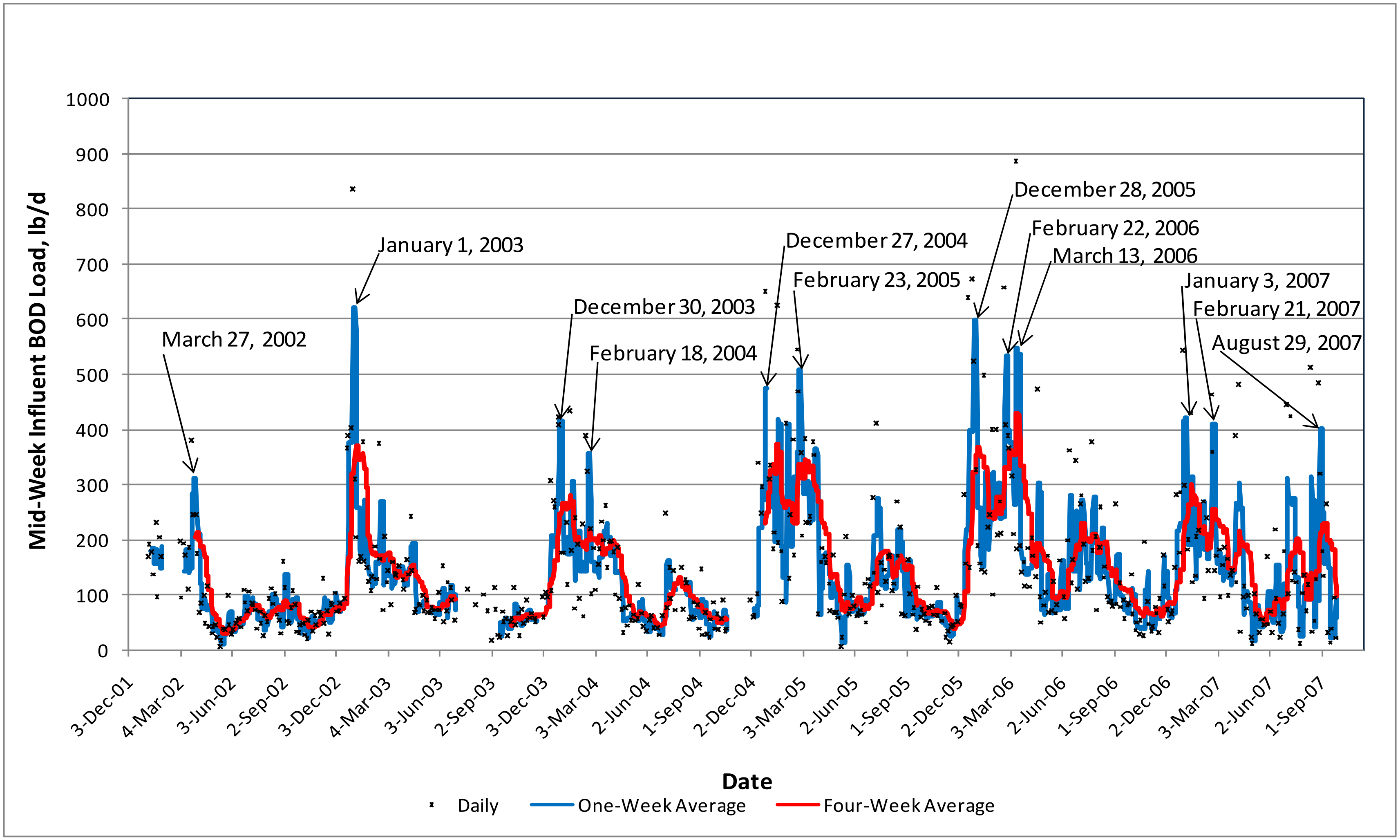


Figure 1-6
Mid-Week Influent BOD Loads

Table 1-2
Weekend versus Weekday BOD Loads for Peak Occupancy Periods

Weekend Day and Date	Weekend BOD Load, lb/d	Previous Weekday BOD Load, lb/d			Ratio Weekend to Prev. Weekday BOD Load	Following Weekday BOD Load, lb/d			Ratio Weekend to Follow Weekday BOD Load	Minimum Weekend to Weekday Ratio
		Mon	Wed	Avg		Mon	Wed	Avg		
Saturday, January 03, 2004	653	423	409*	416	1.57	176	176	176	3.71	1.57
Sunday, January 18, 2004	692	232	119	175.5	3.95	433**	181	307	2.26	2.26
Sunday, February 15, 2004	855	229	61	145	5.90	389**	324	356.5	2.40	2.40
Saturday, January 01, 2005	689	650	---	650	1.06	310	335	322.5	2.14	1.06
Saturday, January 15, 2005	496	183	213	198	2.50	625**	195	410	1.21	1.21
Saturday, February 19, 2005	723	381	172	276.5	2.62	545**	469	507	1.43	1.43
Sunday, February 20, 2005	770	381	172	276.5	2.78	545**	469	507	1.52	1.52

* Sample on Tuesday instead of Wednesday
** Holiday

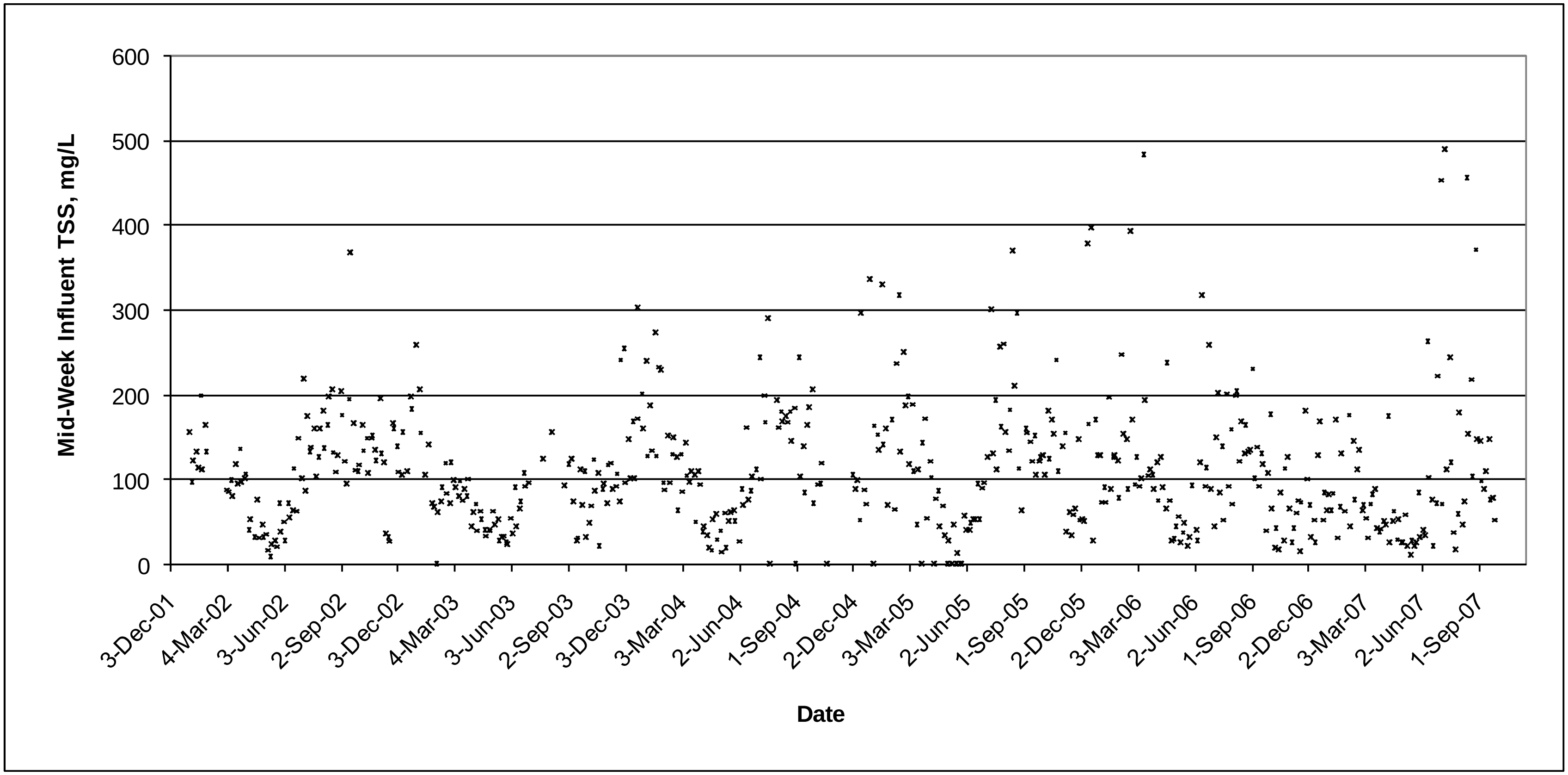


Figure 1-7
Mid-Week Influent TSS Concentrations

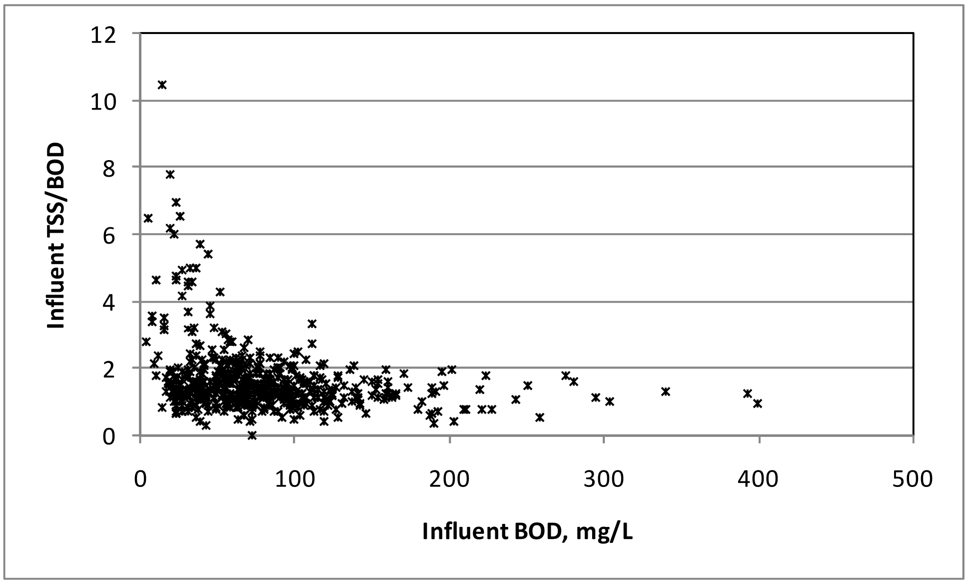


Figure 1-8
Influent TSS/BOD Ratio versus Influent BOD

1.4.3 TOTAL KJELDAHL NITROGEN AND AMMONIA-NITROGEN

As previously mentioned, nitrogen entering the wastewater treatment plant is generally in the form of organic nitrogen and ammonia nitrogen. These two forms of nitrogen together are measured as TKN. Ideally, there would be a long-term database of influent TKN values, however, TKN is not routinely measured at DSPUD. Therefore, TKN values must be assessed based on limited data from special sampling completed for and subsequent to the 2004 Joint Engineering Study and based on ammonia-nitrogen data. The ammonia-nitrogen data are considered first below.

As was the case for TSS, the most important aspect of TKN and ammonia-nitrogen concentrations is their values relative to BOD. Although a long-term database of influent ammonia-nitrogen concentrations is available for comparison with influent BOD data, the data are not from the same samples. Influent BOD data are based on composite samples, each collected from one morning to the next. Influent ammonia-nitrogen data are based on grab samples, each taken on the same morning that the composite sample for BOD was ended. However, since the grab samples for ammonia-nitrogen were taken from the influent equalization basin outflow, substantial compositing was accomplished by means of residence time and mixing in the equalization basin.

For this study, influent ammonia data were obtained for the period from October 2002 through July 2006 (from the McDowell data tabulation) and for January through September 2007. Based on these data, the ratios of influent ammonia-nitrogen to influent BOD versus the influent BOD concentrations are shown graphically in Figure 1-9. As for TSS, there is considerable data scatter at the lower BOD values. In Figure 1-10, the same ratios are shown only for BOD concentrations greater than 150 mg/L. The values occurring with higher BOD concentrations are most important for process design. Based on Figure 1-10, the corresponding ratios of influent ammonia-nitrogen to influent BOD ranged from about 0.05 to 0.25. A typical value for domestic wastewater is about 0.13. It is not known how much of the scatter shown in Figures 1-9 and 1-10 are based on the fact that BOD and ammonia testing was not from common samples and how much of the scatter is due to true variability in the influent wastewater characteristics.

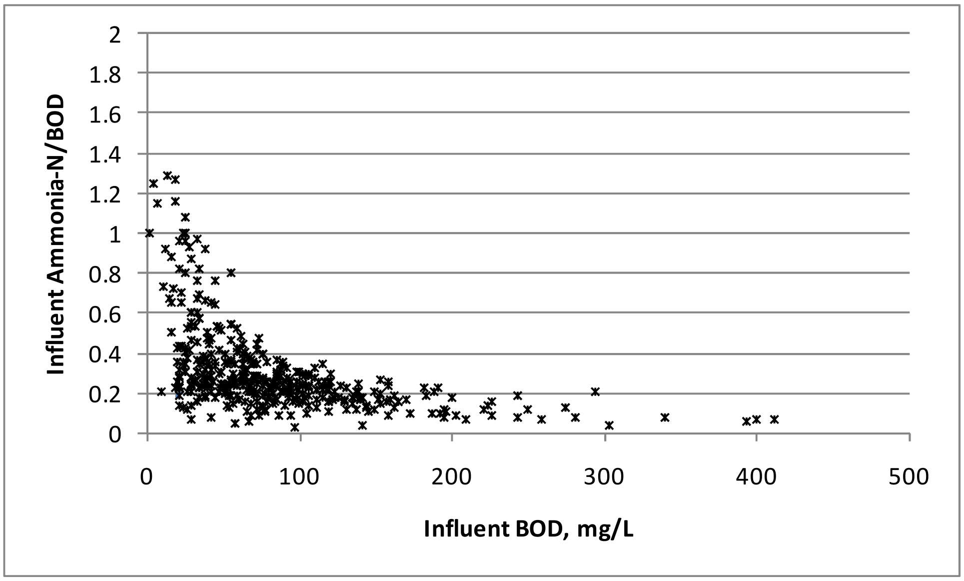


Figure 1-9
Influent Ammonia-N/BOD versus Influent BOD (All Data)

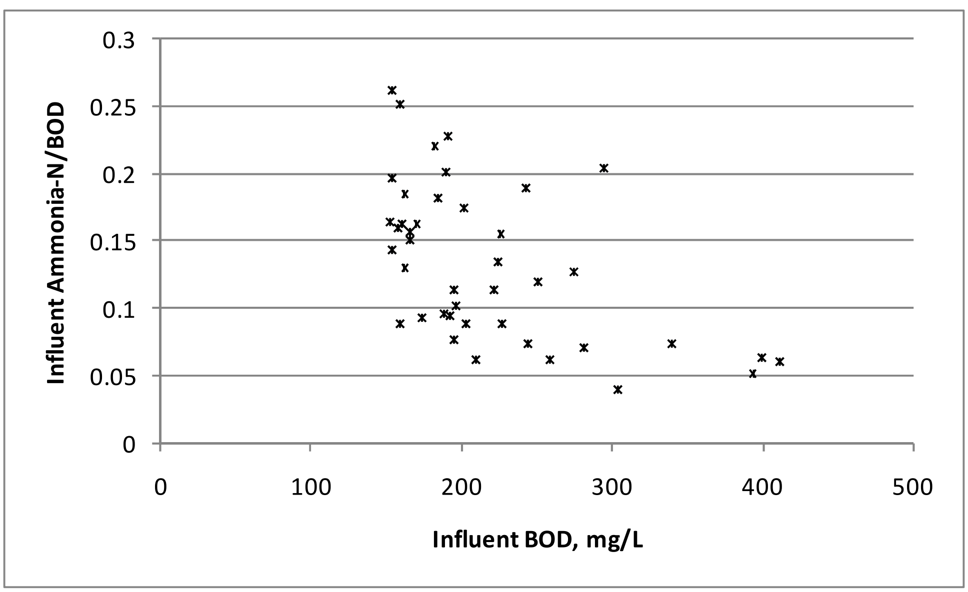


Figure 1-10
Influent Ammonia-N versus Influent BOD for BOD > 150 mg/L

The ratios of influent TKN to BOD taken from the special monitoring completed for and subsequent to the 2004 Joint Engineering Study are shown in Table 1-3 and discussed later in this document.

1.4.4 ALKALINITY

Influent alkalinity data from the Daily Wastewater Treatment Plant Lab Data Sheets were reviewed for the period of January through mid-December 2007. Like the ammonia-nitrogen data, these data are from equalization basin effluent grab samples. The values recorded twice weekly on Tuesday and Thursday mornings (back-dated to Monday and Wednesday due to compositing in the equalization basin) are shown in Figure 1-11. As shown, the alkalinity range has been from under 100 mg/L to almost 700 mg/L as calcium carbonate.

In Figure 1-12, influent alkalinity data are graphed versus influent BOD. Although there is a slight general trend to higher alkalinity values with higher BOD, it is apparent from these data that alkalinities as low as about 120 to 150 mg/L as calcium carbonate can occur even with BOD in the 200 to 400 mg/L range. This is a somewhat unexpected result as alkalinity, like BOD is generally added through water use. A stronger trend toward higher alkalinity with higher BOD would be expected. As stated for ammonia-nitrogen, however, the BOD data are based on composite samples, while the alkalinity data are based on grab samples. For a more certain comparison, both should be based on the same composite samples. The low alkalinity values sometimes recorded with relatively high BOD may be the result of the different samples used and may not represent the true relative values.

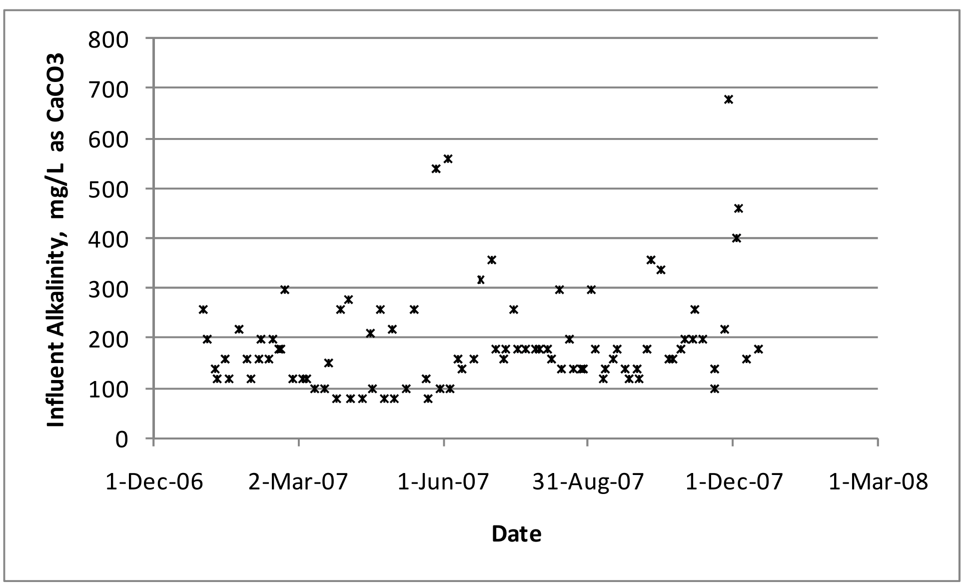


Figure 1-11
Influent Alkalinity

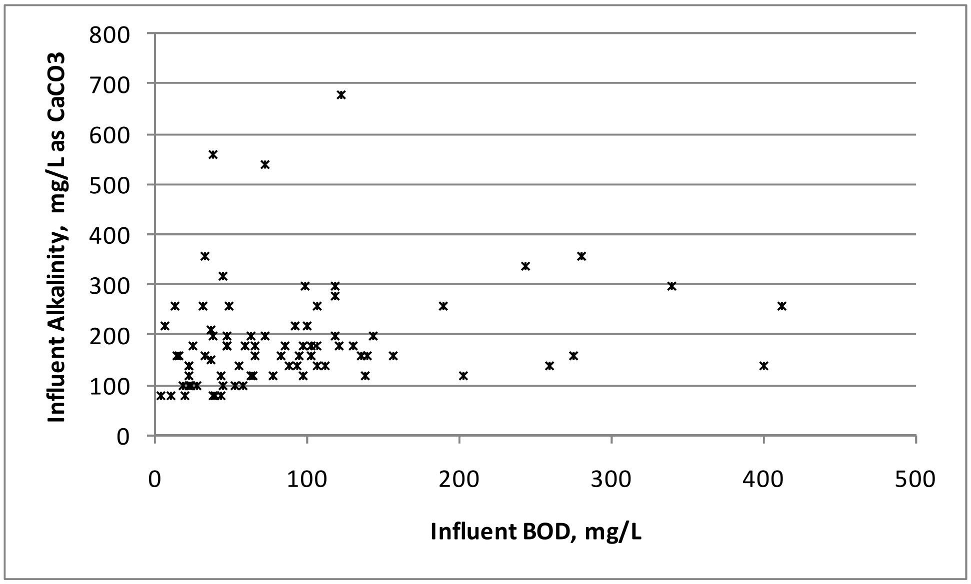


Figure 1-12
Influent Alkalinity versus BOD

Low influent alkalinity values would result in the need to add more chemicals like soda ash or lime to provide adequate alkalinity to support nitrification and give a stable pH. Unless additional data and subsequent evaluations can be used to support higher alkalinity with high BOD, the design of chemical addition facilities should be based on the conservative lower alkalinity values.

1.4.5 SPECIAL INFLUENT MONITORING COMPLETED FOR AND SUBSEQUENT TO THE JOINT ENGINEERING STUDY

As previously mentioned, special influent monitoring was completed for and subsequent to the DSPUD/SLCWD Joint Engineering Study on Wastewater Flows and Loads, dated June 10, 2004, by ECO:LOGIC Engineering in cooperation with Dewante and Stowell. The special monitoring was conducted on weekends between August 2003 and March 2005, and on weekdays on August 8, 2003 and between January 2005 and April 2005. The DSPUD influent wastewater results from these efforts are shown in Table 1-3. These data are separate from and are not included in the database of self monitoring report results that were used to generate the BOD and TSS charts previously presented in this technical memorandum.

As shown in Table 1-3, weekend daily BOD loads during the special monitoring effort were frequently over 500 lb/d, seven daily values exceeded 600 lb/d and the highest value was 855 lb/d (on Sunday, February 15, 2004). By comparing these values with the peak one-week average values from mid-week data shown in Figure 1-6 (about 400 to 600 lb/d), it is clear that these weekend loads represent critical high load events for plant operation and design.

