Arnold Sewer System Master Plan

May 2005

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Arnold Sewer System
Master Plan

Calaveras County Water District

May, 2005

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Executive Summary

The Calaveras County Water District (District) is embarking on an effort to develop a District-wide financial plan for all its water and sewer service areas. To accomplish this task, a master plan describing conveyance, treatment, and effluent holding and disposal system improvement needs is required for the Arnold Sewer System. At the District's request, HDR evaluated the possibility of treating and disposing of sewage from the Millwoods Septage System at the Arnold Wastewater Treatment Plant (Arnold WWTP). HDR also evaluated the possibility of conveying, treating, and disposing of wastewater from the Avery Commercial area to the Arnold WWTP.

Current and Projected Flows

Analyses of historic data were conducted to determine the number of equivalent single family units (ESFUs) served and to characterize historic influent flows. Projected ESFUs and future flows were based on the growth anticipated for the service area and the District's standard unit flow rate of 195 gallons per day (gpd) per ESFU. Table 1 presents the projected breakdown of the existing and future ESFUs for the various service area scenarios.

Table 1. Projected Breakdown of Existing and New Connections

Location	Projected ESFUs at Buildout	Notes / Description
Existing Service Area ESFUs	638	Historic ESFUs as of 2004, includes Arnold and Avery ESFUs.
Infill -- Outside of Cedar Ridge Development	381	Infill in existing Arnold service area. Growth based on an additional 5 ESFUs per year.
Infill - Cedar Ridge Development	213	A new development that has been accepted into the service area. All new ESFUs are expected to be connected within the next 10 years.
Base Scenario	1,232	Buildout projection based on infill development and Cedar Ridge development ESFUs only.
Millwoods Septage System	177	Existing septage system outside of the service area. Area is essentially built-out (i.e., no increase in connections)
Existing Service Area Plus Millwoods (Scenario 1)	1,409	Buildout projections are based on the Base Scenario projections plus allowing the Millwoods septage system to be connected to the Arnold Sewer System.
Avery	22	Existing septage system outside of the service area. A portion of the system has already been connected to the Arnold system. The 22 ESFUs represent new connections.
Existing Service Area Plus Avery (Scenario 2)	1,254	Buildout projections are based on the Base Scenario projections plus allowing Avery to be connected to the Arnold Sewer System
Existing Service Area Plus Millwoods and Avery (Scenario 3)	1,431	Buildout projections are based on the Base Scenario projections plus allowing Avery and Millwoods to be connected to the Arnold Sewer System

Currently the Arnold WWTP receives approximately 75,000 gpd on an average dry weather flow (ADWF) basis. At buildout (under the Base Scenario), the ADWF is projected to increase to approximately 240,000 gpd based on the existing service area. Projected ADWFs associated with Scenarios 1, 2, and 3 are approximately 275,000, 245,000, and 280,000 gpd, respectively. The existing treatment and effluent holding and disposal facilities have a rated ADWF capacity of 170,000 gpd.

Regulatory Considerations

The Regional Water Quality Control Board (RWQCB) was contacted on December 16, 2004 to discuss potential changes and/or additions the District might expect in the near future. The RWQCB provided insight about its perceived areas of concern for the Arnold Sewer System. A summary of the information gathered is described below:

- The current Waste Discharge Requirements (WDR) is scheduled to expire in fiscal year 2007. A new Report of Waste Discharge will be required at that time.
- The RWQCB has concerns regarding the underlying groundwater quality at the Arnold WWTP. More groundwater monitoring wells for the percolation beds and irrigation fields will likely be required when the WDR is renewed.

In addition, based on past experience with similar wastewater treatment facilities, the following additional changes/requirements may be incorporated into the next WDR:

- Disinfection By-Products: Research has shown that chlorine disinfection results in the formation of disinfection by-products, primarily trihalomethanes (THMs) and haloacetic acids (HAAs), which are known human carcinogens. To minimize the impact on groundwater quality, the District should consider installing ultraviolet light (UV) disinfection when the existing disinfection system requires substantial maintenance or replacement.

Alternative Analyses

Alternative analyses were prepared to determine the cost effectiveness for incorporating the Millwoods service area and a future Avery commercial area into the Arnold service area.

Incorporation of Millwoods Service Area

Adding a settling tank adjacent to the existing Millwoods leachfield and routing the Millwoods septic tank effluent directly to the Arnold Sewer System were the two alternatives considered in the evaluation. The following is a summary of key findings and recommendations:

- **Septic Tank Improvements (Millwoods):** Regardless of which alternative is selected, screens would have to be installed in several existing septic tanks along with concrete lids and septic tank discharge piping improvements. The total estimated project cost for these improvements is $385,000.