Table 1-3
Special Influent Monitoring Data Completed for and Subsequent to the Joint Engineering Study

Effective Date Corresponding to Data in Plant Log (a)	Flow, Mgal/d	Alk., mg/L	TSS, mg/L	BOD, mg/L	COD, mg/L	TKN, mg/L	NH4N, mg/L	BOD Load, lb/d	Alk/ BOD	TSS/ BOD (b)	COD/ BOD (b)	TKN/ BOD (b)
Weekend Data from Sierra Environmental Monitoring (Except January 15, 2005 from Camner)												
Saturday, August 02, 2003	0.199	150	170	120	360	35		199	1.25	1.42	3.00	0.29
Saturday, August 09, 2003	0.203	340	47	225	480	100		381	1.51	0.21	2.13	0.44
Sunday, August 31, 2003	0.195	170	110	120	300	42		195	1.42	0.92	2.50	0.35
Saturday, October 11, 2003	0.259	200	47	88	190	33		190	2.27	0.53	2.16	0.38
Saturday, November 29, 2003	0.274		320	270	460	47		617		1.19	1.70	0.17
Saturday, December 27, 2003	0.390		160		530	69		0				
Saturday, January 03, 2004	0.356		230	220		59		653		1.05		0.27
Sunday, January 18, 2004	0.361		300	230	580	74		692		1.30	2.52	0.32
Sunday, February 15, 2004	0.410		280	250	560	72		855		1.12	2.24	0.29
Saturday, November 20, 2004			170	220	330	32				0.77	1.50	0.15
Saturday, January 01, 2005	0.258		280	320	810	63		689		0.88	2.53	0.20
Saturday, January 15, 2005	0.323		324	184	786	77.9		496		1.76	4.27	0.42
Saturday, February 19, 2005	0.347		170	250	500		44	723		0.68	2.00	
Sunday, February 20, 2005	0.355		280	260	290	60		770		1.08	1.12	0.23
Saturday, March 26, 2005	0.305		120	200	320	37	39	509		0.60	1.60	0.19
	0.303	215	201	211	464	57		498	1.61	0.96	2.25	0.28
Weekday Data from Cramner Analytical Laboratory (Except August 8, 2003 from Sierra Environmental)												
Friday, August 08, 2003	0.172	440	120	160	270	150		230	2.75	0.75	1.69	0.94
Monday, December 27, 2004	0.345			226	522	64.1		650			2.31	0.28
Monday, January 17, 2005	0.254		338	295	805	67.4		625		1.15	2.73	0.23
Wednesday, January 19, 2005	0.171		190	137	310	13.7		195		1.39	2.26	0.10
Wednesday, January 26, 2005	0.181		85	58	206	23.8		88		1.47	3.55	0.41
Wednesday, February 02, 2005	0.217		164	227	509	33.1		411		0.72	2.24	0.15
Wednesday, February 09, 2005	0.204		464	144	776	37.2		245		3.22	5.39	0.26
Wednesday, February 16, 2005	0.222		142	93	152	28.2		172		1.53	1.63	0.30
Wednesday, February 23, 2005	0.309		219	182	208	48.9		469		1.20	1.14	0.27
Wednesday, March 02, 2005	0.209		127	119	291	33.1		207		1.07	2.45	0.28
Wednesday, March 09, 2005	0.290		103	96	464	25.4		232		1.07	4.83	0.26
Wednesday, March 16, 2005	0.278		119	105	436	24.6		243		1.13	4.15	0.23
Wednesday, March 23, 2005	0.221		160	192	339	34		354		0.83	1.77	0.18
Wednesday, March 30, 2005	0.260		50	30	125	29.1		65		1.67	4.17	0.97
Wednesday, April 13, 2005	0.256		84	74	11	24.1		158		1.14	0.15	0.33
	0.261		187	170	409	47		382		1.21	2.58	0.33

(a) Constituent data based on composite samples collected from the morning before to the morning of the date given.
(b) Data highlighted when outside of indicated ranges: 0.75 < TSS/BOD < 1.25; 1.5 < COD/BOD < 2.75; 0.1 < TKN/BOD < 0.4

Of key importance in Table 1-3 are the relative values of TSS, COD, and TKN compared to BOD. The weekend and weekday constituent ratios are similar, but both are highly variable, with problematic values that seem to be well outside of normal ranges. In many, if not most cases, it is believed that these problematic values are probably due to non-representative sampling or laboratory error. The problematic values are highlighted in Table 1-3. Since it would normally be expected that TSS/BOD would be near 1.0, values for this parameter outside of the range of 0.75 to 1.25 were highlighted. The COD/BOD ratio would be expected to be around 2, so values outside of the range of 1.5 to 2.75 were highlighted. For TKN, a ratio to BOD of around 0.2 would be expected, so values outside the range of 0.1 to 0.4 were highlighted.

Despite the problematic values shown in Table 1-3, the average TSS/BOD and COD/BOD ratios are not too far from expected results. However, that is not the case for TKN/BOD. In this case, the average ratios are about 50% above expected values. The ammonia-nitrogen data previously discussed also indicate the possibility of TKN/BOD ratios higher than typical domestic wastewater. The apparent high ratio of TKN to BOD has major implications for the design of a biological nitrification and denitrification process. For denitrification, unusually large anoxic basins and/or addition of supplemental BOD by artificial means (such as methanol addition) could be required as the result of high TKN/BOD ratios.

1.4.6 SPECIAL INFLUENT MONITORING IN JANUARY AND FEBRUARY 2008

To support this evaluation, special influent monitoring was completed in January and February 2008. Composite samples of equalization tank effluent (plant influent) were analyzed by plant staff for COD and ammonia-nitrogen for the Friday, Saturday, and Sunday periods from January 25 through January 27 and February 1 through February 3. A more extensive monitoring effort was completed surrounding the Presidents' Day Holiday, and included sampling for the following days:

- Friday, February 15, 2008 through Tuesday, February 19, 2008
  (Presidents' Day was Monday, February 18, 2008)
- Friday, February 22, 2008 through Sunday, February 24, 2008

For the days listed above, COD and alkalinity were tested on-site by plant staff; COD, BOD, TSS, VSS, TKN, and ammonia were tested by Sierra Environmental Monitoring; and BOD, TSS, VSS, TKN, and ammonia were tested by Cranmer Analytical Laboratory. As indicated, duplicate tests for all parameters except alkalinity were completed by different laboratories.

Results for all of the special monitoring and analyses based on those results are shown in Table 1-4. In the table, pink highlighting is used to indicate data values that are considered questionable because of discrepancies between duplicate samples, abnormal ratios with other constituent values, and/or based on trends. Discrepancies between duplicate samples are illustrated by comparing results for COD, BOD, ammonia-N (NH4-N) and TKN as derived from two different laboratories in Figures 1-13 through 1-16. As indicated, substantial discrepancies were found, particularly for the COD and TKN tests. Because of the questionable values, it was necessary to use engineering judgment to assess the most likely values for the various data. These are listed in the lower section of Table 1-4. In that section, green highlighting is used to indicate a judgment value that is substantially different from at least one of the corresponding laboratory results. As indicated, the estimated average BOD load for Presidents' Week 2008 is 628 lb/d, which is somewhat less than the estimated peak week load of 780 lb/d previously determined. This is believed to be reasonable, since loadings during the Christmas / New Year's period in a given year could certainly exceed the Presidents' Week 2008 load.

Table 1-4
Special Influent Monitoring January and February 2008 Results and Analysis

		Flow,	Constituent Concentrations, mg/L							Constituent Loads, lb/d							Constituent Ratios					
Date	Day	Mgal/d	COD	BOD	TSS	VSS	TKN	NH4-N	Alkalinity	COD	BOD	TSS	VSS	TKN	NH4-N	Alkalinity	COD/BOD	TSS/BOD	TKN/BOD	NH4-N/BOD	VSS/TSS	NH4-N/TKN
DSPUD On-Site Testing Data																						
1/25/2008	Fri	0.284	367					18		869					43							
1/26/2008	Sat	0.224	527					56		985					105							
1/27/2008	Sun	0.177	447					46		660					68							
2/1/2008	Fri	0.205	508					51		869					87							
2/2/2008	Sat	0.269	287					55		644					123							
2/3/2008	Sun	0.211	603					60		1061					106							
2/15/2008	Fri	0.247	498						220	1026						453						
2/16/2008	Sat	0.369	680						220	2093						677						
2/17/2008	Sun	0.396	746						300	2464						991						
2/18/2008	Mon	0.337	363						220	1020						618						
2/19/2008	Tue	0.271	559						220	1263						497						
2/22/2008	Fri	0.258	1022						200	2199						430						
2/23/2008	Sat	0.295	381						200	937						492						
2/24/2008	Sun	0.220	280						200	514						367						
Cranmer Analytical Laboratory Data																						
2/15/2008	Fri	0.247		192	196	ND	51.9	44.3			396	404		107	91			1.02	0.27	0.23		0.85
2/16/2008	Sat	0.369		236	213	ND	74.1	31.3			726	655		228	96			0.90	0.31	0.13		0.42
2/17/2008	Sun	0.396		279	252	ND	86.2	51.4			921	832		285	170			0.90	0.31	0.18		0.60
2/18/2008	Mon	0.337		185	131	ND	75.6	46.5			520	368		212	131			0.71	0.41	0.25		0.62
2/19/2008	Tue	0.271		193	179	ND	62.9	37.8			436	405		142	85			0.93	0.33	0.20		0.60
2/22/2008	Fri	0.258		418	468	ND	69.5	35.9			899	1007		150	77			1.12	0.17	0.09		0.52
2/23/2008	Sat	0.295		155	176	ND	61.1	43.5			381	433		150	107			1.14	0.39	0.28		0.71
2/24/2008	Sun	0.220		153	144	ND	42.1	34.2			281	264		77	63			0.94	0.28	0.22		0.81
Sierra Environmental Monitoring Data																						
2/15/2008	Fri	0.247	230	220	440	380	33	27		474	453	906	783	68	56		1.05	2.00	0.15	0.12	0.86	0.82
2/16/2008	Sat	0.369	310	180	210	380	47	35		954	554	646	1169	145	108		1.72	1.17	0.26	0.19	1.81	0.74
2/17/2008	Sun	0.396	530	250	220	370	54	44		1750	826	727	1222	178	145		2.12	0.88	0.22	0.18	1.68	0.81
2/18/2008	Mon	0.337	350	180	210	190	62	58		984	506	590	534	174	163		1.94	1.17	0.34	0.32	0.90	0.94
2/19/2008	Tue	0.271	300	250	160	140	29	29		678	565	362	316	66	66		1.20	0.64	0.12	0.12	0.88	1.00
2/22/2008	Fri	0.258	870	340	500	470	48	40		1872	732	1076	1011	103	86		2.56	1.47	0.14	0.12	0.94	0.83
2/23/2008	Sat	0.295	280	160	180	160	52	44		689	394	443	394	128	108		1.75	1.13	0.33	0.28	0.89	0.85
2/24/2008	Sun	0.220	270	150	170	160	41	29		495	275	312	294	75	53		1.80	1.13	0.27	0.19	0.94	0.71
Engineering Judgement Based on Above																						
1/25/2008	Fri	0.284	367					40		869					95							
1/26/2008	Sat	0.224	527					56		985					105							
1/27/2008	Sun	0.177	447					46		660					68							
2/1/2008	Fri	0.205	508					51		869					87							
2/2/2008	Sat	0.269	550					55		1234					123							
2/3/2008	Sun	0.211	603					60		1061					106							
2/15/2008	Fri	0.247	498	206	200	180	41	36	220	1026	424	412	371	84	73	453	2.42	0.97	0.20	0.17	0.90	0.87
2/16/2008	Sat	0.369	500	208	212	190	47	33	220	1539	640	651	586	145	102	677	2.40	1.02	0.23	0.16	0.90	0.71
2/17/2008	Sun	0.396	530	265	236	212	54	48	300	1750	874	779	701	178	158	991	2.00	0.89	0.20	0.18	0.90	0.88
2/18/2008	Mon	0.337	350	183	171	153	62	52	220	984	513	479	431	174	147	618	1.92	0.93	0.34	0.29	0.90	0.84
2/19/2008	Tue	0.271	500	222	170	153	44	33	220	1130	501	383	345	99	75	497	2.26	0.77	0.20	0.15	0.90	0.76
2/22/2008	Fri	0.258	870	379	484	436	65	40	200	1872	816	1041	937	140	86	430	2.30	1.28	0.17	0.11	0.90	0.62
2/23/2008	Sat	0.295	330	158	178	160	52	44	200	812	387	438	394	128	108	492	2.10	1.13	0.33	0.28	0.90	0.84
2/24/2008	Sun	0.220	275	152	157	141	41	32	200	505	278	288	259	75	58	367	1.82	1.04	0.27	0.21	0.90	0.77
Pres. Week Avg. (b)		0.310	536	243	230	207	51	39	229	1387	628	596	536	133	101	592	2.21	0.95	0.21	0.16	0.90	0.76

(a) Key to Color Coding
- [red] Data that appear to be questionable based on other data on same date or based on trends.
- [green] Engineering judgement value that is substantially different than at least one of the corresponding actual data values (normally occurs where actual data values are substantially different for duplicate samples).

(b) Presidents week average determined based on data from Saturday February 16 through Friday February 22, and assuming values for Wednesday and Thursday would be the same as values for Tuesday.

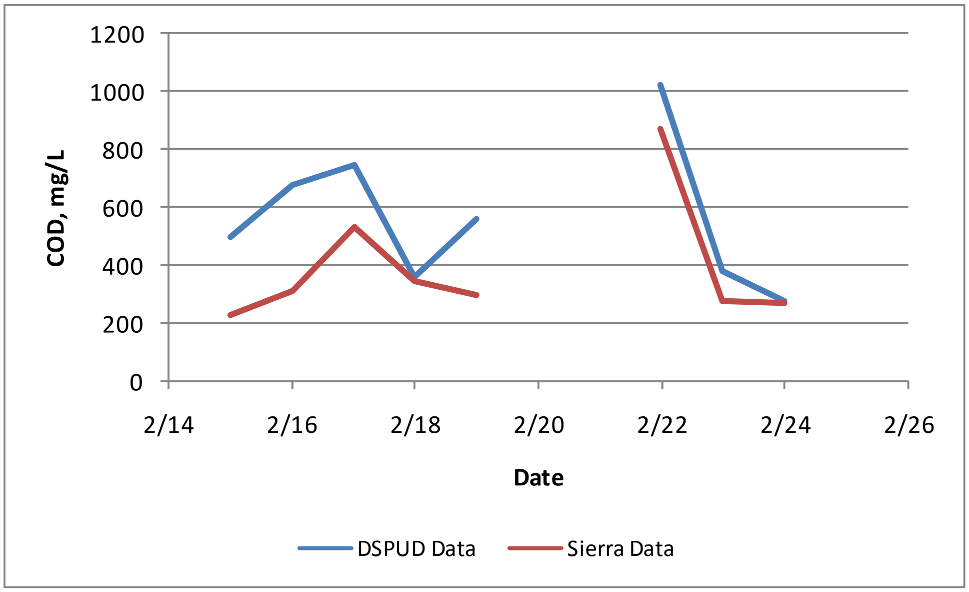


Figure 1-13
Presidents' Week 2008 COD Data Comparison

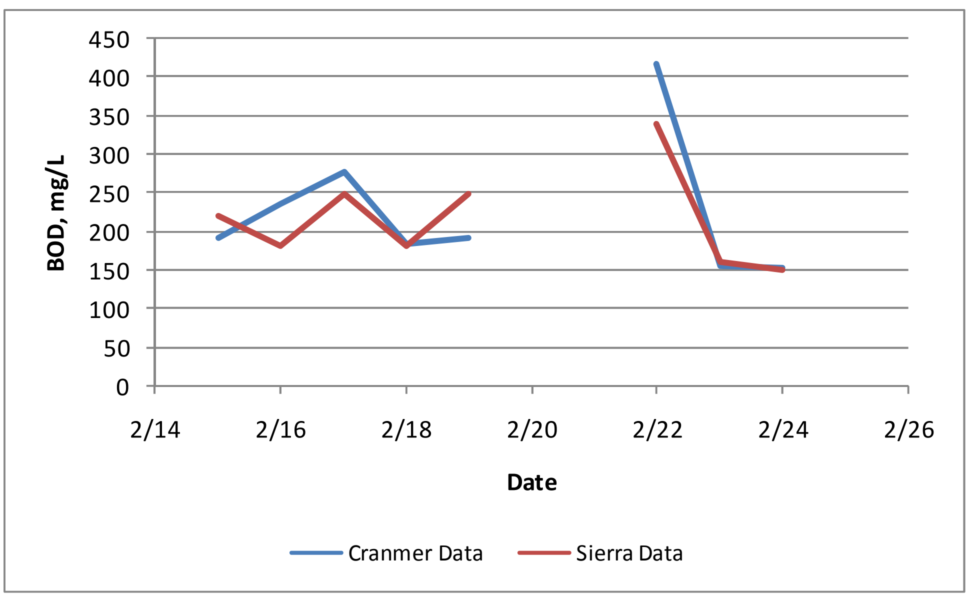


Figure 1-14
Presidents' Week 2008 BOD Data Comparison

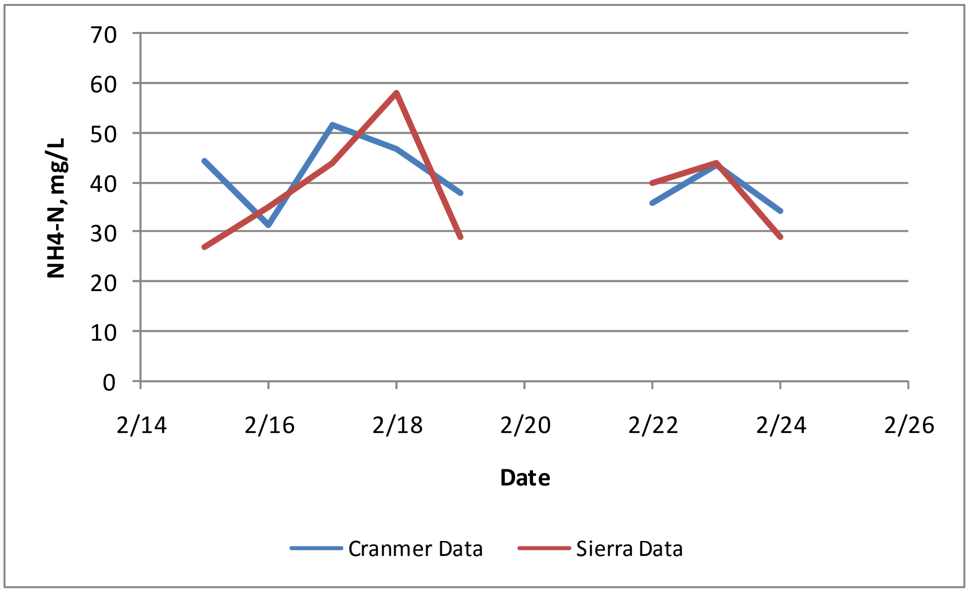


Figure 1-15
Presidents' Week 2008 Ammonia-N Data Comparison

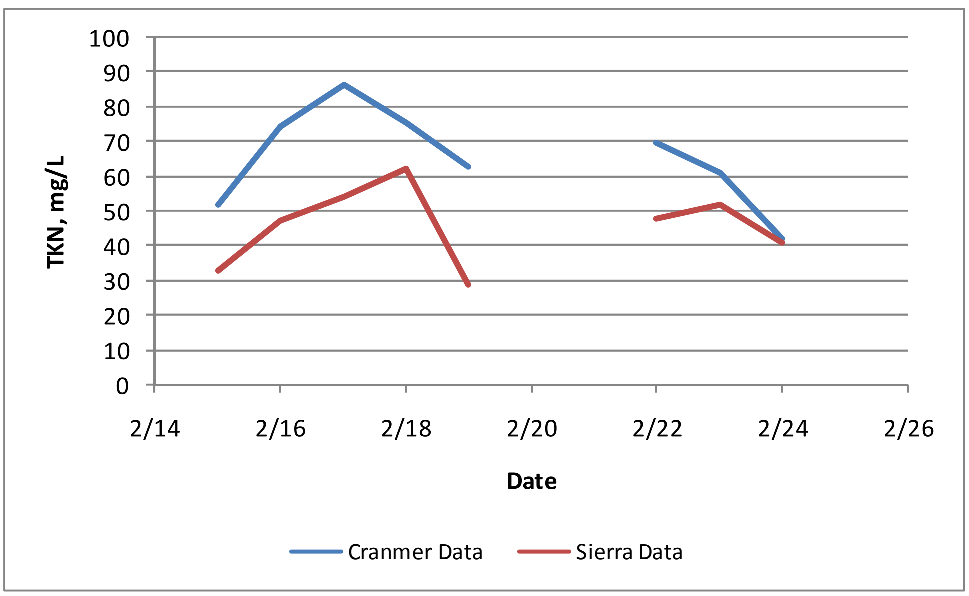


Figure 1-16
Presidents' Week 2008 TKN Data Comparison

1.4.7 BIOLOGICAL PROCESS DESIGN LOADS FOR EXISTING CONDITIONS

Based on the data and discussions presented above, the following are believed to be reasonable design influent BOD loads for existing conditions (without allowances for future growth):

Annual Average Load (AAL)	215 lb/d
Average Day Maximum Monthly Load (ADMML)	520 lb/d
Average Day Maximum Weekly Load (ADMWL)	780 lb/d
Maximum Daily Load (MDL)	900 lb/d

In developing the above list, the AAL was determined as the average mid-week load from October 2004 through September 2007 (see Figure 1-6), multiplied by 1.3. The 1.3 is an estimated factor to convert mid-week average data to weekly average data, including the weekends. The ADMML was based on a reasonably conservative mid-week value of 400 lb/d from Figure 1-6, multiplied by the same 1.3 adjustment factor. The ADMWL was based on a reasonably conservative mid-week value of 600 lb/d from Figure 1-6, multiplied by 1.3. Finally, the peak day value was based on two midweek values in the 800 to 900 lb/d range indicated in Figure 1-6 and actual values of 855 and 874 lb/d for Sunday February 15, 2004 and Sunday February 17, 2008, respectively. It is believed that a true peak day BOD load in the Christmas/ New Year's period could be higher than these values.

To determine design TSS and TKN loads for existing conditions, the BOD loads can be multiplied by 1.0 and 0.3, respectively. The 0.3 factor for TKN is unusually high for domestic wastewater, but is supported by data from the previously mentioned Joint Engineers Study, as well as the special monitoring in January and February 2008. With peak load conditions, it is estimated that the influent alkalinity will be around 150 mg/L as calcium carbonate, or higher.

Because of the importance of the TKN/BOD ratio in nitrification/denitrification design, TKN should be routinely analyzed for using the same influent composite samples as used for BOD and TSS. Additionally, weekend monitoring should be accomplished routinely, particularly in peak occupancy periods. The weekend monitoring is needed to confirm whole-week average constituent loads, as opposed to relying solely on mid-week data.

1.5 ESTIMATE OF FUTURE USERS FLOWS AND LOADS

This section and the following section were completed in the fall of 2009, after DSPUD authorized development of a Wastewater Treatment and Disposal Facilities Plan and both DSPUD and SLCWD determined growth allowances to be incorporated in the proposed project. The growth allowances adopted by the two Districts are as follows:

DSPUD: 332 EDU
SLCWD: 80 EDU

Incremental flows and loads due to the new EDUs can be projected based on data developed in the Joint Engineering Study on Wastewater Flows and Loads, prepared by ECO:LOGIC Engineering and Dewante and Stowell, dated June 10, 2004. In that study, peak 3-day flow and load allowances for new EDUs were determined from historical records and special monitoring within the two Districts. These peak 3-day flow and load factors per EDU and the total projected peak 3-day flows and loads based on the numbers of new EDUs indicated above are shown in Table 1-5.For this analysis, it is useful to convert the peak 3-day flows and loads into an average day maximum weekly flows (ADMWFs) and average day maximum weekly loads (ADMWLs). This can be done based flow and load factors developed previously in this memorandum. Specifically, for flow, it has been determined that in the winter ski season, flows on the weekends are typically 1.5 times the flows on weekdays. If it is presumed that a peak week includes three days with weekend flows and four days with weekday flows, the ADMWF can be calculated to be approximately 81% of the peak 3-day flow. Similarly, for loads, it has been determined that the average daily load in a typical week is 1.3 times the average daily load on typical weekdays. Assuming a typical week to consist of five weekdays and two weekend days, it can be determined that a typical weekday load is about 49% of a typical weekend day load. For a peak week, with four weekdays and three weekend days, the average daily load for the entire week would be 71% of the average daily load during a peak three-day weekend. The 81% factor for flows and the 71% factor for loads are used to determine ADMWFs and ADMWLs from the peak 3-day flows and loads in Table 1-5.

Table 1-5
Flows and Loads for Future Growth

Parameter	DSPUD	SLCWD	Total
Peak 3-Day Weekend Flows and Loads Per EDU			
Flow, gpd	440	250	
BOD_5 Load, lb/d	0.88	0.83	
TSS Load, lb/d	1.28	0.52	
TKN Load, lb/d	0.29	0.14	
Total Projected Peak 3-Day Weekend Flows and Loads For New Growth			
Number of New EDU	332	80	412
Flow, gpd	146,080	20,000	166,080
BOD_5 Load, lb/d	292	66	359
TSS Load, lb/d	425	42	467
TKN Load, lb/d	96	11	107
Total Projected Average Day Maximum Weekly Flows and Loads For New Growth (a,b)			
Flow, gpd	118,325	16,200	134,525
BOD_5 Load, lb/d	207	47	255
TSS Load, lb/d	302	30	331
TKN Load, lb/d	68	8	76

(a) Peak week flows estimated at 81% of peak 3-day flow.
(b) Peak week loads estimated at 71% of peak 3-day loads.

1.6 SUMMARY OF EXISTING AND FUTURE FLOWS AND LOADS

Based on the information developed in this Technical Memorandum, design influent flows and loads for the DSPUD WWTP are summarized in Table 1-6. Although the existing flow and load criteria are based on data through early 2008, District staff has reviewed the flows and loads occurring later in 2008 and 2009, up to the date of preparation of this document, and have indicated that the more recent data would not change the assessment previously developed. Future flows and loads are determined based on these existing flows and loads and the incremental flows and loads for new development shown in Table 1-5. Although the ratio of TSS/BOD_5 for new growth shown in Table 1-5 is higher than 1.0, this ratio and the corresponding ratio for existing conditions is highly variable. Therefore, within the accuracy of this analysis, it is considered appropriate to use the same 1.0 ratio used to represent existing conditions to also represent future conditions.

In developing Table 1-6, it was assumed that that ADMWF developed for future growth corresponds to the "high" projection developed for existing conditions. With one exception, it was presumed that ratios between average daily maximum weekly data and other data would be the same for the incremental flows and loads as previously developed for existing flows and loads. The one exception is that the peak hour flow was presumed to not increase above the existing value, because the existing value is believed to be adequately conservative and because it is expected that both districts will be able to mitigate infiltration and inflow at least to the extent of avoiding any further increases in this peak hour flow.

Table 1-6
Design Flows and Loads Summary

Parameter	Existing Conditions	Allowance for Growth	Future Condition
Design Flows, Mgal/d			
Average Annual Flow (AAF)	0.23	*0.05*	*0.28*
Average Day Maximum Monthly Flow (ADMMF)			
Typical	0.35	*0.07*	*0.42*
High	0.43	*0.09*	*0.52*
Average Day Maximum Weekly Flow (ADMWF)			
Typical	0.43	*0.09*	*0.52*
High	0.61	*0.13*	*0.74*
Peak Day Flow (PDF)	0.97	*0.21*	*1.18*
Peak Hour Flow (PHF)	*1.7*	*0.00*	*1.70*
BOD Load, lb/d			
Average Annual Load (AAL)	215	*70*	*285*
Average Day Maximum Monthly Load (ADMML)	520	*170*	*690*
Average Day Maximum Weekly Load (ADMWL)	780	*255*	*1035*
Peak Day Load (PDL)	900	*294*	*1194*
BOD Concentration, mg/L			
AAL combined with AAF	112	172	123
ADMML combined with Typical ADMMF	178	273	195
ADMML combined with High ADMMF	145	222	159
ADMWL combined with Typical ADMWF	218	334	238
ADMWL combined with High ADMWF	153	235	168
PDL combined with ADMWF	251	385	275
PDL combined with PDF	111		122
TSS Loads and Concentrations	1.0 x BOD	1.0 x BOD	1.0 x BOD
TKN Loads and Concentrations	0.3 x BOD	0.3 x BOD	0.3 x BOD

Appendix B

TM No. 2 - Equalization Storage

Donner Summit Public Utility District WWTP
Technical Memorandum No. 2

Equalization Storage

Prepared By: Jeffrey R. Hauser, P.E.

Date: April 23, 2009

2.1 PURPOSE AND OVERVIEW

The purpose of this memorandum is to determine the volume of equalization storage required to limit peak flows through the Donner Summit PUD Wastewater Treatment Plant (DSPUD WWTP). To accomplish this objective, historical influent flows to the DSPUD WWTP are analyzed to determine the volumes of equalization storage that would have been required for peak flow trimming to various limiting flows in previous years. The results of the historical analysis are then used, together with other considerations, to project future equalization storage requirements.

2.2 ANALYSIS OF HISTORICAL FLOWS AND EQUALIZATION REQUIREMENTS

Daily influent flow data were obtained for the period from January 1, 2002 through April 16, 2009. For various selected limiting flows, the volumes of wastewater that would have been accumulated in storage on a day-by-day basis over the entire period were determined. For each day in which the influent flow was greater than the limiting flow, the increment of flow over the limit was presumed to be stored and was added to the volume, if any, remaining in storage from previous days. When the influent flow decreased below the limiting flow, the difference between the limiting flow and the influent flow was presumed to be removed from storage and was subtracted from the volume, if any, remaining in storage from previous days, until the storage volume was depleted.

Limiting flows of 0.30 through 0.55 Mgal/d were investigated. Graphs showing accumulated storage volumes over the entire period of analysis for limiting flows in this range and in 0.05 Mgal/d increments are shown in the appendix (Figures A1 through A6). A summary graph showing maximum storage volumes for the first, second and third largest storage events for the period of analysis is presented in Figure 2-1 (0.025 Mgal/d increments). For the purposes of this analysis, a storage event begins on the first day that wastewater is introduced into a previously empty storage tank and continues until the storage tank is completely emptied again. Because storage volumes for the limiting flows under 0.35 Mgal/d were considered to be too large to be practical, they are not shown in Figure 2-1, but a graph for the limiting flow of 0.30 Mgal/d can be seen in the appendix.

By reviewing Figure 2-1 and the graphs in the appendix, several observations can be made as follows:

- Storage events would generally occur during the winter ski season and during the spring snowmelt period.
- With lower limiting flows (0.30 to 0.35 Mgal/d), individual storage events could last for many months.
- The most severe storage events for all limiting flows would have been initiated with the peak flows occurring in late December 2005 and early January 2006.
- With higher limiting flows, the December 2005/January 2006 event would have required far more storage than any other event.

Because the flows of late December 2005 and early January 2006 were substantially larger than any other recorded flows and would have required much larger storage volumes than other events, the analysis described above was repeated with those peak flows artificially reduced to a maximum of 0.4 Mgal/d, which is not considered to be a particularly high flow for this time of the year. The results of this hypothetical analysis are shown in appendix Figures A7 through A12 and are summarized in Figure 2-2. As can be seen in Figure 2-2, when the December 2005 / January 2006 event is reduced as described, the largest storage event becomes much more comparable to the second and third largest events for all limiting flows.

2.3 DISCUSSION OF THE DECEMBER 2005/JANUARY 2006 EVENT

It is important to understand the conditions of the December 2005/January 2006 peak flow event in order to assess whether that event should be considered an anomaly that is not likely to be repeated, or whether similar conditions could reasonably be expected in the future. This topic was addressed in Technical Memorandum No. 1 on Design Flows and Loads, which was completed in draft form in May 2008. The following is an excerpt from that memorandum:

> *The highest weekly average flow of 0.61 Mgal/d ending on January 2, 2006 was apparently the result of heavy precipitation in the previous days, almost all of which fell as rain, not snow. There was a total of 14.4 inches of precipitation in a period of 16 days through January 1, 2006, including 4.25 inches recorded on December 30, 2005, and 1.64 inches recorded on January 1, 2006 (NCDC dates shifted back one day as previously noted). The maximum daily wastewater treatment plant influent flow measured during 2002 through 2007 was 0.967 Mgal/d, recorded on December 30, 2005, on the same day as the peak day precipitation of 4.25 inches. Based on depth-duration-frequency data available from the California Data Exchange Center (CDEC), the daily rainfall amount of 4.25 inches was an 18-year return frequency event. The 16-day rainfall total of 14.4 inches was about an 11-year return frequency event.*

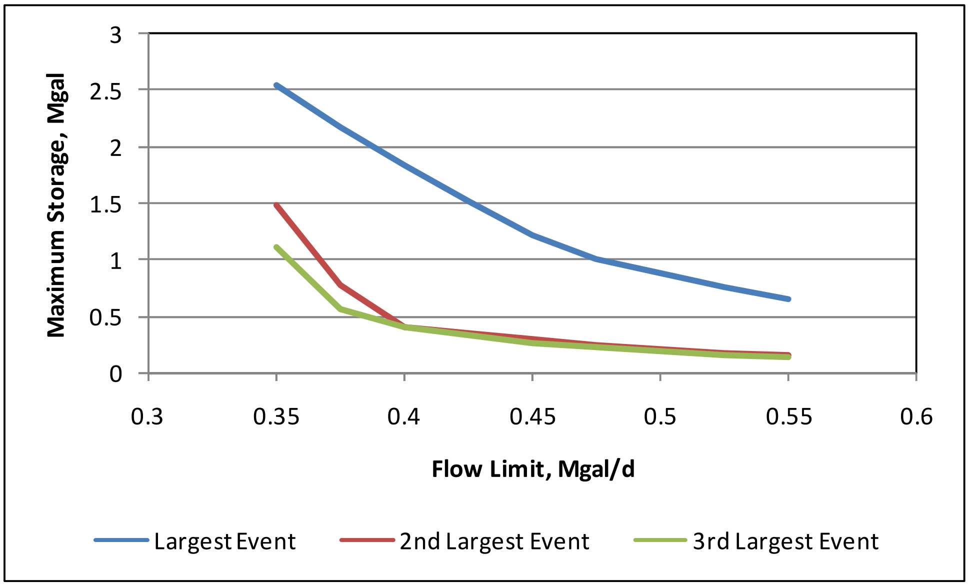


Figure 2-1
Summary of Maximum Storage Requirements for Various Limiting Flows

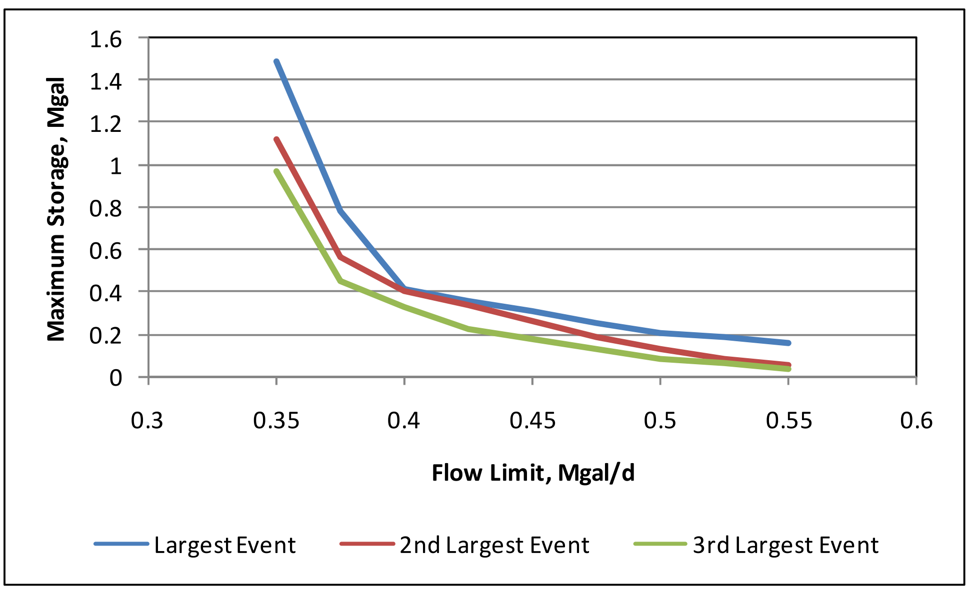


Figure 2-2
Summary of Maximum Storage Requirements for Various Limiting Flows with Flows from December 21, 2005 through January 2, 2006 Artificially Reduced to 0.4 Mgal/d

It is clear that a large portion of the flows occurring during the peak flow event of December 2005/January 2006 were due to infiltration and inflow resulting from a warm winter storm. However, considering that the rainfall amounts were only in the range of 11 to 18 years in return frequency, it is not considered unreasonable to design for such an event.

Another consideration regarding the December 2005/January 2006 event is whether the infiltration and inflow that occurred could be prevented from occurring again in the future through collection system improvements, some of which may have already been completed by DSPUD and SLCWD. While it is likely that system improvements have reduced or will reduce flows from some infiltration and inflow sources, it is impossible to know how the collection system will respond to conditions similar to those in 2005 and 2006 without a repeat of those conditions. Also, it is unfortunately true that new infiltration and inflow sources will develop even as others are eliminated and that infiltration and inflow correction is a never-ending endeavor as the collection systems continue to age.

Considering all factors, it would not be prudent to disregard what happened in December 2005 and January 2006 when projecting future equalization storage requirements. The degree to which that historical event should impact the determination of future requirements is a matter of judgment that should be considered together with all other issues that relate to desired safety factors and levels of risk.

2.4 EQUALIZATION OPERATION AND VOLUME SAFETY FACTOR

The evaluation of historical equalization volume requirements presented above is an idealized analysis. Using the historical data, it is possible to accurately determine the storage volume that would have been required to equalize flows to specific limits in specific events. In actual operations, however, there is a great deal of uncertainty involved. As various peak flow events evolve, the plant operator will not know how severe the events will turn out to be. Rather, based on the operator’s assessment of historical flows, weather conditions, occupancy in the service area, available equalization volume and other relevant factors, the operator will determine the equalized flow that should be passed through the plant at a specific time. Ideally, a constant flow would be maintained over an extended period of time, without exhausting the available volume of equalization storage. If the operator believes he can limit flow to a certain amount and then finds that influent flows are higher than anticipated and the storage volume is being exhausted faster than desired, he will have to increase the flow through the plant. In this case, the maximum volume accumulated in storage will be far greater than what it would have been if the higher plant flow were established from the beginning of the event. As the available equalization storage volume is diminished below comfortable limits, there will be great concern about overflowing the storage basin and the operator will maximize the flow through the plant.

One possible operational scenario to minimize the potential for overflowing the equalization basin would be to keep the tank essentially empty at all times by matching the plant flow to the influent flow, up to the maximum capacity of the plant. However, this scheme virtually eliminates the benefits flow equalization under most conditions. Even if this type of peak flow trimming operation were established, there would still be a risk that actual flow conditions could be more severe than those upon which the equalization tank was designed.

Based on the above considerations, it is clear that a substantial margin of safety should be incorporated in sizing an equalization basin and/or other safety features should be incorporated into the plant design, such as:

- Provisions for overflow of the equalization storage basin into an emergency storage basin.
- Provisions for abnormal emergency peak flows through the plant. Such emergency peak flows might be allowed to temporarily degrade effluent quality, but this would be preferable to overflowing raw sewage on the ground.

For obvious reasons, it would be desirable to avoid using emergency provisions such as those listed above; therefore, ample sizing of the equalization storage basin is prudent.

2.5 ACTIVE VOLUME VERSUS TOTAL VOLUME

The discussion of equalization volumes presented above is based on active volume. Active volume is the volume between the minimum and maximum desired levels in the tank.

The contents of the equalization tank will have to be aerated and mixed. For aeration and/or mixing to occur, there must be a minimum depth of wastewater in the tank, which will depend on the type of aeration and mixing equipment used. Usually, at least a couple of feet of depth is needed. The volume represented by this minimum depth would be added to the active storage volume when determining the total tank volume.

Similarly, near the top of the tank, there will be a freeboard requirement. Under normal operations, it would be desirable to keep the maximum water level in the tank at least six inches below any overflow outlet. Furthermore, the top of the tank should be at least a foot or two above the maximum emergency water level in the tank.

2.6 RECOMMENDED EQUALIZATION VOLUME SIZING

Based on the considerations presented above, it is recommended that any equalization basin should be sized as follows:

1. The active volume should be based on the “Largest Event” curve shown in Figure 2-1. Since this volume would be based on the severe event of December 2005/January 2006, it would be judged to have an inherent safety factor, such that no further volume safety factor would be needed.
2. To the active volume indicated above, additional volume would be added to allow for non-active volume below the minimum tank operating depth and to allow for appropriate freeboard, all as to be determined during design of the basin.

It is further recommended that the plant design incorporate features, such as previously mentioned or other appropriate measures, to provide for emergency operations in case of exhaustion of the available equalization storage volume.

The equalization volume determined in accordance with these procedures would be appropriate for the existing level of development within the service area. If significant additional development is to be provided for, appropriate adjustments in the limiting flow and/or storage volume would have to be made.

Determination of the most cost-effective blend of equalization volume and plant capacity will depend on the specific processes to be employed in the plant as well as on available site space and other design considerations.

Appendices to TM No. 2

Figures

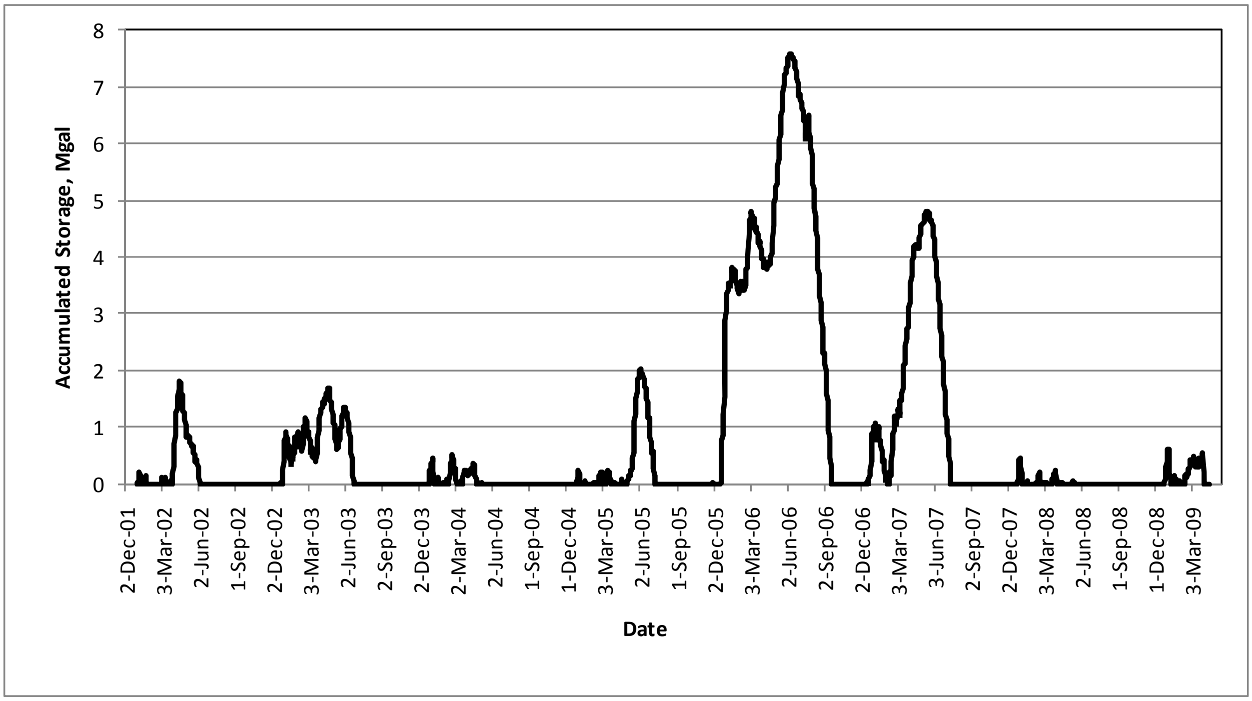
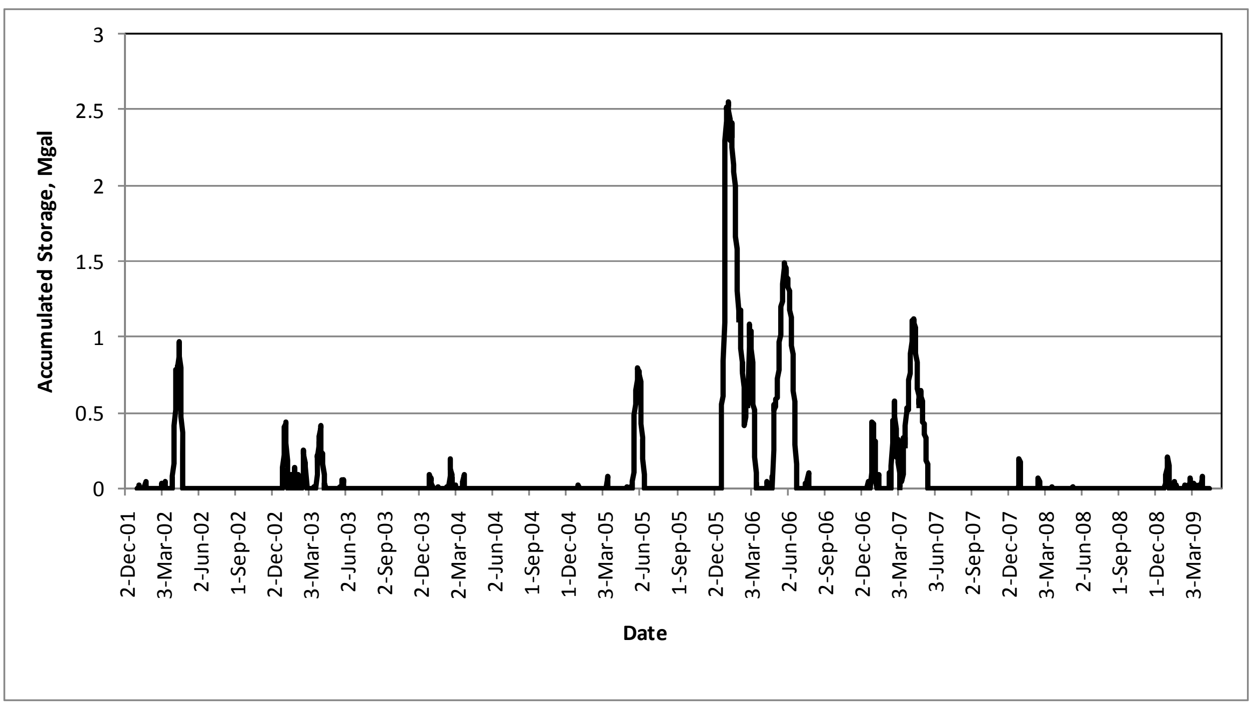


Figure A1
Storage Volumes with Limiting Flow = 0.30 Mgal/d



FigureA2
Storage Volumes with Limiting Flow = 0.35 Mgal/d

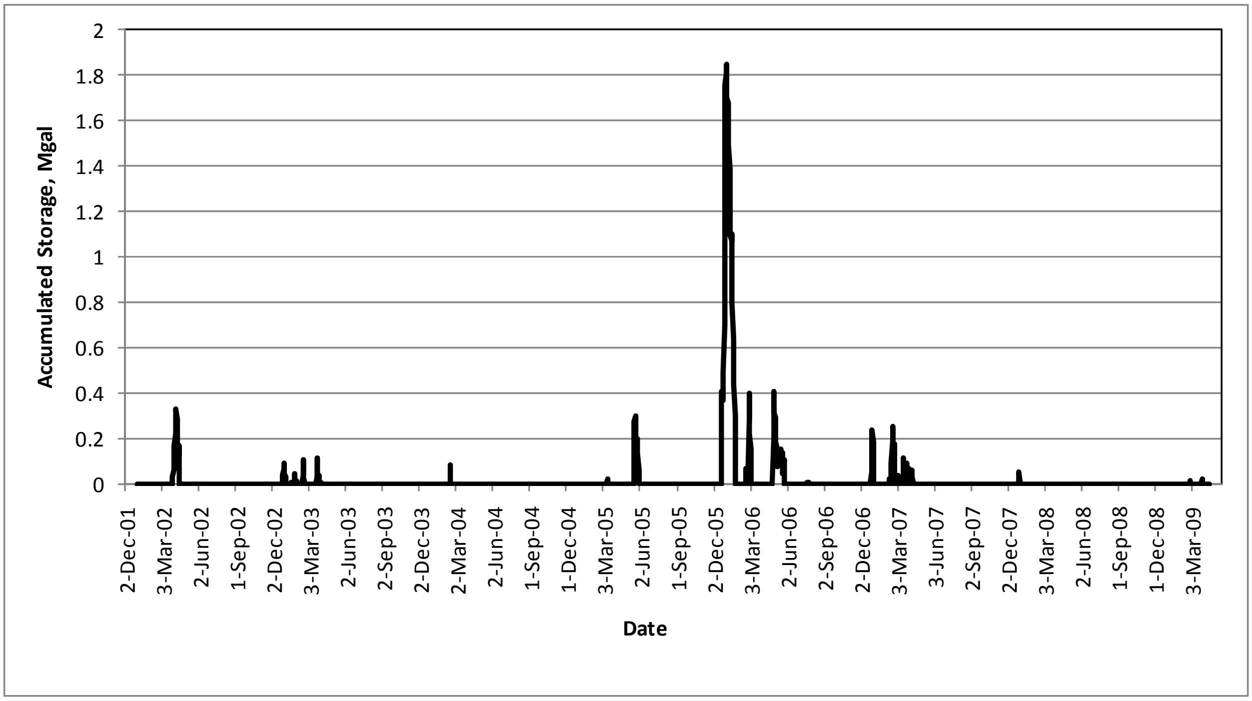


Figure A3
Storage Volumes with Limiting Flow = 0.40 Mgal/d

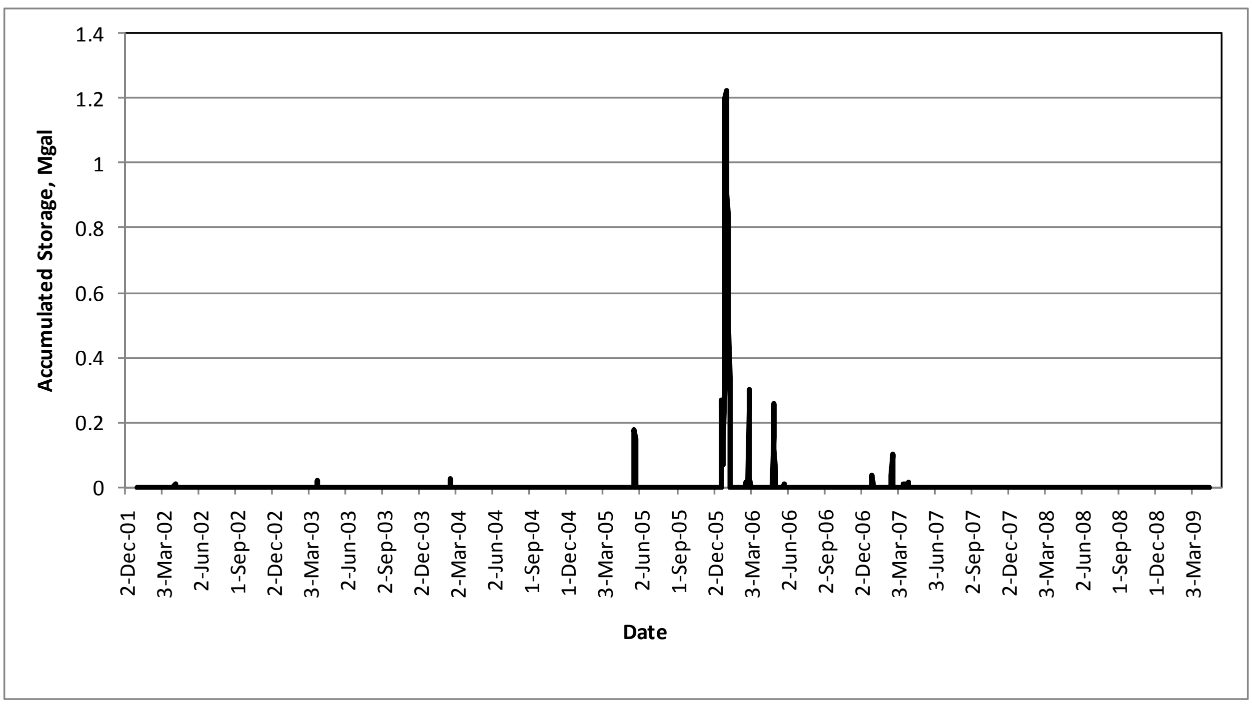


Figure A4
Storage Volumes with Limiting Flow = 0.45 Mgal/d

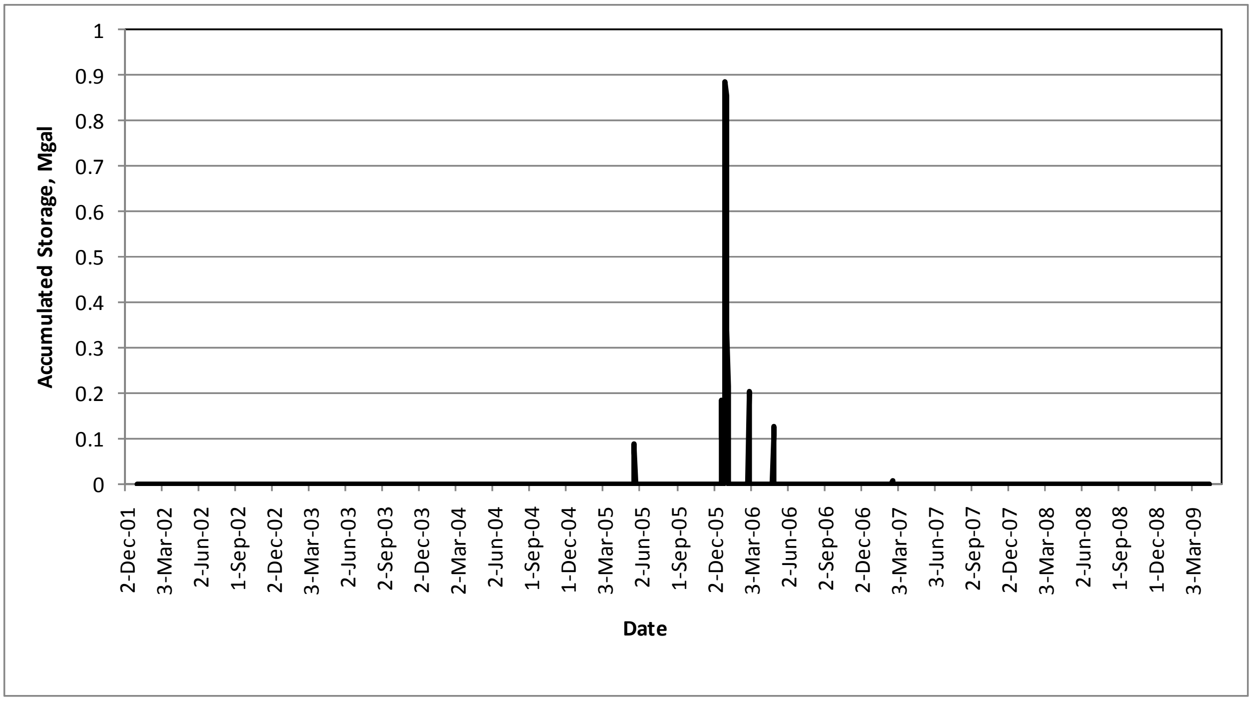


Figure A5
Storage Volumes with Limiting Flow = 0.50 Mgal/d

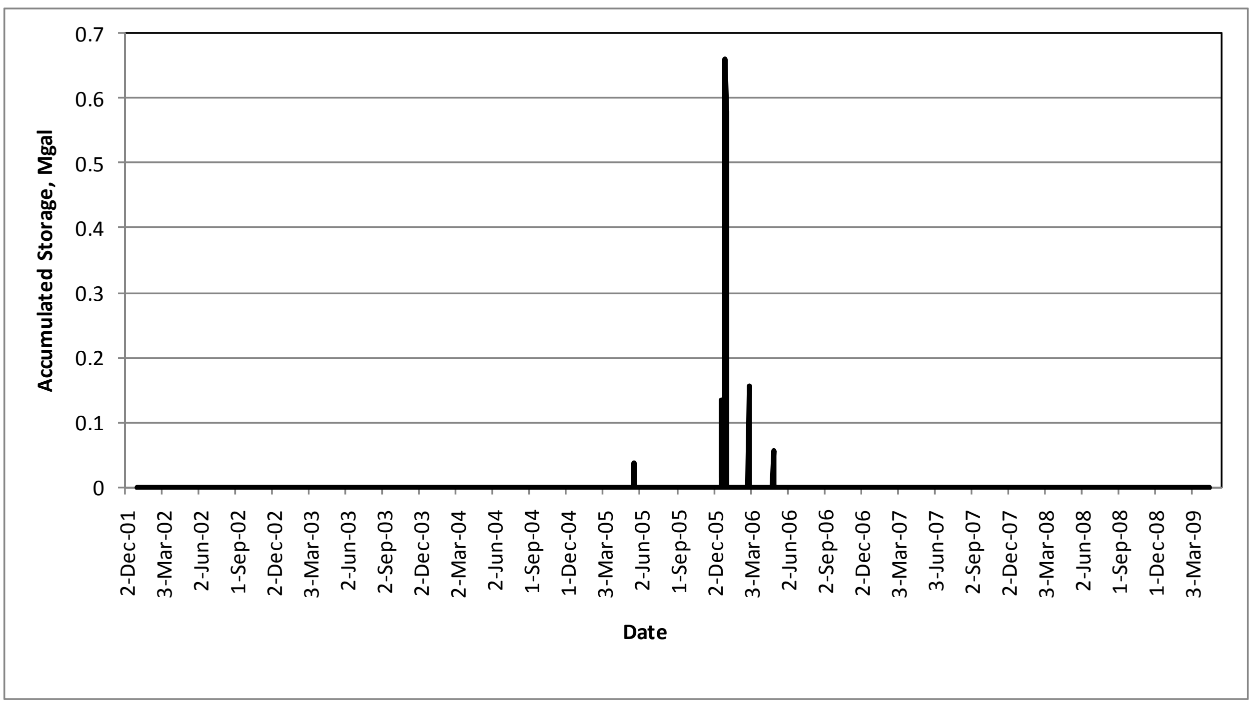


Figure A6
Storage Volumes with Limiting Flow = 0.55 Mgal/d

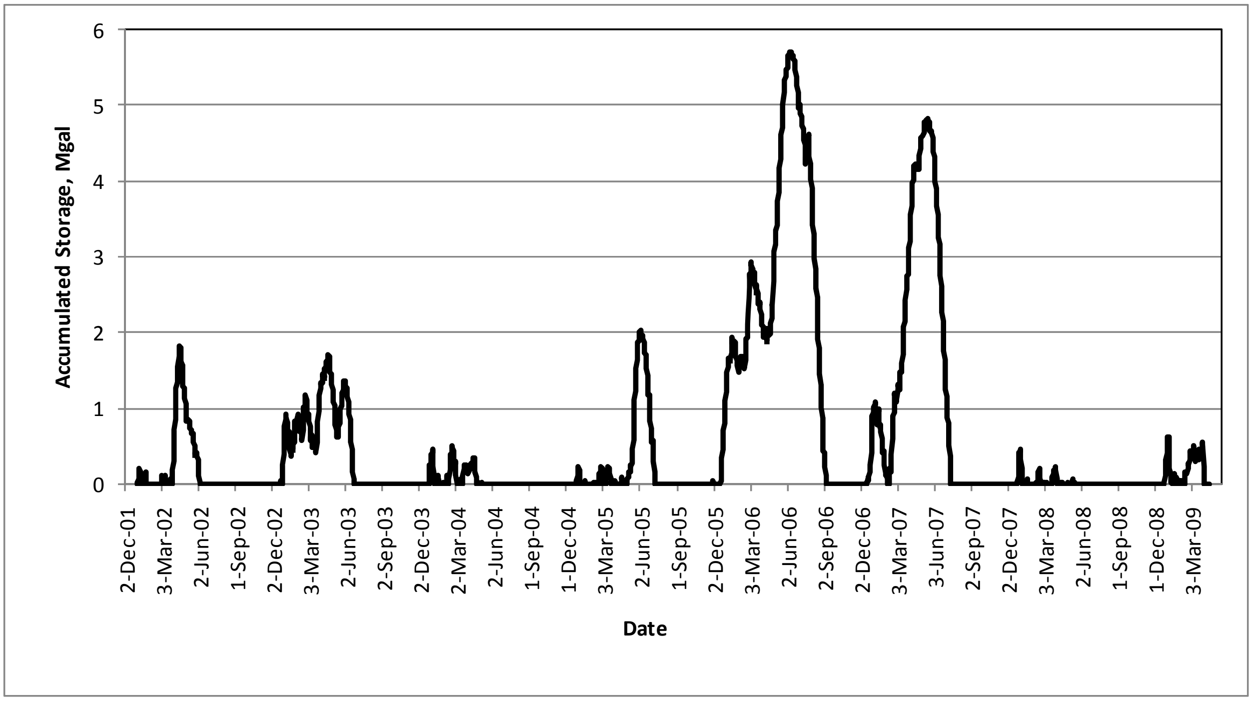


Figure A7

Storage Volumes with Limiting Flow = 0.30 Mgal/d with Flows from December 21, 2005 through January 2, 2006 Artificially Reduced to 0.40 Mgal/d

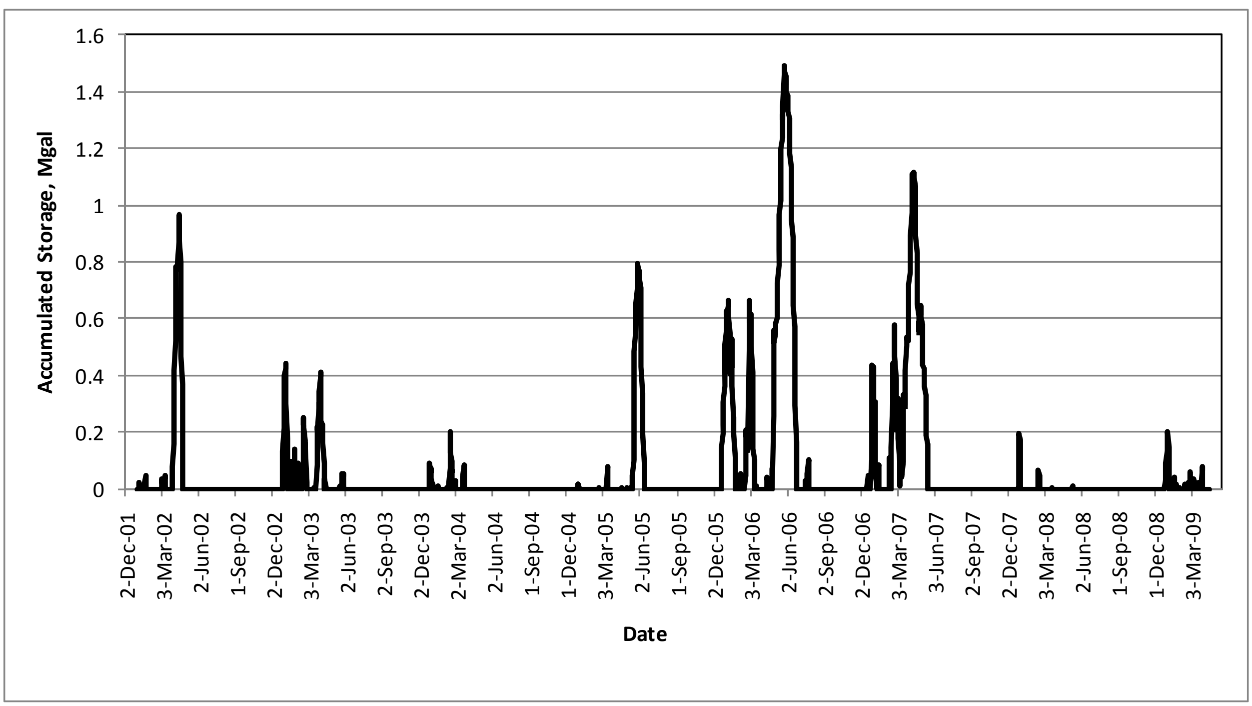


Figure A8

Storage Volumes with Limiting Flow = 0.35 Mgal/d with Flows from December 21, 2005 through January 2, 2006 Artificially Reduced to 0.40 Mgal/d

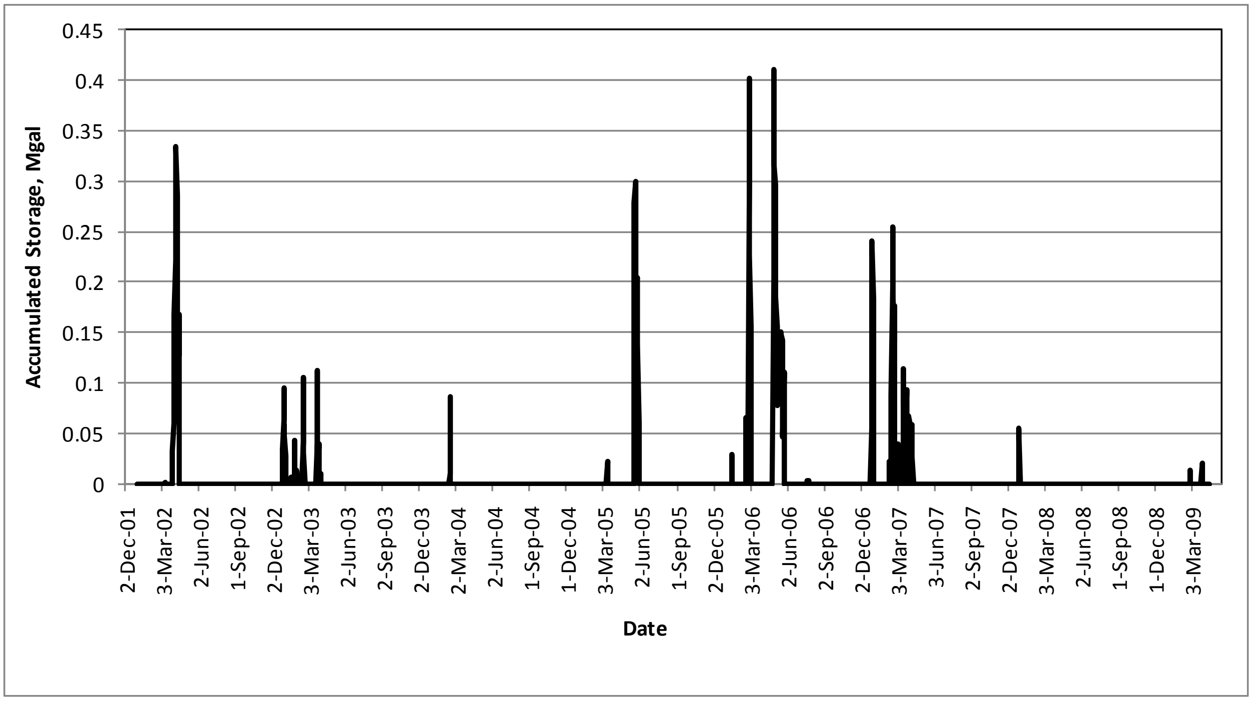


Figure A9

Storage Volumes with Limiting Flow = 40 Mgal/d with Flows from December 21, 2005 through January 2, 2006 Artificially Reduced to 0.40 Mgal/d

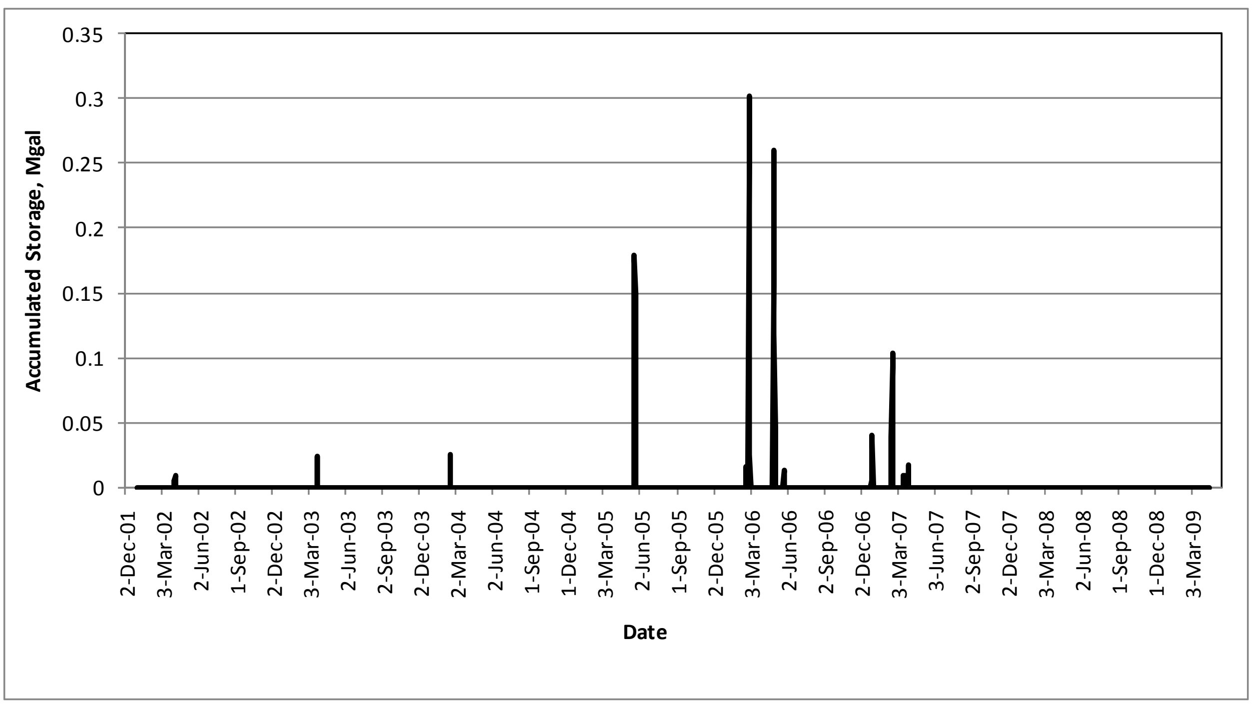


Figure A10

Storage Volumes with Limiting Flow = 0.45 Mgal/d with Flows from December 21, 2005 through January 2, 2006 Artificially Reduced to 0.40 Mgal/d

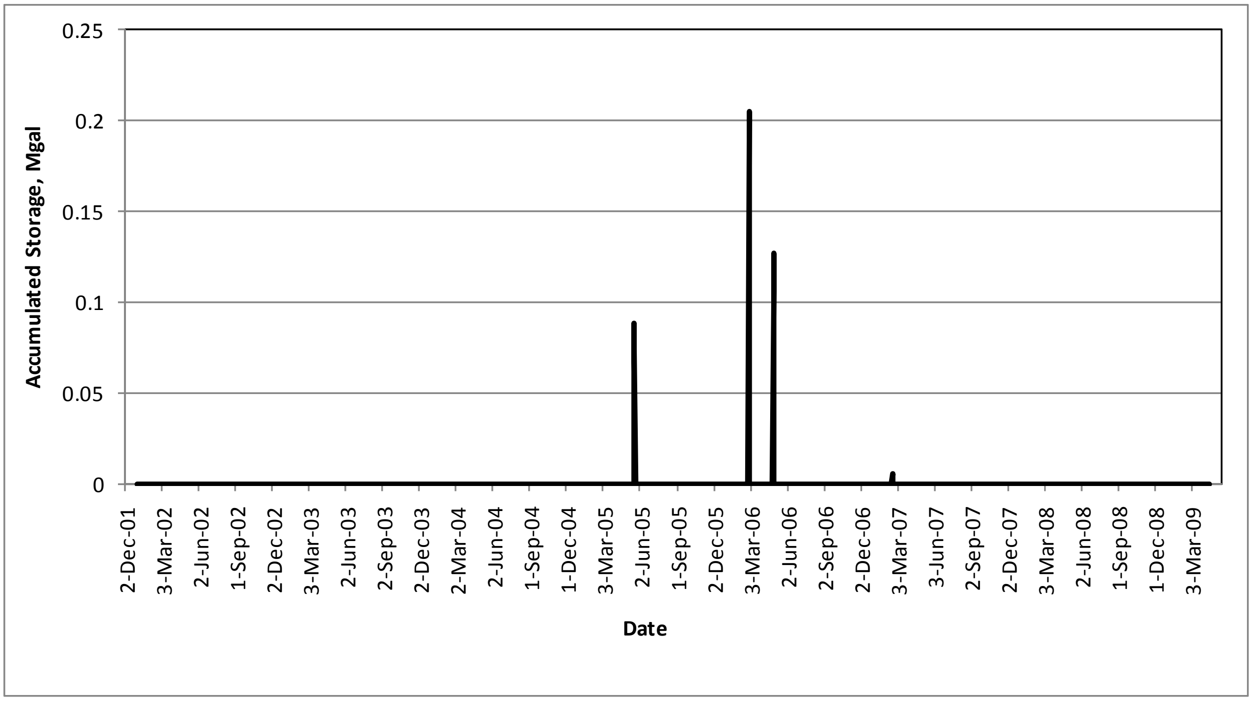


Figure A11
Storage Volumes with Limiting Flow = 0.50 Mgal/d with Flows from December 21, 2005 through January 2, 2006 Artificially Reduced to 0.40 Mgal/d

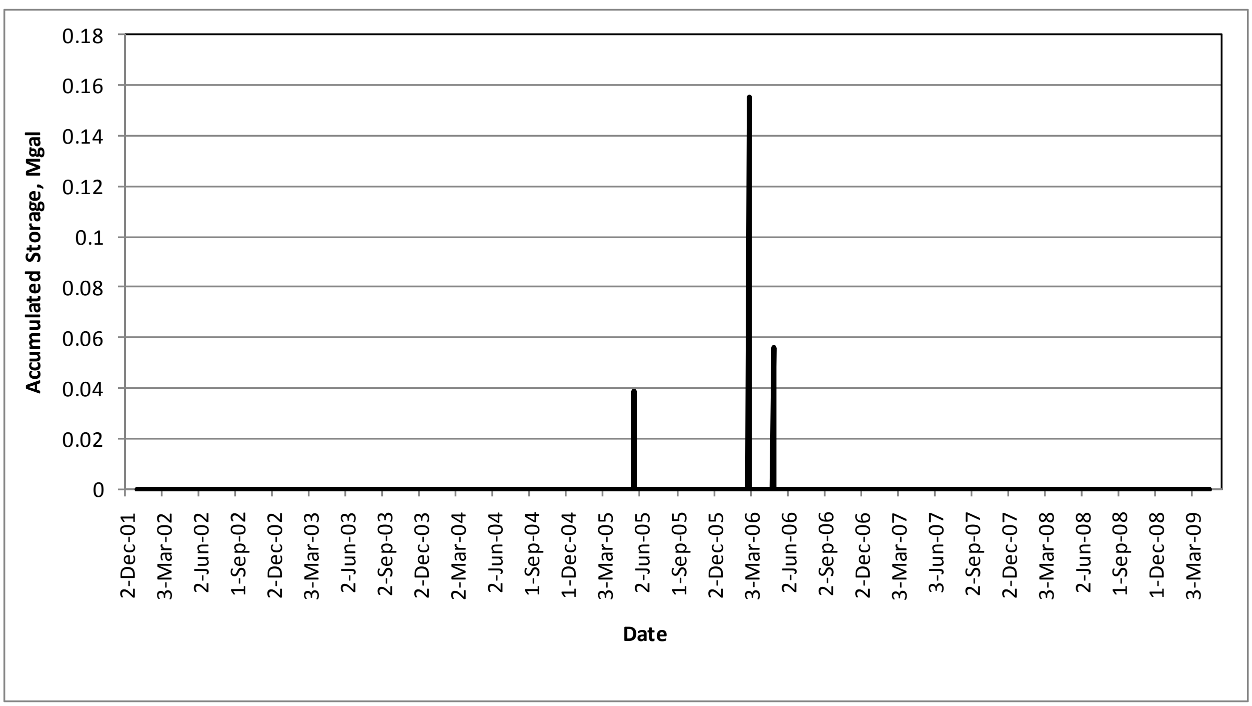


Figure A12
Storage Volumes with Limiting Flow = 0.55 Mgal/d with Flows from December 21, 2005 through January 2, 2006 Artificially Reduced to 0.40 Mgal/d

Appendix C

Preliminary Investigation of Wastewater Management Options – June 10, 2009

Donner Summit Public Utility District

Preliminary Investigation of Wastewater Management Options

REGISTERED PROFESSIONAL ENGINEER
JEFFREY R. HAUSER
Exp. 12-31-10
No. 31744
CIVIL
STATE OF CALIFORNIA

June 10, 2009

Prepared for
Donner Summit Public Utility District

Prepared by
ECO:LOGIC

3875 Atherton Road
Rockland, CA 95765

916.773.8100 TEL
916.773.8448 FAX

www.ecologic-eng.com

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Preliminary Investigation of Wastewater Management Options

1. INTRODUCTION

All wastewater from the Donner Summit Public Utility District (DSPUD) and the Sierra Lakes County Water District (SLCWD) is currently treated at the DSPUD wastewater treatment plant (WWTP) and is discharged either to the South Yuba River (SYR) or used to irrigate the Soda Springs ski area. In accordance with the requirements of the National Pollution Discharge Elimination System (NPDES) permit issued by the State of California Regional Water Quality Control Board, Central Valley Region (RWQCB), DSPUD discharges its effluent to the ski area whenever weather and other conditions are suitable for irrigation, but at least during the months of August and September. Typically, the period during which irrigation is possible, referred to as the “dry season” in this document, begins in early to mid-July and continues through late October or early November. The remainder of the year, during which irrigation is not possible, is referred to as the “wet season”.

In addition to many other requirements, DSPUD’s previous NPDES permit that was adopted on June 6, 2002, contained effluent limitations on ammonia-nitrogen (ammonia-n) and nitrate-nitrogen (nitrate-n) that were applicable to discharges to the South Yuba River. The ammonia-n limit was dependent on the temperature and pH of the effluent, but was generally in the range of about 3 to 6 mg/L as a monthly average to protect aquatic life in the river. The nitrate-n limit was 10 mg/L as a monthly average to protect human infants that may drink water from the river (not related to algal growth). DSPUD was issued a Cease and Desist Order that required full compliance with these limits by April 1, 2007. The 2002 permit also included a prohibition against causing fungi, slimes, or other objectionable growths in the South Yuba River.

With the main objective of complying with the ammonia and nitrate limits, DSPUD made major WWTP improvements in the years 2002 through 2006. Unfortunately, those improvements were not successful in attaining reliable compliance (reasons for noncompliance are discussed later in this document).

In April 2009, DSPUD was issued a new NPDES permit with a more stringent limit on ammonia-n (monthly average = 2.1 mg/L) and the same limit as the previous permit for nitrate-n (monthly average = 10 mg/L). The 2009 permit also contains a prohibition against causing water in the South Yuba River to contain biostimulatory substances that promote aquatic growths in concentrations that cause nuisance or adversely affect beneficial uses. These ammonia, nitrate, and biostimulation provisions are perhaps the most onerous issues in the 2009 NPDES permit; however, there are many other provisions that must be met, and all of these taken together have the potential of requiring major revisions to DSPUD’s treatment and/or disposal facilities that may cost many millions of dollars. Because of these concerns, DSPUD authorized preparation of this document to identify and evaluate on a conceptual level various wastewater management options that it may wish to consider to provide for cost-effective compliance with regulatory requirements. It is anticipated that this document will assist DSPUD to determine which wastewater management options should be considered in more detail, including specific cost evaluations, in a subsequent Facility Plan.

In the section that follows, the NPDES permit requirements are considered in more detail, including possible implications. In subsequent sections, wastewater disposal options are considered followed by treatment options to suit the disposal options. Then, combined disposal and treatment options are identified and subjectively evaluated, including a recommendation on whether or not there should be further study. Finally, additional issues that would impact many or all of the options are considered.

2. NPDES PERMIT REQUIREMENTS AND POSSIBLE IMPLICATIONS

Key effluent limitations for river discharge contained in the 2009 NPDES permit are summarized in Table 1. For each parameter, an assessment of the existing plant performance and compliance strategies are indicated.

In addition to effluent limitations, the permit contains receiving water limitations, most of which it is believed the existing plant can comply with. The one notable exception is the requirement that the discharge shall not cause the water in the South Yuba River to contain biostimulatory substances that promote aquatic growths in concentrations that cause nuisance or adversely affect beneficial uses.

The permit requires DSPUD to complete a number of special studies and reports, one of which is a study to evaluate the impact of the discharge on aquatic growths in the South Yuba River. This required study is the direct result of substantial algal growths in the South Yuba River downstream from the point of the DSPUD discharge in the spring of 2008. If it is found that the discharge is causing or contributing to growths that violate the biostimulation provisions, the permit will be reopened to impose additional restrictions needed for compliance. These could include new and/or more stringent effluent limitations on nutrients and/or prohibition against discharge during certain periods.

Although the previous tentative permit had allowed for dilution credits that substantially relaxed the limitations on nitrate (1.8 times higher) and dichlorobromomethane (24.5 times higher), these were eliminated in the final adopted permit. However, the permit does allow for possible reopening if DSPUD can provide new information to justify dilution credits. To allow dilution credits to be considered, DSPUD would have to install a discharge diffuser and flow monitoring station in the South Yuba River and conduct a mixing zone study. Even then, because nitrate is regulated based on a monthly average concentration and there may be months with little or no flow in the South Yuba River at the point of discharge, it is highly questionable whether dilution credits would be allowed.

Table 1
Key NPDES Permit Requirements, Plant Performance and Compliance Strategy

Parameter	Units	Effluent Limits[a]	Existing Plant Performance[b]	Compliance Strategy
BOD	mg/L	10/15/30	Generally compliant.	Continue/improve biological treatment, coagulation and filtration.
pH	Units	6.5 to 8.0[c]	Generally compliant.	Automatic chemical addition for alkalinity and pH control.
TSS	mg/L	10/15/30	Generally compliant.	Continue/improve biological treatment, coagulation and filtration.
Aluminum	µg/L	71/--/143	Frequently noncompliant. (---, ---, 620, 1310, 38.4, 127)	Monitor acid soluble aluminum. Possible Water Effects Ratio (WER).
Ammonia-N	mg/L	2.1/--/5.6	Frequently noncompliant. (Frequent non-certified lab data over 25 mg/L)	Improved treatment required.
Copper	µg/L	1.5/--/3.1	Frequently noncompliant. (4, 4, 7.8, 4.2, 5.9, 6)	Increase effluent hardness by using lime instead of soda ash for required alkalinity addition. Consider increased potable water pH. Possible Water Effects Ratio (WER).
Cyanide	µg/L	4.3/--/8.5	Occasionally noncompliant. (23, <2, 33, <2, DNQ 4, <2)	Evaluate future monitoring results. Consider changing from chlorine to UV disinfection. Consider immediate on-site testing without sample preservation.
Aldrin	µg/L	ND(d)	Rare noncompliance. (<0.002, <0.002, <0.002, DNQ 0.005, <0.002, <0.0028)	Evaluate future monitoring. Public education, source control if needed.
Alpha BHC	µg/L	ND(d)	Rare noncompliance. (<0.005, <0.005, 0.044, <0.005, <0.005, <0.00034)	Evaluate future monitoring. Public education, source control if needed.
Dichlorobromomethane	µg/L	0.56/--/1.2	Uncertain (e). (<0.5, <0.5, <0.5, DNQ 0.3, 1.2, 0.2)	Violations of this chlorine disinfection byproduct will be more likely with complete nitrification. Consider dilution credit, chloramination, UV disinfection.
Nitrate-N	mg/L	10/--/--	Frequently noncompliant. (Frequent non-certified lab data over 15 mg/L. Would be worse with good nitrification.)	Improved treatment required.
Silver	µg/L	0.23[d]	Rare noncompliance. (<0.09, <0.08, 0.26, 0.18, < 0.1, <0.12)	Evaluate future monitoring. Public education, source control if needed.
Zinc	µg/L	15/--/30	Frequently noncompliant. (22, 33, 22, 23.6, 25.3, 30.8)	Increase effluent hardness by using lime instead of soda ash for required alkalinity addition. Consider increased potable water pH. Possible Water Effects Ratio (WER).

Parameter	Units	Effluent Limits[a]	Existing Plant Performance[b]	Compliance Strategy
Manganese	mg/L	50[f]	Possible noncompliance. (---, ---, 8.7, 8.3, 52.8, 88.4)	Evaluate future monitoring and consider manganese removal in treatment process evaluations.
Total Coliform	MPN/100 mL	2.2, 23, 240[g]	Generally compliant.	Continue/improve biological treatment, coagulation, filtration, and disinfection.
Turbidity	NTU	2, 5, 10[h]	Generally compliant.	Continue/improve biological treatment, coagulation and filtration.

[a] Unless indicated otherwise, limits are Average Monthly/Average Weekly/Maximum Daily.

[b] Where a series of six results are shown in parenthesis, they are from special California Toxics Rule and related grab samples taken in June 2001, April 2002, November 2003, February 2004, December 2005, and December 2006, respectively. "DNQ" indicates an estimated value that is below the method quantitation limit.

[c] Range is based on instantaneous minimum and instantaneous maximum.

[d] Instantaneous maximum.

[e] Dichlorobromomethane is a chlorine disinfection byproduct that is mitigated by the presence of ammonia. Ammonia concentrations at the time of historical sampling are unknown.

[f] Annual average.

[g] 2.2 weekly median, 23 once in 30 days, 240 at any time.

[h] 2 daily average, 5 more than 5% of time in 24 hours, 10 at any time.

Since human health concerns regarding dichlorobromomethane are based on long-term average conditions (lifetime exposure) and there is believed to be substantial dilution available during most of the wet season, it is believed to be much more likely that dilution credits would be allowed for this parameter.

The reader is referred to the permit itself for complete coverage of all permit provisions.

A Cease and Desist Order was adopted together with the 2009 NPDES permit. This order provides a compliance schedule and interim permit limits for the following parameters: Ammonia, Nitrate, Copper, Cyanide, Zinc, Aldrin, Alpha BHC, and Silver. In essence, the Cease and Desist Order allows DSPUD to continue discharging these pollutants at historical levels while it pursues improvements to assure full compliance with the limits indicated in Table 1 by April 2014 (see Section 6 of this document for schedule of activities needed to attain compliance). However, since the permit limit on nitrate is the same as it was in the 2002 permit, DSPUD is not protected against mandatory fines for violation of the 10 mg/L nitrate-n limit.

Out of all the requirements contained in the NPDES permit, those regarding effluent ammonia and nitrate concentrations and biostimulation in the South Yuba River are considered the most problematic, because compliance is likely to require major improvements to the DSPUD wastewater treatment and/or disposal systems. Possible options for addressing these issues are discussed in the remaining sections of this document.

3. EFFLUENT DISPOSAL OPTIONS

In the following paragraphs, various alternatives for wastewater effluent disposal are considered. The methods of disposal will govern the required levels of treatment, which are considered later in this document.

Wet Season Direct Discharge to SYR, Dry Season Spray Irrigation

These are the methods of effluent disposal currently used by DSPUD. Key issues are the need to upgrade the plant for compliance with existing ammonia, nitrate and disinfection byproducts limits. Additionally, the need to prevent biostimulation in the South Yuba River would undoubtedly result in much more stringent requirements on nitrate, plus possible new requirements on phosphorous and/or other biostimulatory substances, adding much more to the cost of improvements, if feasible at all. Even after such improvements, it is likely that algae growths could continue to occur downstream from the DSPUD discharge due to nutrients from other point and nonpoint sources. The degree to which the DSPUD discharge would contribute to such growths would be in question. Because of these issues and because long-term and costly studies would be required to determine appropriate nutrient limitations for river discharge in algae growth periods, continued use of this option is considered to be infeasible. At least some modification of current effluent disposal practices is believed to be needed.

Limited Wet Season Direct Discharge to SYR, Seasonal Storage, Dry Season Spray Irrigation

This option is similar to that above, with a major difference: seasonal storage facilities would be provided to allow curtailing direct discharge during periods in the wet season when flows, temperatures, solar exposure and other conditions would facilitate algal growths in the South Yuba River, regardless of the presence or absence of the DSPUD effluent. In other words, DSPUD would stop discharging effluent when the effluent would probably contribute to nuisance biostimulation in the river. An investigation is needed to determine the conditions and times of effluent storage. This topic will be addressed in the biostimulation study that DSPUD is currently proceeding with as required in the NPDES permit.

An analysis of historical springtime flows for the years 2002 through 2008 was completed to assess the magnitude of possible storage requirements. For each year, beginning with the day before irrigation was started in that year and extending backwards, the volume of storage that would have been required to contain all of the plant effluent was determined as a function of the number of days. The results are shown in Figure 1. It is currently estimated that springtime discharges to the South Yuba River would have to be ceased approximately 45 to 60 days prior to the start of spray irrigation disposal. Based on the results shown in Figure 1, storage requirements could be in the 15 to 20 million gallon range (approximately 45 to 60 acre-feet), not including any allowance for precipitation in the reservoir area, and not allowing for any growth or increase in spring occupancy within the service area.

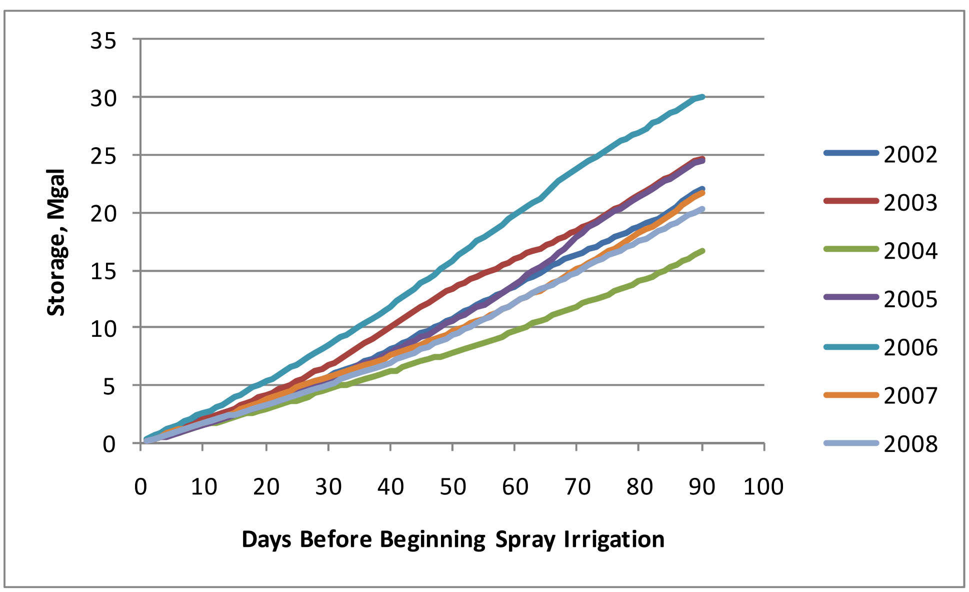

Figure 1
Seasonal Storage Requirements Based on Historical Flows

The effluent stored in the wet season would have to be disposed of by spray irrigation during the following dry season, greatly increasing the land area required for irrigation. Assuming a depth of two feet of water could be applied during the dry season, the land area required to dispose of the stored effluent could be in the range of 20 to 30 acres (to be verified and adjusted as needed), in addition to that required for dry season flows. Currently 45 acres are used to dispose of dry season flows. Thorough water balance calculations for a specific project would have to be performed to determine actual requirements.

Substantial questions exist regarding management of the reservoir and control of flows to and from it. Ideally, the reservoir would collect and store only wastewater effluent and limited precipitation during the time that the effluent is routed to storage and remains in storage in the spring and summer. To prevent accumulation and handling of precipitation during the remainder of the year, a reservoir outlet valve could be left open to allow natural runoff to flow from the reservoir to the river. However, unless specific steps are taken to mitigate the situation, dead algae and other debris accumulated in the reservoir as well as soil erosion from the reservoir area would be flushed into the river, which would be unacceptable. Two potential options to mitigate this problem are: 1) to line the reservoir, and 2) maintain a minimum pool for settling and have periodic discharges during times of high river flow. These options are discussed below.

If a suitable impermeable liner was used within the reservoir, the reservoir could be drained and then cleaned each summer. Effluent could be used to wash down the liner, and the used wash water, depending on quality, could be disposed of by irrigation or routed back through the treatment plant. Once the reservoir is cleaned, allowing precipitation to drain through the reservoir should not pose any significant water quality issues. The liner would protect the reservoir area from erosion. During the winter, snow would accumulate in the reservoir area, but most of this snow should melt away naturally before the reservoir outlet valve must be closed and springtime effluent storage initiated. If undesirable quantities of snow remained in late spring, some of the effluent otherwise being directly discharged to the river could potentially be sprayed over the remaining snow in the reservoir to melt it, with the combined effluent and snowmelt being allowed to flow to the river. There would be issues with handling effluent stored in the fall, if such storage is required after cessation of irrigation disposal to prevent biostimulation in the river at that time (to be determined in the biostimulation study). Hopefully the duration of storage would be short enough and temperatures cold enough to prevent significant deterioration in the quality of the stored effluent, which would have been previously treated to river discharge standards. In that case, the stored effluent could be gradually released to the river, once river discharge is possible. This potential issue will require further evaluation.

Under the minimum pool option, the reservoir would function as a settling basin for soils eroded from the reservoir catchment area and for dead algae or other debris that would accumulate in the reservoir. The reservoir water level would never be lowered below the minimum level required to provide this function. Therefore, the total volume of the reservoir would be this minimum pool volume plus the volume required for active storage. Each summer, the reservoir would be emptied down to the minimum pool elevation by spray irrigation at appropriate reuse sites. If any additional storage is required before river discharge can be started in the fall, the volume then stored would accumulate above the minimum pool, as would any precipitation occurring in the reservoir drainage area. At times of high river flows during the winter and spring, the reservoir would be rapidly emptied down to the minimum pool by direct discharge into the South Yuba River. It is hoped that permitting for the periodic reservoir discharge at times of high river flows and turbidity could be obtained. There is some precedence for a permit of this type.

If winter discharge from the reservoir under one of the options above or an alternative plan were not allowed, the reservoir would have to be sized to contain the 100-year frequency precipitation in the reservoir catchment area during the wet season, in addition to the required effluent storage volume. All of the stored water would have to be disposed of by irrigation in the dry season. As a result, this option would drastically increase the size requirements and costs for the storage reservoir and irrigation disposal areas.

Finding a suitable reservoir site and easements to and from it, acquiring the land, extending electrical service, and addressing environmental issues would be significant challenges in implementing this alternative. In the Treatment and Disposal Facilities Plan prepared for DSPUD in June 1984, reservoir sites for a seasonal storage reservoir of similar size requirements were investigated. The most promising site was in a ravine across the South Yuba River and approximately ½ mile northwest of the existing DSPUD discharge location.

Wet Season Storage, Dry Season Irrigation

Under this option, all wastewater effluent would be stored during the wet season and disposed of by irrigation in the relatively short dry season. This option, if feasible, would be preferred by the RWQCB because it would eliminate all direct impacts on the South Yuba River. Additionally, the level of treatment required for irrigation disposal would be lower than required for river discharge, resulting in lower treatment plant construction and operation costs.

The main problems associated with this option are the need for one or more extremely large storage reservoir(s) and the large land area required for irrigation. To properly determine the volume of storage required and the area needed for irrigation, detailed water balance calculations would have to be completed for a specific project. However, rough estimates can be developed. The estimates that follow are based on existing flows, without any allowance for additional growth or increased occupancy of existing units, which would increase the required storage volumes and irrigation areas. Reductions in infiltration and inflow volumes, if assured, would reduce the requirements.

If it is assumed that all of the DSPUD effluent is stored from November 1 through June 30 (actual storage duration would likely be longer for 100-year precipitation conditions), the volume of wastewater stored based on flows from 2002 through 2008 could be over 70 Mgal. Additional storage would have to be provided for 100-year return frequency precipitation in the catchment area of the storage reservoir(s), less evaporation from the reservoir(s). Depending on the configuration of the reservoir(s), the resultant total storage requirement could be more than 200 Mgal or 600 acre-feet (to be verified and adjusted as needed based on detailed water balance calculations).

Assuming a total applied water depth of about two feet in the subsequent dry season, and neglecting evaporation from the storage reservoir, approximately 300 acres of irrigation land would be required to dispose of the stored effluent (to be verified and adjusted as needed based on detailed water balance calculations). Additional irrigation land would be required for the dry season flows, but would be partly offset by evaporation from the storage reservoir. It is estimated that the total land requirement could be over 300 acres under irrigation, plus buffer areas.

As discussed for the seasonal storage alternative, finding a suitable reservoir site and spray irrigation disposal area and easements to and from them, acquiring the land, extending electrical service, and addressing environmental issues would be significant challenges in implementing this alternative. In this case, all of those challenges would be amplified due to the larger land areas and facility sizes involved. The total land requirement for storage and disposal plus buffer land could be around 500 acres for existing development and occupancy rates. In the Treatment and Disposal Facilities Plan prepared for DSPUD in June 1984, reservoir and irrigation disposal sites for a year-round land containment system such as described above were investigated. The most promising site for both storage and disposal was believed to be just west of Serene Lakes, which is an area currently being considered for development by Royal Gorge.

Because of the anticipated high costs (including land acquisition costs), large land requirements, environmental impacts, and anticipated public resistance, this alternative is not considered to be feasible.

Year-Round Direct Discharge to South Yuba River

This option is mentioned for completeness. However, it is recognized that there would be tremendous public and regulatory opposition to a year-round discharge. Even if it were to be allowed at all, it is expected that treatment requirements would be extreme and cost-prohibitive. This option should not be considered further.

Subsurface Disposal

Subsurface disposal via leach fields or percolation basins or similar systems, if feasible, could be considered for seasonal or year-round use. However, in the Donner Summit area, it is unlikely that a site with geologic and soil conditions that would allow the effluent to stay underground long enough to blend with natural groundwater and lose its identity as wastewater effluent could be identified. Rather, it is likely that, if adequate soil conditions could be found to allow the effluent to be disposed of below the ground surface initially, bedrock would be encountered below, causing the effluent to flow laterally and surface at some location down gradient from the point of discharge. Extensive soils, geological and hydrological investigations and modeling would have to be completed to determine the fate of the effluent. Under the best likely scenario, the effluent could exit the ground from the bed of a flowing surface water course, such as the South Yuba River. If flow to the surface water course was the clear fate of the effluent, discharge requirements needed to protect the beneficial uses of the surface water course (including prevention of biostimulation) would be imposed. However, it may be possible to attain some of the required treatment naturally as the effluent moves through the soil. It is noted that the Tahoe-Truckee Sanitation Agency (TTSA) wastewater treatment facility in Truckee disposes of its effluent into permeable soils along the Truckee River and that their discharge requirements are established to protect the beneficial uses of that river, with some credit given to incremental natural treatment during flow through the permeable soils.

A key issue with regard to subsurface disposal is groundwater degradation. Certainly, the discharge would not be allowed to cause the underlying groundwater to exceed applicable water quality criteria. For example, it would not be allowed to cause nitrate-n concentrations in excess of the 10 mg/L drinking water limit. This alone could necessitate a full nitrification and denitrification system similar to that required to meet existing NPDES permit requirements. Salinity and other issues would also exist. It would not be possible to have a subsurface discharge without increasing above background levels the groundwater concentrations of several constituents. The degree to which such increases may or may not be allowed would have to be determined by working with the Regional Board.

It is important to note that subsurface effluent disposal was the normal means of disposal in the Donner Summit area prior to the late-1980s. DSPUD’s effluent was discharged to a large leach field along the South Yuba River. However, the effluent did not stay underground; rather, much of it surfaced and flowed on top of the ground into the river. Even the effluent that did stay underground as it flowed into the river undoubtedly contributed to the unacceptable impacts that were obvious in the river, mainly attached algae growths on the river bottom during the summer and fall. In addition to DSPUD, all of the lodges, ski areas, businesses and residences in the Norden area had on-site subsurface disposal systems and many of those were known to fail with surfacing effluent. All of these subsurface disposal systems were abandoned with the Norden extension of the DSPUD sewage collection system in the late 1980s.

It was because of the failures of subsurface disposal systems and the lack of reasonably cost-effective alternatives for containment of all effluent on land that seasonal direct discharge to the South Yuba River was first permitted in the late-1980s.

Export Sewage to TTSA

Under this option, the DSPUD and SLCWD wastewater would be pumped over the summit and would connect with existing sewers in the Truckee area for flow to the TTSA wastewater treatment plant. The specific location for connection to existing sewage piping is currently unknown. According to Blake Tresan, District Engineer for the Truckee Sanitary District (TSD), it is unlikely that a connection would be made at the west end of Donner Lake, because sewage from this area flows through a series of six pumping stations, which are already near capacity, to get to the east end of Donner Lake. It is much more likely that the DSPUD wastewater would be piped through its own pressure pipe all the way to the east end of Donner Lake or all the way to the TTSA interceptor sewer. Assuming a pipeline from the DSPUD WWTP to the east end of Donner Lake, the total pipe length might be around 55,000 feet. Assuming an average cost of about $100 per lineal foot, the construction cost for the pipeline could be around $5.5 million.

The export pump station would be additional. With engineering, environmental, administrative and other related costs, plus a reasonable contingency allowance, the total cost of the export pump station and pipeline could be around $10 million.

The potential of DSPUD sewage going to the TTSA facility was discussed with Marcia Beals, General Manager for TTSA. Ms. Beals had the following concerns:

1. The TTSA plant was recently expanded from 7.4 to 9.6 Mgal/d. The 9.6 Mgal/d capacity was developed to serve projected buildout within the existing service area. Without a subsequent expansion (which is considered unlikely), the flow from DSPUD, if allowed, would effectively displace future development in the TTSA service area. This is unlikely to gain approval.
2. The recent TTSA expansion and increase in effluent flow to the Truckee River were very difficult to get approved through the environmental and regulatory processes (planning, design, and construction took approximately 10 years). The Truckee River is a water supply for the City of Reno and terminates at Pyramid Lake within the Paiute Indian Reservation. Accordingly, there are large and powerful interests that oppose any activity that would potentially degrade the quality of the Truckee River. TTSA was forced to upgrade their level of treatment, which includes nitrogen and phosphorous removal, contributing to a total capital cost (including engineering, administration, environmental, as well as construction) of approximately $70 million for the expansion project.
3. In order for DSPUD sewage to flow to TTSA, DSPUD would have to annex to TSD. Approval of both TSD and TTSA would be required.

Although the method of determination of DSPUD's cost to buy capacity in the TTSA facility, if allowed at all, is not currently known, TTSA's current connect fee for an equivalent dwelling unit (EDU) is $5,000. Using the current combined DSPUD/SLCWD peak week flow of about 600,000 gpd and assuming a flow of 300 gpd per EDU (not verified as an appropriate basis with TTSA), the existing DSPUD/SLCWD flows may be equivalent to approximately 2000 EDU. If the 2000 EDU were located within the current TTSA service area for which TTSA plant capacity has already been built, the total connection fee would be $10 million. However, for an annexation area that would require new capacity to be built, the buy-in cost would undoubtedly be much higher. The current TTSA service charge is $288 per year per EDU. However, TTSA also collects about 15 percent of their revenue from property taxes. Since properties within the DSPUD and SLCWD service areas would not be subject to the tax, the service charge would have to be increased accordingly. Additionally, service charges would have to be paid to TSD at the current rate of $19/month per EDU.

The discussion above is based on exporting raw sewage to TTSA. Consideration could also be given to exporting treated sewage; however, there is no apparent advantage to that. Unless the DSPUD effluent were piped all the way to the TTSA treatment facility, the effluent from DSPUD would get blended with raw sewage in Truckee and would still use up capacity in the TTSA WWTP. Perhaps there could be recognition of the lower loading of pollutants in establishing user fees, but that would be rather inconsequential to the other costs involved. All of the issues with regard to increased sewage effluent flows in the Truckee River would still exist and the costs for export facilities and buy-in to the TTSA system would remain nearly the same. Since DSPUD would also incur the cost of treatment at DSPUD, the total cost for exporting effluent would likely be substantially higher than for exporting raw sewage. If DSPUD treated to the same level as TTSA and piped the effluent to blend with the effluent of the TTSA facility for joint disposal, the costs for treatment at TTSA could potentially be avoided, but the costs for treatment at DSPUD undoubtedly would be at least as high as if the discharge were to the South Yuba River. When all costs associated with exporting the effluent are considered, this option would be much more costly than a South Yuba River discharge. Additionally, it is highly unlikely that TTSA would allow the DSPUD effluent to be combined in that manner, as TTSA would have responsibility for the combined effluent quality.

Based primarily on very difficult environmental and institutional issues that could delay a prospective project for many years, if it could be approved at all, it is considered unlikely that export of sewage to TTSA would be a viable option for DSPUD. Additionally, it does not seem as though there could be a significant cost incentive (if any) for pursuing this option. There are also potential water rights issues associated with moving the discharge from the South Yuba River to the Truckee River.

Summary of Disposal Options

All of the disposal options considered above and the pros and cons of each are summarized in Table 2. Recommendations on which options should be considered further are included in Section 5.

Table 2
Summary of Disposal Options

Option	Pros	Cons	Comments
Subsurface Disposal	❑ No direct river discharge	❑ Probable lack of suitable soils / geology ❑ Probable effluent surfacing ❑ Groundwater degradation	❑ This method of disposal was extensively used historically in the Donner Summit area, but failed.
Wet Season Storage, Dry Season Irrigation	❑ No direct river discharge ❑ Lowest treatment requirements	❑ Huge land area requirement ❑ High cost	❑ Finding and acquiring adequate suitable land would be very difficult.
Limited Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	❑ No direct river discharge when nuisance biostimulation could occur	❑ Cost and operational issues associated with seasonal storage	❑ A direct discharge from the seasonal storage reservoir to the SYR at times in the winter is needed to eliminate major storage and disposal issues associated with wet season precipitation in the reservoir area.

Option	Pros	Cons	Comments
Wet Season Discharge to SYR, Dry Season Irrigation	❑ No seasonal storage reservoir	❑ Undetermined extreme low-level nutrient requirements to mitigate biostimulation. ❑ Discharge may still be suspect for contributing to biostimulation	❑ This option is judged to be infeasible.
Year-Round Discharge to SYR	❑ No land disposal area or systems required	❑ Unacceptable to public and regulatory agencies	❑ Mentioned for completeness, but no further consideration recommended.
Export Raw Sewage to TTSA Sewers	❑ Eliminate discharge to SYR ❑ Eliminate DSPUD WWTP ❑ Eliminate land disposal on Donner Summit	❑ TTSA capacity committed to existing service area and difficult to expand further ❑ Difficult environmental issues associated with discharge to Truckee River ❑ Water rights issues	❑ Working through the political and environmental issues involved would undoubtedly take many years and would likely fail. ❑ It is unlikely that there would be a significant cost incentive that would justify pursuing this option.
Export Treated Effluent to TTSA Sewers	❑ Eliminate discharge to SYR ❑ Eliminate land disposal on Donner Summit	❑ Same as above, plus: ❑ DSPUD would still need capacity in TTSA plant, though pollutant load reduced. ❑ DSPUD continues to operate its own WWTP and thus must pay for two plants.	❑ No significant advantage and many disadvantages compared to exporting raw sewage.
Export Treated Effluent to TTSA Discharge Point or Other Truckee River Location	❑ Eliminate discharge to SYR ❑ Eliminate land disposal on Donner Summit ❑ Eliminate need to expand TTSA WWTP	❑ Environmental issues for Truckee River are even more difficult than for SYR. ❑ Required treatment would be at least as difficult and expensive as staying in the SYR. ❑ Water rights issues	❑ This is simply a relocation of the DSPUD discharge from the SYR to the Truckee River. ❑ The one benefit of this option compared to exporting raw sewage to TTSA sewers is that no TTSA plant expansion would be needed. However, the treatment system required at DSPUD would more than offset this advantage.

4. TREATMENT OPTIONS

The level of treatment to be provided will depend on the effluent disposal option. In this section, options for modifying the plant to meet the requirements in the existing NPDES permit are considered as a base case. This level of treatment would be appropriate for continued wet season discharge to the South Yuba River during times when biostimulation is not a threat. Subsequent to developing options for this base case, differences in treatment for other disposal options are discussed.

The existing wastewater treatment plant is intended to provide ammonia and nitrate removal by biological treatment. However, as previously indicated, the plant does not reliably meet requirements for these parameters. Most options for improving the plant are also based, at least partly, on biological treatment to remove ammonia and nitrate. Therefore, before beginning a discussion of specific options for improving the plant, it is helpful to discuss this type of biological treatment in general, and to discuss the existing wastewater treatment plant.

Biological Treatment to Remove Ammonia and Nitrate

Biological treatment to remove ammonia and nitrate is accomplished by the processes of nitrification and denitrification. Nitrification is the sequential oxidation of ammonia to nitrite and then nitrate by ammonia oxidizing bacteria (AOB) and nitrite oxidizing bacteria (NOB). Collectively, the AOB and NOB are referred to as nitrifying bacteria or nitrifiers. Denitrification is the conversion of nitrate to nitrogen gas by bacteria that use organic substances in the wastewater (measured as BOD) or supplemental organic materials as their food and use the nitrate as a substitute for oxygen for their respiration. The bacteria that can use nitrate as a substitute for oxygen will do so only when oxygen is not available. Biological treatment to remove ammonia also results in removal of BOD.

Since nitrification must occur before denitrification can be accomplished, one option for a nitrification and denitrification system would be to have one or more aerobic basins for BOD removal and nitrification followed by an anoxic basin for denitrification. However, with this configuration, essentially all of the influent BOD would be consumed in the aerobic basins, leaving no external food for the bacteria that accomplish denitrification in the anoxic zone. Without an external food supply, denitrification would occur very slowly using decaying bacteria as the food source. To speed up the denitrification process, allowing smaller reactor basins, methanol or another suitable food could be added to the anoxic zone.

As an alternative to the aerobic-anoxic configuration mentioned above, the anoxic basin can be located upstream from the aerobic basin, if the nitrate formed in the aerobic basin is recycled back to the anoxic basin for denitrification. In this case, the incoming wastewater would be the food for the bacteria accomplishing denitrification, potentially eliminating the need for purchasing and feeding a supplemental food supply. This anoxic-aerobic configuration with a mixed liquor recycle stream from the aerobic basin to the anoxic basin (such as currently exists at DSPUD) is called a Modified Ludzack-Ettinger (MLE) system. The return activated sludge

(RAS) stream from the secondary clarifier serves as an additional nitrate supply to the anoxic zone. Unfortunately, the mixed liquor recycle stream (and perhaps the RAS) will contain a substantial amount of oxygen, and this oxygen must be consumed, using up valuable food, before denitrification can take place. Thus, the amount of denitrification that can be accomplished is limited by the food supply, the amount of nitrate that can be recycled, and the amount of oxygen that is recycled with the nitrate.

Nitrifying bacteria grow relatively slowly, particularly in cold conditions, which makes reliable nitrification at DSPUD difficult. At DSPUD, wastewater flows and loads during the fall months are much lower than those that occur in the winter. A sudden onset of high loads typically occurs around Christmas and high loading conditions occur sporadically throughout the winter in response to peak temporary occupancies in lodges and homes and peak usage of ski areas. The amount of nitrifying bacteria that can be grown from the wastewater in the fall is inadequate to handle the sudden onset of peak winter loads. When this is combined with the fact that nitrifying bacteria are particularly slow growers in the winter, it is considered necessary to feed ammonia in gradually increasing amounts during the fall months to build up the nitrifier population in preparation for winter loads. Intermittent ammonia addition between winter peak load events is also considered beneficial to maintain the nitrifier population. A more complete discussion of the influent loading patterns to the plant and the need for ammonia addition is included in the letter report from Jeff Hauser of ECO:LOGIC Engineering to Tom Skjelstad of DSPUD, dated January 15, 2009.

As discussed in the above-mentioned letter report, the required ammonia additions during low load and low flow conditions can result in very high influent TKN concentrations, perhaps as high as 200 mg/L at times. The supplemented influent TKN could be continuously over 125 mg/L for weeks in late November and early December. When starting from such high influent TKN concentrations, it may not be cost-effective to get down to the required effluent nitrate-n concentration of 10 mg/L using a two-stage MLE system, because this would require extremely high mixed liquor recycle flows, which would result in large amounts of oxygen delivered to the anoxic zone. The anoxic zone would have to be enlarged and methanol (or alternative carbon source) added to offset the oxygen supply. A better approach may be to add another anoxic basin downstream from the aeration basin or a second and separate nitrate removal system, such as a denitrification filter after the secondary clarifiers. In both cases, methanol or an alternative carbon source would be added to provide the food necessary for denitrification in this second location, but the total carbon addition requirements would be lower than for the original two-stage alternative, due to less oxygen impacts. Also, with two locations for nitrate removal, a higher reliability could be attained.

If a second anoxic basin is used after the aerobic basin, a final small aeration basin would then be added to strip out remaining nitrogen bubbles and to increase the effluent dissolved oxygen concentration. In this case, there would be four reactor basins in series (anoxic-aerobic-anoxic-aerobic). In this four-stage system, the mixed liquor recirculation stream from the first aerobic basin to the first anoxic basin, together with RAS flow, would be used to deliver that amount of nitrate that could be denitrified in the first anoxic zone using the food of the influent wastewater. The remaining nitrate would be removed in the second anoxic basin using the supplemental food source (methanol or other appropriate substance).

A potential alternative to feeding ammonia during low-load periods to build up a nitrifier population by growing the nitrifiers in the reactor basins is to purchase cultured nitrifiers grown off-site. There are several companies that can supply nitrifying bacteria in liquid suspensions, which can be dosed into the DSPUD treatment system. For this scheme to work, the nitrifiers would have to be added in the right amounts just as each high load event occurred and would have to begin removing ammonia immediately. If the nitrifiers were added before the peak loads actually occurred, supplemental ammonia would have to be added to support the added nitrifiers, in which case the benefit of adding the nitrifiers instead of growing them in the process would be largely eliminated. Unfortunately, when nitrifiers grown off-site in a “laboratory” are added to the process, there will undoubtedly be an acclimation period before the full ammonia removal potential would be realized. Also, loss of a substantial amount of the added nitrifiers due to predation would be possible. Several companies that can supply nitrifiers were contacted and none had experience or knew of applications similar to that considered here. At this time, the option of growing nitrifiers in the process appears to be more feasible than adding nitrifiers grown off-site.

Existing Wastewater Treatment Plant

The existing wastewater treatment plant includes flow equalization, screening, integrated fixed film activated sludge (IFAS) biological treatment, filtration, and chlorine disinfection. The biological treatment system is provided in two circular steel package plants that were originally designed as activated sludge systems without provisions for ammonia removal (nitrification) or nitrate removal (denitrification). During 2002 through 2006, both package plants were upgraded from activated sludge to IFAS by adding webbing material supported on stainless steel frames in the reactor basins to support attached biological growth in addition to the suspended growth already in the basins. Also, the reactor basins were subdivided into anoxic and aerobic compartments to provide an MLE configuration for nitrification and denitrification. The IFAS system was designed and provided by Brentwood Industries and is called the AccuWeb system.

The AccuWeb system was designed to meet monthly average effluent ammonia-n and nitrate-n concentrations of 5 and 10 mg/L, respectively. The first AccuWeb installation in a portion of Plant 2 (one of the steel package plants), constructed in 2002, was a demonstration project with a design capacity of 144,000 gpd. DSPUD proceeded with the subsequent installations to complete the retrofits of Plants 1 and 2 in 2005 and 2006, however, a firm capacity for these improvements has not been established.

As part of the plant upgrade to the AccuWeb system, chemical feed facilities were added to feed ammonia during low load periods to grow enough microorganisms to handle high ammonia loads before the high loads occurred. Additionally, a chemical feed system for alkalinity was provided, since the nitrification process consumes alkalinity and could produce unacceptably low pH values and inhibit proper treatment without the alkalinity addition.

It is believed that the existing plant was not able to meet the ammonia and nitrate requirements in the 2002 NPDES permit for some or all of the following possible reasons:

- Inadequate reactor volume and/or biological growth media surface area in the aeration basin for complete ammonia removal.
- Lack of automated controls and optimized strategies for ammonia addition, resulting in inadequate buildup of nitrifying bacteria populations in advance of peak load events.
- Inadequate anoxic reactor volume and/or biological growth media surface area for denitrification.
- Inadequate food supply to the anoxic zone for the amount of nitrate to be removed.
- Inadequate mixed liquor recycle flows.

Treatment Improvements to Meet Existing NPDES Permit Requirements

Options for modifying the existing wastewater treatment plant to meet existing NPDES permit requirements are considered in this section. The main focus is on meeting ammonia and nitrate limits, although all permit requirements are taken into consideration. Two general types of processes can be considered for meeting the ammonia and nitrate limits: 1) biological and 2) physical/chemical. Combined biological and physical/chemical systems are also considered.

Biological Treatment for Ammonia and Nitrate

Biological treatment can be provided using suspended growth (activated sludge), attached growth (bacteria growing on support media), and/or combinations of suspended and attached growth, such as occurs with the existing integrated fixed film activated sludge (IFAS) system. Many modifications of each type of system are possible. In the following paragraphs, several systems that are judged to be most applicable for application at DSPUD are considered, including:

- Upgrade the existing two-stage IFAS system.
- Upgrade the existing IFAS system, including conversion to a four-stage reactor configuration.
- Upgrade the existing two-stage IFAS system and add denitrification filters.
- Convert to a different IFAS system (two-stage with and without denitrification filters or four-stage).
- Convert to a submerged attached growth process.
- Build a new four-stage membrane bioreactor (MBR).

All of the systems listed above can have large populations of bacteria in relatively small reactor volumes, resulting in a small footprint. This is highly beneficial at DSPUD, due to limited site space and difficult topography. Also, a small footprint is beneficial if covering the basins to conserve heat is to be considered.

For all systems, ammonia feed and control systems would be included to build up and maintain the nitrifier population during low-load periods as previously discussed.

Upgrade the Existing Two-Stage IFAS System: Based on information provided by Brentwood Industries, manufacturer of the existing AccuWeb system, the existing reactors should be able to provide full nitrification (essentially complete removal of ammonia) for influent TKN loads up to about 230 lb/d. Since the required design capacities for influent TKN based on existing flows and loads are 156 and 234 lb/d for peak month and peak week conditions, respectively (from Technical Memorandum No. 1, Draft, May 12, 2008), it would seem that the existing aerobic volumes and media surface areas should be marginally adequate. However, in an analysis of actual plant performance in the winter of 2007/2008, full nitrification was never achieved when influent TKN loads exceeded 125 lb/d, and was not achieved reliably at lower loads. The reliable nitrification capacity that would be possible with new automated controls and process optimization is unknown. To obtain reliable compliance with the new ammonia-n limit of 2.1 mg/L, additional aerobic volume may be required for existing flows and loads; however, further analysis in cooperation with Brentwood Industries is warranted.

The denitrification capacity of the existing system is even more questionable than the nitrification capacity. In the fall of 2007, while relatively low effluent ammonia concentrations were being achieved, effluent nitrate-n concentrations were frequently in the 20 to 35 mg/L range. The degree to which these effluent nitrate-n concentrations could have been reduced by adding supplemental food (methanol or other) is not known. In the winter of 2007/2008, poor nitrification performance made it impossible to assess denitrification performance. Additional anoxic volume may be required for existing flows and loads; however, further analysis in cooperation with Brentwood Industries is warranted. To attain the high degree of denitrification required to meet permit nitrate limits, particularly with supplemental ammonia addition, high mixed liquor recycle rates and addition of methanol or an alternative carbon source will certainly be required.

Because this alternative may require only limited modifications to the existing treatment structures and does not include any new processes (like denitrification filters), this alternative has the potential of being the least-cost alternative.

Unfortunately, only limited data are available on the biological treatment capacity of the AccuWeb system. Including DSPUD, there are only three full-scale wastewater treatment plants in existence using the AccuWeb media. The other two are in Connecticut and Florida, and treatment issues and requirements are quite different from those at DSPUD. Since the DSPUD installation, Brentwood has switched from using the webs to using structured sheet media. Problems with red worms eating the biomass needed for treatment have been experienced with the webs. The structured sheet media results in a thinner and denser biomass that does not support the growth of red worms. Brentwood reportedly has developed good treatment performance models for the structured sheets, but not for the webs, which Brentwood no longer supplies.

The uncertainties and potential red worm problem associated with the AccuWeb media make assessment of a plant upgrade based on continued use of this system difficult. Based on preliminary discussions, Brentwood is willing to help with an assessment, but would not be able to provide a process performance warranty for an upgraded system.

Upgrade the Existing IFAS System, Four-Stage: This option has the same uncertainties and issues associated with the AccuWeb media as discussed above. Under this option, the existing aerobic reactors would be retained for nitrification in the second stage of a four-stage reactor system. As discussed previously for the two-stage alternative, additional aerobic volume may be required, depending on evaluations to be conducted in cooperation with Brentwood Industries. Similarly, it may be possible to retain the existing anoxic basins as the first stage of the four-stage system. However, depending on final volume requirements for all of the anoxic and aerobic zones, there are many possible ways to incorporate the existing reactor and clarifier volumes, together with new structures, into a four-stage system. The most cost-effective configuration would have to be determined.

The four-stage system should provide more reliable denitrification performance, using less methanol (or alternative carbon source) than a two-stage system. Mixed liquor recirculation rates from the first aerobic zone to the first anoxic zone could be tailored to use up the readily biodegradable substrate in the raw sewage, with little or no methanol addition and minimized dissolved oxygen interference. This would also minimize the size requirements for the first anoxic zone. Since the nitrate concentration at the downstream end of the second anoxic zone would be essentially the same as the effluent nitrate concentration, simple feedback controls based on the nitrate concentration could be used to supply the correct amount of methanol to reliably meet the effluent nitrate limit (controls for methanol feed in a two stage system would be more complicated and less precise).

In addition to modifying and/or adding reactor and/or clarifier structures, upgraded ammonia feed and control systems and new methanol (or alternative carbon source) storage and feed systems would be needed under this alternative.

Upgrade the Existing IFAS System, Two-Stage, Add Denitrification Filters: This option has the same uncertainties and issues associated with the AccuWeb media as the previous two alternatives. The main benefit of using a two-stage system with subsequent denitrification filters is that it has the potential of requiring the least modifications to the existing reactor basins and clarifiers. Compared to the two-stage option without denitrification filters, much lower mixed liquor recycle rates and smaller anoxic reactor basins would be required. Methanol usage and the reliability of the overall system in meeting ammonia and nitrate limits should be comparable to that of a four-stage system.

As discussed for the two-stage option without denitrification filters, additional aerobic volume (compared to the existing aerobic volume) may be required to obtain reliable nitrification with existing flows and loads. Although anoxic volume requirements would be minimized under this alternative, the adequacy of existing anoxic volumes and possible need for increased volumes would have to be investigated. Additionally, the following improvements would be required for existing flows and loads:

1. Build new denitrification filters downstream from the existing secondary clarifiers and upstream from the existing effluent granular media filters.
2. Upgrade existing ammonia feed and control system.
3. Build new facilities for storage and feed of methanol or alternative food.

Many different configurations of denitrification filters are available from various manufacturers. In all systems, the bacteria that remove nitrate are grown attached to the filter media and methanol (or an alternate substrate) is fed as the food to support denitrification. Periodic backwashing is required to scour away excess biological growth. The denitrification filters would be located in a new building to protect the equipment, provide access for operation and maintenance and to conserve heat.

Convert to a Different IFAS System: Several different IFAS systems can be considered. One option would be to use the new structured sheet media currently produced by Brentwood Industries. The structured sheet media consists of corrugated plastic sheets, layered together in blocks. Like the existing webs, the structured sheet media would be fixed in certain positions within the reactor basins. According the Brentwood, treatment results should be more predictable and reliable with the structured sheets. Since this is a new product, however, there are no reference installations with significant operating history.

Another option would be to use loose media retained in reactors with sieves or other suitable barriers. There are several manufactured systems of this type with many installations worldwide. The media are typically small plastic shapes that provide large amounts of surface area for biological growth. The new media are simply dumped into the reactor basin, with various degrees of fill being possible. Under the mixing and/or aeration conditions in the reactor basins, the media are suspended and move about freely. As the treated wastewater flows out of the reactor basins to the clarifiers, the media with attached growth are retained in the reactor basins. Systems can be operated with or without returning settled solids from the clarifiers to the reactor basins. If settled solids are returned, a substantial population of suspended bacteria can be developed in the reactor, so that treatment is accomplished both by attached and suspended growth. This is then an IFAS system. If solids are not returned from the clarifier to the reactor and essentially all treatment is accomplished by attached growth, the system is a moving bed bioreactor (MBBR). The IFAS configuration would be preferred at DSPUD for two primary reasons: (1) more treatment capacity could be provided in a smaller space, and (2) by including mixed liquor in the process, fine dispersed solids can be accumulated in biological flocs and removed in the secondary clarifier, leading to better reliability in meeting the effluent turbidity limit.

Two-stage systems with and without denitrification filters and four-stage systems can be investigated in accordance with previous discussions. These types of IFAS systems are designed based mostly on empirical data and models developed by the respective manufacturers, which would provide process performance warranties.

Convert to a Submerged Attached Growth System: The denitrification filter previously described is an example of a submerged attached growth system. Several manufacturers have developed submerged attached growth systems that can be used to provide complete biological wastewater treatment, including BOD removal, nitrification, and denitrification. Like the denitrification filter previously mentioned, attached biological growth occurs within a media bed that also provides for suspended solids removal. No secondary clarifier is needed. Excess biological solids are accumulated in the bed and removed periodically by backwashing. Both upflow and downflow systems are used. Media used in these systems include specifically sized fired clay and polystyrene beads. Proprietary process names include Biocarbone, Biofor, and Biostyr. There are hundreds of these systems in existence throughout the world. The TTSA wastewater treatment plant in Truckee converted to the Biostyr process in their recent plant upgrade and expansion. The plant is reportedly able to produce an effluent with typical ammonia-n and nitrate-n concentrations of around 0.5 and 1.5 mg/L, respectively.

For BOD removal, nitrification, and denitrification, three submerged attached growth systems would typically be used in series – one for each of these major functions. Because these are basically biologically active filter systems, they are limited by hydraulic loading rates per unit area. It is impractical to have the large recycle flows that would be associated with denitrification in an anoxic zone upstream from an aerobic zone such as occurs in the MLE configuration previously described. Therefore, it is not practical to use the influent wastewater as a food source for denitrification. Instead, all denitrification is accomplished using methanol or other suitable substrate after BOD removal and nitrification. Accordingly, the methanol usage for this system would be substantially greater than in the two-stage or four-stage IFAS systems previously discussed. The requirement for chemical alkalinity addition would also be much higher.

Submerged attached growth systems would have a very small footprint. These systems have been cost-effective mostly in larger plants, but can be considered for plants as small as DSPUD. These types of systems are proprietary treatment systems, the designs of which are based on empirical data and models developed by the respective manufacturers, which will provide process performance warranties.

The submerged attached growth systems alone would not be able to meet a 2 NTU effluent turbidity requirement. Therefore, the existing granular media filtration system at DSPUD would continue to be used after a new submerged attached growth system. It may be necessary to provide improved coagulation and flocculation ahead of these filters.

Build a New Four-Stage MBR: An MBR is a suspended growth (activated sludge) biological treatment system. In an MBR, clarifiers and effluent filters used in a conventional system are replaced with membrane filters submerged in the biological treatment system mixed liquor. Wastewater effluent is pulled through the membranes by pumping, leaving the solids in the reactor basins. The membranes would provide an absolute barrier to mixed liquor solids. The MBR effluent would typically have a turbidity under 0.2 NTU. By contrast, the existing biological treatment and granular media filtration system at DSPUD is designed to have an effluent turbidity under 2 NTU.

MBR systems have several distinct advantages, when compared to activated sludge and/or IFAS systems:

1. The need for clarifiers and granular media filters is eliminated as mentioned above.
2. Since solids settling in a clarifier is no longer required, mixed liquor solids can be increased to about 8,000 to 10,000 mg/L, compared to 3,000 to 4,000 mg/L in activated sludge and IFAS systems. This means that reactor basins can be 1/3 to 1/2 the size required for conventional activated sludge.
3. A much higher quality effluent is produced with high reliability.
4. Because membrane filters remove many colloidal solids that cannot be removed by clarification, there is frequently a benefit in further removals of heavy metals or other constituents of concern that have a particulate or colloidal fraction.
5. The MBR effluent is much easier to disinfect, leading to reliable effluent coliform compliance with lower chlorine doses. Additionally, if a switch is made to ultraviolet (UV) light disinfection, the required size of the UV facilities is substantially reduced as compared to systems without membrane filtration.
6. In general, MBRs are state-of-the-art treatment systems that produce the highest quality effluent, assuring more reliable compliance with current treatment standards and a better chance of meeting new and/or more stringent standards in the future.

The MBR system would include two concrete reactor basin trains, with each train including a pre-anoxic zone, an aeration zone, and a post-anoxic zone, each of which could be further staged. These would be followed by membrane basins that could be either concrete or prefabricated steel packaged units with the membranes installed. The membrane basins would include air scouring to keep the membranes clean and would therefore act as additional aerobic reactor volume. All reactor and membrane basins would be covered or inside a building. Permeate pumps for pulling the effluent through the membranes, blowers for air scour and for process aeration, mixed liquor recirculation pumps, chemical feed systems, and other ancillary facilities also would be inside a building.

Physical/Chemical Treatment for Ammonia or Nitrate

Ammonia and nitrate can be removed by various physical/chemical processes, including the following:

- Ammonia:
  - Air Stripping
  - Ion Exchange
  - Breakpoint Chlorination
  - Reverse Osmosis
- Nitrate:
  - Reverse Osmosis

Air stripping is considered impractical at DSPUD due to freezing of the stripping towers. Reverse osmosis would be prohibitively expensive and would produce a residual brine solution that would be extremely difficult and expensive to dispose of or eliminate. Therefore, these options are not considered further.

In general, physical/chemical treatment systems for nitrogen removal have been used only at a few municipal wastewater treatment plants throughout the country, dating back to the 1970s and 1980s. Most of these systems have since been abandoned in favor of biological treatment systems. The following are excerpts from the EPA Nitrogen Control Manual, dated September 1993:

> *"The physical/chemical processes for nitrogen control are at the opposite end of the spectrum from lower technology approaches. Although receiving only limited application, there is enough knowledge to determine that they have limited or no potential for most municipal applications."*

> *"The physical/chemical processes are briefly discussed in Section 2.5, more in the interest of completeness and to point out the problems of the past in order to avoid their repetition rather than to recommend their use."*

> *"Several physical/chemical nitrogen control treatment processes have been advanced and tried in municipal wastewater treatment applications. Only two remain in routine service. Physical/chemical treatment, except in highly specialized situations, is the process of last resort, especially at small plants."*

Ion Exchange for Ammonia Removal: Ammonia can be removed from filtered wastewater effluent by passing it through a packed bed ion exchange column (similar to a granular media filter) containing natural clinoptilolite media. In the clinoptilolite, the ammonium ion is removed by exchanging it for sodium ions, which are released into the wastewater. Other positive cations, most notably calcium, will compete with ammonium for the available exchange sites, reducing the capacity of the media to remove ammonia. When the clinoptilolite media has removed a certain amount of ammonium (and competing ions), the media is first backwashed and then regenerated by applying high concentration sodium chloride solutions in a stepwise process. The regenerant solutions are stored in different tanks, depending on previous uses and the accumulated ammonia concentrations. The regenerant solution with the highest accumulated ammonia concentration is circulated through the ion exchange column first, followed by regenerant that has been used less and has less accumulated ammonia. The final regenerant solution to be used during a regeneration cycle consists mostly of regenerant that has just been stripped of accumulated ammonia. During the regeneration process, the exchange sites are again occupied by sodium ions and the ammonium and competing cations are released into the regenerant solutions. After a regeneration cycle, the regenerant that was used first and contains the highest amount of accumulated ammonia is subjected to a stripping process to remove most of the accumulated ammonia. Caustic soda or lime is added to the spent regenerant to raise the pH and convert the ammonium ion into dissolved ammonia gas that can be removed by air stripping. However, the high pH also causes precipitation of magnesium hydroxide and calcium carbonate that must be removed by clarification before air stripping is accomplished. Once air stripping of ammonia is accomplished, the stripped regenerant is stored for use as the final step of the next regeneration cycle. The exhaust gas from the stripper is passed through an adsorption tower with sulfuric acid to take up the ammonia and form ammonium sulfate that can be sold as a fertilizer.

As described above, ammonia removal by ion exchange is a complex and mechanically intensive process. It has been used only in a couple full scale applications in the country. This was the method of nitrogen removal at TTSA for about 30 years, until the system was recently replaced with a submerged attached growth biological treatment system. The other full-scale application was at the Upper Occoquan Sewage Authority in Virginia. That facility has switched to suspended growth biological nitrification and denitrification. They have the ability to use breakpoint chlorination as a final polishing step for ammonia control.

According to Richard Svetich, a scientist who was responsible for running the ion exchange system at TTSA, the system was originally designed with the intent of producing an effluent total nitrogen level of 2 mg/L, but was unable to meet that objective. The TTSA ion exchange system was typically operated to produce an effluent ammonia-n concentration of about 5 to 6 mg/L. This was determined to be acceptable because further nitrogen removal from the effluent was found to occur by natural means after it was discharged underground and flowed through the soil to the Truckee River. According to Mr. Svetich, attaining an effluent concentration of 2 mg/L of ammonia-n in the ion exchange system would require a very conservative design with lightly loaded ion exchange columns, frequent regeneration and with very large chemical usage and expenses associated with regeneration and regenerant recovery. Pilot testing would be required to develop design criteria for use at DSPUD.

One possible option would be to design the ion exchange system to remove most of the ammonia (perhaps to around 5 mg/L) and then use breakpoint chlorination to remove the remainder of the ammonia down to the effluent limit. Breakpoint chlorination is discussed in the next sub-section of this document.

A significant issue associated with ammonia removal by ion exchange is that the biological process that is used for BOD removal should be operated to avoid nitrification. Otherwise ammonia would be converted to nitrate, which would not be removed in the ammonia ion exchange system and could cause violation of the effluent nitrate limit. Although obtaining complete nitrification at DSPUD is problematical as discussed elsewhere in this paper, operating to prevent nitrification altogether also would be difficult and may jeopardize other treatment objectives, particularly the final effluent turbidity limit of 2 NTU. To prevent nitrification altogether, the plant would have to be operated at a low mean cell residence time (MCRT of a few days, depending on temperature). Operation at a low MCRT requires more careful operator attention, produces more sludge, and would be less reliable in terms of meeting effluent BOD, TSS, and turbidity limits.

An option that might be most applicable for the conditions at DSPUD would be to use an ion exchange system after a nitrifying and denitrifying biological treatment process. In this way, the biological treatment system could remove as much ammonia as possible, without the need for supplemental ammonia addition to build up the nitrifier population. It is likely that the ion exchange system would have to be followed by breakpoint chlorination for further polishing of the effluent ammonia. However, considering the large difference between low fall loads and high winter loads at DSPUD, the amount of ammonia escaping the biological process upon the onset of winter loads would be such that the ion exchange system in this case would not be substantially different than if no biological ammonia removal was provided.

Because of the complexity, anticipated high costs, and other issues discussed above, it is considered unlikely that ion exchange would be a good option for DSPUD.

Breakpoint Chlorination for Ammonia Removal: Ammonia can be removed by adding chlorine in the form of chlorine gas or sodium hypochlorite in the process of breakpoint chlorination. As the chlorine is added, it combines with the ammonia first to form chloramines. The chloramines are measured as combined chlorine residual. Up to a weight ratio of about 5 parts of chlorine per part of ammonia nitrogen, the measured chlorine residual would increase as the chlorine is added. As more chlorine is added, the chloramines would be broken down, resulting in decreasing chlorine residual with increased chlorine dose, until a minimum residual is reached at a theoretical ratio of 7.6 parts of chlorine per part of ammonia nitrogen. This point of minimum chlorine residual is the breakpoint. Further addition of chlorine past the breakpoint would result in increasing chlorine residuals. The increasing residuals would be in the form of free chlorine (not chloramines). As the chloramines are being eliminated approaching the breakpoint, the chlorine is converted to the chloride ion and the nitrogen from the ammonia is converted into nitrogen gas, as well as some nitrous oxide and nitrogen trichloride.

In actual practice, it has been found that the amount of chlorine required to reach the breakpoint is greater than the theoretical requirement, perhaps around 10 parts of chlorine per part of ammonia nitrogen. Thus, to remove 30 mg/L of ammonia-n, around 300 mg/L of chlorine would be required.

When chlorine gas is used for breakpoint chlorination, there is a net consumption of 14.3 mg/L of alkalinity per mg/L of ammonia-n removed. This is double the consumption of alkalinity by biological nitrification. Therefore, lime or caustic soda would typically be added to offset the alkalinity loss in breakpoint chlorination. If sodium hypochlorite is used, alkalinity consumption is not a problem.

Because of the chlorine or sodium hypochlorite added, and because of the need to add alkalinity with chlorine, breakpoint chlorination results in a substantial increase in effluent salinity. When sodium hypochlorite is used, the total dissolved solids (TDS) added is 7.1 mg/L per mg/L of ammonia-n removed. When chlorine is used and alkalinity is replaced using caustic soda, the TDS added is 14.8 mg/L per mg/L of ammonia-n removed.

If a breakpoint chlorination process is used for ammonia removal, additional chlorine would be added beyond that required for ammonia removal to obtain a chlorine residual required for disinfection. As mentioned above, the residual would be in the form of free chlorine (not chloramines). The use of free chlorine for disinfection would also occur if the ammonia was removed biologically, unless some ammonia were added back in prior to disinfection. With free chlorine disinfection, there is a substantial risk of forming disinfection byproducts in amounts that would be above allowable limits. Although the chlorine added to reach the breakpoint does not result in free chlorine residual, the large amounts of chlorine involved in breakpoint chlorination would certainly cause concern regarding disinfection byproducts.

If chlorine gas were used for breakpoint chlorination, concerns regarding chlorine safety and public risk would be raised. At the minimum, chlorine containment and scrubbing systems would be required at the plant to mitigate the potential consequences of a leak within the plant. However, that would not address concerns regarding the safety issues involved in transporting chlorine gas to the plant and unloading it at the plant. Because of the safety concerns associated with chlorine gas, many communities discontinued its use in favor of using sodium hypochlorite. Now, with disinfection byproducts concerns, even the use of sodium hypochlorite is being discontinued in many plants in favor of using ultraviolet (UV) disinfection.

It is noted that chlorine gas is currently used for disinfection at DSPUD. However, only relatively small quantities are used and the chlorine is provided in 100 lb cylinders. Even with the 100 lb cylinders, the Uniform Fire Code requires containment and scrubbing systems, as well as other safety features that currently do not exist at the plant. Such systems should be provided with any plant upgrade. However, if breakpoint chlorination with chlorine gas is to be used, the plant will need to switch to ton cylinders of chlorine. In that case, the safety concerns and need for mitigation are greatly increased.

Breakpoint chlorination potentially could be used as the primary ammonia removal system or as a supplemental system to be used after biological treatment or ion exchange for ammonia removal. However, because of the large chlorine doses involved and related issues as discussed above, use as the primary ammonia removal method is not recommended.

The most likely application for breakpoint chlorination at DSPUD would be as a supplement to biological ammonia removal, particularly if that could eliminate the need for supplemental ammonia addition to build up the nitrifier population during the fall and during low-load periods in the winter. This would, in turn, eliminate a large amount of potential methanol usage and perhaps eliminate the need for a four-stage biological process. Unfortunately, however, the difference between the low load conditions of fall and the high-load conditions of winter are so extreme that this is not likely. As documented in the letter report from Jeff Hauser of

ECO:LOGIC Engineering to Tom Skjelstad of DSPUD, dated January 15, 2009, the influent TKN load during the weeks and months preceding the Christmas Holiday period are estimated to be only about 20 to 40 lb/d, compared to over 150 or 200 lb/d during peak winter periods. Therefore, without supplemental ammonia addition (and the associated additional methanol or other food addition), it would be expected that 70 percent or more of the TKN coming in during the initial peak loads could end up as ammonia in the effluent. With peak load influent TKN concentrations expected to be over 60 mg/L, effluent ammonia-n concentrations over 40 mg/L would be expected. Therefore, a chlorine dose of over 400 mg/L could be required for breakpoint chlorination. At a flow rate of 0.5 Mgal/d, that would require about 1700 lb/d of chlorine.

Based on the discussion above, breakpoint chlorination cannot be expected to eliminate the need for supplemental ammonia addition or a four-stage biological treatment process. Because of this and all of the concerns associated with breakpoint chlorination, it is suggested that means other than breakpoint chlorination should be planned to meet the 2 mg/L ammonia-n limit. Breakpoint chlorination should be considered only as a potential final polishing step in the event of minor excursions above the 2 mg/L ammonia-n limit.

The recommendation to consider breakpoint chlorination only as a final polishing option is in concert with the EPA Nitrogen Control Manual, dated September 1993. The following are excerpts from that manual:

> *"The only known operating facility where breakpoint chlorination is the principal nitrogen control strategy is at Sugarbush, Vermont. .. The utilities director's recommendation for others considering full nitrogen control by breakpoint chlorination can be summarized in one word – 'don't'."*
>
> *"It is recommended that breakpoint chlorination be routinely considered only for polishing applications, such as was used at the previously described North Tahoe Truckee Plant, where a low total or unoxidized nitrogen residual is mandatory."*

Upgraded Treatment to Prevent Biostimulation

If DSPUD were to continue discharging to the South Yuba River during times when nuisance algae growth could occur, it would have to remove biostimulatory substances to levels that would not cause or contribute to nuisance growths. At the present time, it is uncertain which substances would have to be removed and to what levels. It is believed that nitrogen and phosphorus, as primary nutrients for algae, would have to be removed to very low levels. Iron and other micronutrients might also be considered.

In the Treatment and Disposal Facilities Plan prepared for DSPUD in June 1984, the option of discharging to the river during times when algae growth could occur was investigated and discussed with the Central Valley Regional Water Quality Control Board. At that time, it was planned that such a discharge would have to meet background concentrations (concentrations in natural runoff without pollution from human activity) of total nitrogen and total phosphorus, which were estimated to be 0.3 and 0.02 mg/L, respectively. In establishing numerical discharge limits for storm water runoff in the Lake Tahoe basin, the Lahontan Regional Water Quality Control Board took a similar approach and established total nitrogen and total phosphorus limits of 0.5 and 0.1 mg/L, respectively. Although it may be feasible at substantial cost to meet these types of phosphorous limits, it is considered impractical to meet such low total nitrogen limits without going to such extreme treatment as reverse osmosis, which would be cost prohibitive.

Depending on the amount of dilution present below the DSPUD discharge, allowable effluent nutrient concentrations may be somewhat higher than the background levels mentioned above, but probably still at relatively infeasible levels. Because of this and because the studies that would be required to establish allowable nutrient concentrations would be expensive and time consuming, such studies are not recommended. Rather, the biostimulation study to be conducted by DSPUD should focus on defining times and conditions during which algae would not grow in nuisance amounts (such as cold winter and high-flow spring conditions), despite the presence of ample nutrients.

Based on the discussion above, it is believed that continued discharge to the river during times when algae can grow in nuisance amounts will be impractical.

Lower Levels of Treatment for Land Disposal

Two of the disposal options considered previously could potentially result in treatment requirements less stringent than those for meeting the numerical effluent limits contained in the existing NPDES permit. These are briefly discussed below.

Treatment for Subsurface Discharge

For subsurface disposal, treatment requirements are uncertain, due to questions regarding the ultimate fate of the effluent, possible impacts on surface water courses and groundwater degradation. It is possible, however, that treatment requirements could be somewhat less stringent than indicated by the numerical effluent limits contained in the existing NPDES permit. It is possible also that some natural treatment during underground flow could be attained.

Treatment for Storage and Irrigation Disposal

This disposal option is likely to result in the least stringent treatment requirements. For example, the existing discharge requirements for irrigation at the Soda Springs Ski Area allow average BOD and TSS concentrations of 30 mg/L (compared to 10 mg/L for river discharge) and total coliform organisms of 23 MPN/100 mL (compared to 2.2 for river discharge). There are no limits on ammonia, nitrate, metals, disinfection byproducts, or other parameters that are of concern for river discharge. In general, it is expected that a relatively simple secondary treatment plant would be adequate for this disposal option.

Summary of Treatment Options

The treatment options discussed above are summarized in Table 3. Recommendations on which options should be considered further are included in Section 5.

Table 3
Summary of Treatment Options

Option	Pros	Cons	Comments
Upgrade the Existing IFAS System, Two Stage	❑ Potentially the lowest cost alternative. ❑ Continue to use the existing AccuWeb modules. ❑ Fewer modifications to existing treatment system required, as compared to the four-stage option.	❑ Uncertainty on performance of AccuWeb. ❑ Potential red worm problems. ❑ Manufacturer has discontinued AccuWeb media in favor of structured sheet media. ❑ Lack of other AccuWeb installations to assess performance. ❑ Very high mixed liquor recirculation rates required. ❑ Higher methanol usage and more difficult control compared to the next two options. ❑ Less reliable than the next two options. ❑ Larger anoxic zone required, as compared to the option with denitrification filters.	❑ Cooperative effort with Brentwood Industries required to assess AccuWeb performance and improvement requirements.
Upgrade the Existing IFAS System, Four-Stage	❑ Continue to use the existing AccuWeb modules. ❑ Lower methanol usage and easier control compare to the two-stage option without denitrification filters. ❑ Higher reliability than two-stage without denitrification filters.	❑ Uncertainty on performance of AccuWeb. ❑ Potential red worm problems. ❑ Manufacturer has discontinued AccuWeb media in favor of structured sheet media. ❑ Lack of other AccuWeb installations to assess performance. ❑ Reconfiguration of existing basins and new basins required.	❑ Cooperative effort with Brentwood Industries required to assess AccuWeb performance and improvement requirements.
Upgrade the Existing IFAS System, Two-Stage, Add Denitrification Filters	❑ Continue to use the existing AccuWeb modules ❑ Fewest modifications to existing treatment system required as compared to both options above. ❑ Lower methanol usage and easier control compare to the two-stage option without denitrification filters. ❑ Higher reliability than two-stage without denitrification filters.	❑ Uncertainty on performance of AccuWeb. ❑ Potential red worm problems. ❑ Manufacturer has discontinued AccuWeb media in favor of structured sheet media. ❑ Lack of other AccuWeb installations to assess performance. ❑ New denitrification filter system must be added.	❑ Cooperative effort with Brentwood Industries required to assess AccuWeb performance and improvement requirements.

Option	Pros	Cons	Comments
Convert to a Different IFAS System	❑ IFAS systems with small plastic cylindrical biofilm carriers suspended in the reactor basins are well demonstrated with hundreds of installations worldwide. ❑ Better understanding of performance characteristics as compared to AccuWeb. ❑ Backed by large international wastewater process manufacturers. ❑ No red worm problems.	❑ No further use of the existing AccuWeb modules. ❑ High cost of conversion.	❑ Two-stage systems with and without denitrification filters and four-stage systems can be considered.
Convert to a Submerged Attached Growth System	❑ Hundreds of successful installations worldwide (including TTSA). ❑ Probable smallest footprint treatment system. ❑ Backed by large international wastewater process manufacturers.	❑ Completely new treatment plant structures required. Existing basins would be converted to alternative uses, perhaps equalization or sludge handling. ❑ No pre-anoxic treatment, so all nitrate must be removed with imported food (methanol or other).	❑ Most existing plants of this type are much larger than DSPUD. Cost effectiveness at small size is questionable.
MBR (Four-Stage)	❑ Hundreds of successful installations worldwide ❑ Membranes provide absolute barrier to solids and lowest turbidity effluent of any biological treatment system. ❑ Because many colloidal solids are removed, MBR may help to meet requirements for some metals and priority pollutants with a particulate component. ❑ No need for clarifiers or filters. ❑ High mixed liquor solids allow small footprint. ❑ Easiest effluent to disinfect.	❑ Completely new treatment plant structures required. Existing basins would be converted to alternative uses, perhaps equalization or sludge handling.	❑ Use of MBRs in recent years has grown exponentially. MBR would likely be the technology of choice for a new plant in situations similar to DSPUD.
Biological Treatment for BOD Removal Followed by Ion Exchange for Ammonia Removal	❑ Ion exchange is not a biological process, so no need to buildup nitrifier population in advance of peak loads. ❑ Not impaired by low temperature.	❑ Mechanically complex. ❑ May not be able to attain ammonia limit unless followed by breakpoint chlorination. ❑ Pilot testing required to establish design criteria. ❑ Must operate biological process to avoid nitrification, which is not desirable. Alternatively, must provide for nitrate removal.	❑ Only two full-scale municipal wastewater treatment plants known to have used ion exchange for ammonia removal (including TTSA). Both plants have abandoned these systems in favor of biological treatment for ammonia.

Option	Pros	Cons	Comments
Biological Nitrification and Denitrification Supplemented by Ion Exchange and Breakpoint Chlorination for Ammonia Removal	❑ No need to build up nitrifier population in advance of peak loads. ❑ Physical/chemical processes not impaired by low temperature.	❑ Same as above. ❑ This option would not allow substantial reduction in ion exchange system compared to above.	❑ Only two full-scale municipal wastewater treatment plants known to have used ion exchange for ammonia removal (including TTSA). Both plants have abandoned these systems in favor of biological treatment for ammonia.
Breakpoint Chlorination for Ammonia Removal	❑ Breakpoint chlorination is not a biological process, so no need to buildup nitrifier population in advance of peak loads. ❑ Not impaired by low temperature.	❑ Huge chlorine doses required (several hundred mg/L). ❑ Disinfection byproducts. ❑ Safety issues associated with gaseous chlorine transport and use (unless switch to sodium hypochlorite). ❑ Adds substantial salinity. ❑ Must operate biological process to avoid nitrification, which is not desirable. Alternatively, must provide for nitrate removal.	❑ EPA Manual on Nitrogen Control indicates this technology should be considered only for polishing small amounts of ammonia.
Treatment to Prevent Biostimulation	❑ Avoids need for seasonal storage	❑ Probably not feasible to meet nutrient limits needed to avoid biostimulation.	❑ Because of the anticipated infeasibility of this option and because it would be expensive and time consuming to determine appropriate nutrient concentrations to prevent biostimulation, it is recommended that determination of these concentrations should not be part of the DSPUD biostimulation study.

5. OVERALL WASTEWATER MANAGEMENT OPTIONS

In the previous sections, various disposal and treatment options are considered and evaluated on a conceptual level. In Table 4, disposal and treatment options are grouped into combined wastewater management options. For each option, a subjective rating is provided for each of four key evaluation factors: (1) anticipated costs, (2) reliability, (3) ease of implementation and (4) environmental impacts.

A three point rating system is used as follows:

- "+" indicates the option would likely be advantageous compared to other possible options based on this criterion.
- "0" indicates the option is neither favorable nor unfavorable based on this criterion. "0" can be considered an average or medium rating.
- "-" indicates the option would likely be disadvantaged compared to other possible options based on this criterion.

Anticipated costs represent the total life-cycle costs, including the initial capital cost and the ongoing operation and maintenance costs, such as labor, power and chemical costs. It must be recognized that the ratings given for cost are based on engineering judgment as to likely costs relative to other options, without the benefit of developing specific project sizes, layouts and actual cost estimates. Accordingly, there is a significant margin for error in making these assessments.

The reliability criterion reflects a preliminary assessment of the degree of certainty that the option can be designed with confidence to attain compliance with all regulatory requirements. A range of issues is lumped into the rating, including, but not limited to, such things as:

- the degree to which the technology is established, has been demonstrated successfully in other similar applications and reliable design criteria exist;
- the likelihood of operational problems or performance variability leading to occasional excursions beyond permitted limits; and
- the possibility of undesired side effects, such as disinfection byproducts or salinity issues.

Ease of implementation reflects the anticipated degree to which any legal, administrative, institutional, regulatory, land or right-of-way acquisition, or uncertain technical issues could delay the planning, design, and/or construction of the project.

Environmental impacts reflect the degree to which the option would result in the need to disrupt currently natural areas for the construction of wastewater facilities as well as any ongoing environmental impacts associated with the continued functioning of the option.

Table 4
Overall Wastewater Management Options

Disposal Option	Treatment Option	Cost	Reliability	Ease of Implementation	Environmental Impact	Further Consideration
Subsurface	Unknown	0	0	-	0	No
Wet Season Storage, Dry Season Irrigation	Secondary	-	+	-	-	No
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Upgrade Existing IFAS 2-Stage	+	-	+	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Upgrade Existing IFAS 4-Stage	+	-	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Upgrade Existing IFAS 2-Stage, Denitrification Filter	+	-	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	New IFAS 4-Stage	0	+	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	New IFAS 2-Stage, Denitrification Filter	0	+	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Submerged Attached Growth	0	+	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	MBR 4-Stage	0	+	0	-	Yes
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Non-Nitrifying Activated Sludge, Ion Exchange for Ammonia	0	-	-	-	No
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Non-Nitrifying Activated Sludge, Breakpoint Chlorination for Ammonia	+	-	0	-	No
Wet Season Discharge to SYR, Seasonal Storage, Dry Season Irrigation	Upgrade Existing IFAS 2-Stage, Ion Exchange and Breakpoint Chlorination for Supplemental Ammonia Removal, Denitrification Filter for Supplemental Nitrate Removal	-	-	-	-	No
Wet Season Discharge to SYR, Dry Season Irrigation, No Seasonal Storage	Undetermined Enhanced Nutrient Removal System	-	-	-	0	No
Year-Round Discharge to SYR	Undetermined Extreme Treatment	-	-	-	-	No
Export Raw Sewage to TTSA	None	0	+	-	-	No
Export Treated Effluent to TTSA	Undetermined Enhanced Nutrient Removal System	-	-	-	-	No

A final column in Table 4 is used to indicate a recommendation for further evaluation up to and including process analysis, unit sizing, and detailed life-cycle cost analysis. If during subsequent analyses, information is developed that would jeopardize the viability of an option, termination of further evaluation would be considered at that time.

6. ADDITIONAL CONSIDERATIONS

In this section, various issues that would affect many or all of the wastewater management options considered in this document are discussed, including:

- Infiltration and Inflow
- Equalization Storage
- Covering Basins to Conserve Heat
- Disinfection Alternatives
- Solids Handling
- Planning for Future Growth
- Schedule for Future Work

Infiltration and Inflow

At times during the year, infiltration and inflow (I/I) can constitute a significant portion of the total flow into the DSPUD WWTP. During the spring snowmelt, this is the primary flow component. However, flows that occur during peak occupancy periods in the winter (even without unusual I/I events) are frequently of the same general magnitude or larger than those in the spring.

I/I flows will have a significant impact on the sizing and cost of some treatment, storage, and disposal components, including influent equalization storage, filtration facilities, effluent storage, and effluent spray irrigation facilities (not a complete list). Additionally, since I/I can be much colder than wastewater from homes and businesses, the presence of I/I impacts the design temperature and sizing of biological treatment reactor basins (discussed later in this document). I/I flows also impact the ongoing operation and maintenance costs. Accordingly, it is highly important that both DSPUD and SLCWD have aggressive I/I mitigation programs. This is nothing new; both Districts have understood this and have sought to control I/I for many years. Although substantial progress has been made, more needs to be done. It is noted that some of the highest flows occurring since the year 2002 occurred in 2006 and 2007, after both Districts had made substantial I/I improvements. Although flows in 2008 and 2009 have been generally lower, this is probably due to less severe weather conditions that create I/I, rather than system improvements.

In planning and design of wastewater treatment, storage, and disposal facilities for the future, a key question is how much I/I to include in the flow projections. In general, the answer should be a conservative one. In many cases, projections of reduced I/I have not been realized. Therefore, it is suggested that, unless the specific causes of known I/I flows of the past have been identified and corrected and ample time and events have passed to prove a flow reduction, no reduction should be presumed. Sometimes, I/I flows eliminated at one location simply show up somewhere else.

The statement above does not mean DSPUD and SLCWD should accept the status quo. Just to hold the line at existing I/I amounts will require dedicated efforts from the two Districts. Furthermore, if substantive I/I reductions can be made over the years, that would have the benefit of lowering operating costs and potentially extending system capacity.

The two Districts may want to increase I/I reduction efforts and funding in advance of the design of the upcoming improvement project. However, it is doubtful that convincing results of permanent flow reductions could be realized in time to make a significant change in design based on recent historical flows.

Equalization Storage

Influent equalization storage will be considered for all wastewater treatment options. The existing plant includes an equalization storage tank with a volume of 0.2 Mgal, which, based on the design in 1985, was intended to equalize flows to over a peak 3-day weekend to 0.52 Mgal/d.

Based on the Draft Technical Memorandum No. 2, prepared for the current project in April 2009, and based on recent historical flows, the volume of 0.2 Mgal gallons would still be adequate to equalize influent flows to a maximum of about 0.5 Mgal/d, if the peak flow event that occurred from December 21, 2005 through January 2, 2006 is ignored. With that peak flow event included in the analysis, the theoretical storage requirement to equalize to 0.5 Mgal/d is increased to about 0.8 Mgal. To equalize to 0.4 Mgal/d, the volume requirements are about 0.4 and 1.8 Mgal, without and with consideration of the 2005/2006 peak flow event, respectively.

In the future design of treatment plant improvements, the most cost effective size of equalization storage will be determined. Consideration will be given to emergency peak flow handling should the equalization capacity be exhausted.

Covering Basins to Conserve Heat

As discussed previously in this document, cold winter temperatures are a particular concern for biological nitrification. For example, the net growth rate (growth minus decay) of nitrifying bacteria can about double with a temperature change from 5 °C to 10 °C, depending on the fraction of the time that the nitrifiers are under anoxic conditions (due to mixed liquor circulation through an anoxic zone). Doubling the growth rate would result in the need for about one-half the aerobic reactor volume to accomplish the same level of treatment.

Currently, temperatures in the biological reactors can get down to about 4 °C or 5 °C in the winter. Therefore, covering the basins to conserve heat may be of major benefit.

Heat is lost from wastewater treatment basins with exposed water surfaces by several methods, including: (1) net atmospheric radiation, (2) conduction and convection, and (3) evaporation. Heat is gained by: (1) solar radiation, (2) mechanical energy input due to mixing and/or aeration, and (3) the exothermic biological processes. In the coldest part of the winter, the most significant heat losses from exposed water surfaces are by atmospheric radiation and conduction and convection. The largest temperature changes due to these heat loss mechanisms occur with cold and windy conditions with lower wastewater flows.

Based on preliminary and approximate calculations, covering the basins to minimize atmospheric radiation and conduction and convection to the air above has the potential to increase the temperature in the reactor basins by 5 °C or more, depending on conditions. Therefore, covering the basins should be considered during planning and design.

Another option that could be considered to increase the temperature in the reactor basins is to generate electricity for use in the plant by using diesel driven generators and to cool the diesel engines using heat exchangers in the equalization basin. This option is currently employed at the Kirkwood Meadows Public Utility District. However, the main incentive for using on-site diesel generators at Kirkwood was the extremely high cost of power in that location.

Disinfection Alternatives

As previously noted, the current NPDES permit includes numerical limits on the chlorine disinfection byproduct dichlorobromomethane. There are other chlorine disinfection byproducts that can occur, but the reasonable potential analysis based on previous California Toxics Rule sampling indicated that only dichlorobromomethane had the reasonable potential to exceed water quality objectives. However, if the wastewater effluent was not fully nitrified to remove essentially all ammonia at the time of those previous samples, it is likely that disinfection byproduct formation was limited due to the presence of the ammonia. With ammonia present, chlorine forms chloramines and the disinfection process is referred to as chloramination, versus simply chlorination. Chloramination is known to substantially reduce disinfection byproducts compared to chlorination. It is currently unknown what level of disinfection byproducts would occur with complete nitrification and disinfection by chlorination. Certainly, there is reason to be concerned about disinfection byproducts if the nitrification system is improved and disinfection is by chlorination.

There are three possible methods by which disinfection byproducts can be mitigated:

- dilution in the receiving water
- practicing chloramination instead of chlorination
- switching to UV disinfection

As previously noted, dilution credits are not currently allowed in the NPDES permit. However, there are provisions to reopen the permit and reconsider the matter of dilution credits, if DSPUD installs a diffuser, conducts a mixing zone study, and meters the flow of the South Yuba River at the point of discharge. Obtaining dilution credits for dichlorobromomethane and any other disinfection byproducts that might occur in the future could be highly beneficial. The dilution credits would be based on long-term average flows in the South Yuba River and should be substantial. Therefore, DSPUD should pursue this option.

In a biological treatment process designed to remove ammonia, it is not practical to leave a little ammonia in the effluent for the purposes of chloramination. Instead, after removing essentially all ammonia, a little would be added back in. If the use of chlorine is to be continued or if sodium hypochlorite were to be used, adding some ammonia to mitigate disinfection byproducts should certainly be considered. At this time, it is not known whether chloramination would be fully successful in mitigating disinfection byproducts, particularly if dilution credits are not obtained.

By switching to UV disinfection, the chlorine disinfection byproducts could be eliminated. However, this would involve substantial capital and ongoing operation and maintenance costs.

As previously noted in this document, if chlorine disinfection is to be continued, the gaseous chlorine system should be upgraded to comply with Uniform Fire Code requirements. Alternatively, DSPUD could switch to using sodium hypochlorite.

Solids Handling

The previous discussions have been limited to the liquid stream treatment processes in the wastewater treatment plant. Solids handling must also be considered in any improvement or expansion project. This could include sludge digestion and mechanical dewatering facilities. The options for these improvements should be considered as part of a future Facility Plan.

Planning for Future Growth

As noted in previous communications with DSPUD, it is important for DSPUD and SLCWD to provide guidance on allowances for increased flows and loads due to projected new development in the service area. The detailed alternative analyses that must be started as the next step of project development must be based on certain flows and loads. Determination of what allowances should be made must be based on a plan for funding the incremental capacity. As previously noted, a viable option may be to proceed with detailed alternative analysis assuming minimal or no growth. Then, if appropriate, after the apparent best alternative is identified, a subsequent analysis could be completed to determine the changes required and increased costs for a somewhat larger capacity. To minimize rework, however, the initial growth and capacity determinations used for the alternative analysis should be as close as possible to the final determinations that will be used for project design.

In addition to projected new development, increased flows could occur as the result of increased occupancies of existing services. Historically, many second homes and lodgings have been vacant or lightly occupied and commercial activity has been relatively slow during the spring, summer, and fall. If any changes in the historical patterns are anticipated, these changes must be incorporated into wastewater flow and load projections, just like new growth.

In addition to determining growth and occupancy allowances for the upcoming improvement project, the Districts should also consider a “build-out” scenario. This would be useful in determining the possible ultimate capacity of treatment and disposal facilities, so that reasonable provisions for future staged expansion can be incorporated in the initial project.

Schedule for Future Work

As mentioned previously, full compliance with the new NPDES permit is required by April 2014. A schedule for key activities leading to compliance is shown in Figure 2.

As shown in the schedule, startup of plant improvements should occur in the late summer and fall of 2013 to assure compliance with the NPDES permit by the April 2014 deadline (a winter startup is not advisable). Allowing for two construction seasons, construction should start early in 2012. Preliminary design and detail design are expected to occur mostly throughout 2011. Therefore, facility planning and environmental analyses should be completed during the remainder of 2009 and 2010. Depending on the severity of environmental issues and any opposition to the proposed project, it may be difficult to meet this schedule. Accordingly, time is of the essence as DSPUD continues in the process of project development.

Two key decision points are shown in the schedule for DSPUD and SLCWD. First, soon after receiving this document, the Districts will need to determine which wastewater management alternatives considered herein (or others) should be evaluated in detail in a Facility Plan. At the same time, each District will need to decide how much future growth or change in occupancy rates, if any, should be assumed for the Facility Plan analyses, as discussed above. The final decision point regarding growth and capacity is shown near the end of the environmental process and before final definition of the recommended project, which will then be carried forward into preliminary design. Between the initial and final capacity determinations, the Districts will have some time to assess project funding options and the degree to which new growth will be able to participate in project funding.

Geotechnical investigations and surveys are shown at various times in the schedule. Initial preliminary work may be required to support facility planning. Subsequently, more detailed work will be needed to support preliminary design and detail design.

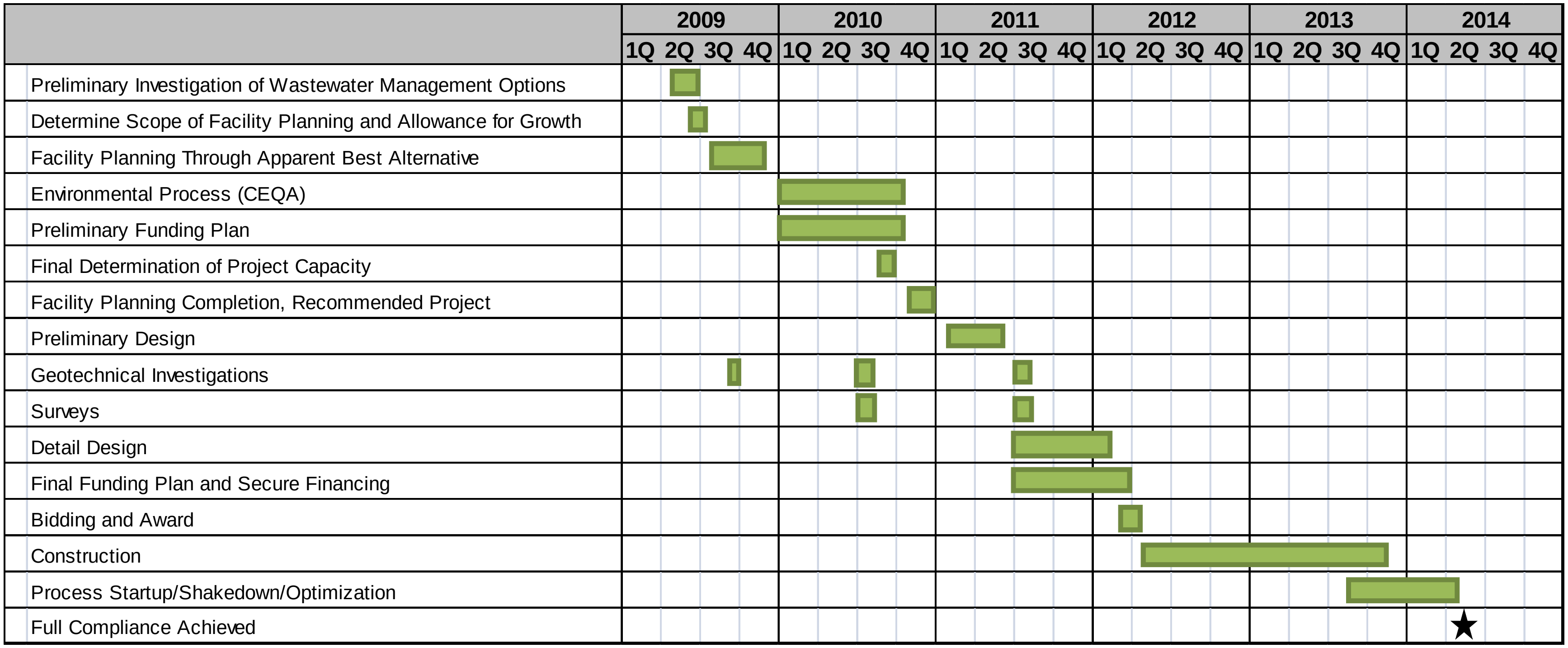

Figure 2
Project Development Schedule