- **Recommended Alternative:** Installing a settling basin and continuing to operate Millwoods as a separate system has a significantly lower net present worth cost. It is estimated that this alternative represents approximately 65 percent of the costs associated with abandoning the Millwoods treatment and disposal systems and routing this flow to the Arnold WWTP for subsequent treatment and disposal. Based on this cost comparison, it is recommended that Millwoods continue to operate as a separate system.

Incorporation of Avery Commercial Area

The Arnold WWTP currently receives a small amount of domestic sewage from the Avery Middle School and Safari Mobile Home Park. The District is considering expanding this service by providing sewer service to a future Avery commercial area. Providing service to this area is not expected to alter the costs or timeline requirements for the Arnold sewer system improvements described later in Table 2 and Table 3. In addition, the Avery force main and pumping station have adequate capacity to serve this expansion. However, a collection would need to be necessary to connect the commercial area to the Avery force main. Assuming this collection system expansion is paid for by the commercial area, adding this service area is attractive from a cost standpoint since it will provide added customers at no additional costs to the District.

Recommended Improvements and Timelines

Capacities for the existing facilities were determined to identify bottlenecks and improvements needed to accommodate future flows. Timeline requirements were based on evaluating project influent flows, specific system capacities, and an infill growth rate of 5 ESFUs per year. Two improvement phases are required for all four buildout scenarios.

A summary of the Phase I Improvements is shown in Table 2 along with estimated costs for the Base Scenario.[1] As shown, the total estimated project cost for the Phase I Improvements is $1,190,000. It is recommended that these improvements be implemented immediately to improve operations and maintenance and provide adequate capacity to accommodate future flows.

[1] Tables describing the improvements and timeline requirements for Scenarios 1, 2, and 3 are presented in Appendix F.

Table 2. Phase I Improvements (Base Scenario)

Cost Component	Estimated Costs ($)[a]
Collection System	
Lift Station 1	60,000[b]
Lift Station 2	250,000
Treatment Plant	
Secondary Clarifier and Return Activated Sludge (RAS) Pump	300,000
Dissolved Oxygen (DO) Control System	40,000
Effluent Pump	35,000
Site Piping	40,000
Effluent Disposal Evaluation	35,000
Subtotal A	760,000
Contingency (30 percent of Subtotal A)	230,000
Subtotal B[c]	990,000
Administration and Engineering (20 percent of Subtotal B)	200,000
Total Estimated Project Cost	**1,190,000**

a Estimated costs presented in terms of 2004 US dollars.
b Cost represents the District's contribution to this lift station and not the total estimated cost.
c Estimate of probable construction cost.

Approximately 22 acres of additional spray field irrigation and six percolation beds are required to accommodate increased flows and serve buildout. These improvements (referred to as the Phase II Improvements) are required to be in service by 2011 or when the ADWF approaches 130,000 gpd. The total estimated project cost for these improvements is $865,00 and includes an additional effluent holding tank.

A summary of the Phase III Improvements is shown in Table 3 along with estimated costs. As shown, the total estimated project cost for the Phase III Improvements is $2,380,000. These improvements are needed to be in service by 2020 when the ADWF approaches 170,000 gpd. The total number of ESFUs served in 2020 is estimated to be 940. Once these improvements are completed, the sewer system will have adequate capacity through buildout.

Table 3. Phase III Improvements (Base Scenario).

Cost Component	Estimated Costs ($)[a]
Collection System – Lift Station 3	125,000
Treatment Plant Expansion	1,400,000
Subtotal A	1,525,000
Contingency (30 percent of Subtotal A)	460,000
Subtotal B[b]	1,985,000
Administration and Engineering (20 percent of Subtotal B)	395,000
Total Estimated Project Cost	**2,380,000**

[a] Estimated costs presented in terms of 2004 US dollars.
[b] Estimate of probable construction cost.

Introduction

Introduction

The District is embarking on an effort to develop a District-wide financial plan for its water and sewer systems. To accomplish this task, master plans technical memoranda describing conveyance, treatment, storage, and disposal system improvements required to meet current and future needs must be developed.

This master plan report presents a summary of the results and findings for the Arnold Sewer System Master Planning Project. The intent of this project is to provide a basis for managed upgrade of the conveyance, treatment, storage, and disposal systems and provide financial information for a District-wide financial master plan.

Background

The District owns and operates the Arnold Wastewater Treatment Plant (Arnold WWTP) located next to Highway 4, four miles south of Arnold. The Arnold WWTP was designed in 1984 and began operation in June of 1986. Wastewater treatment processes consist of an extended oxidation ditch followed by clarification, chlorination, sand filtration, and effluent holding. Solids handling processes consist of two aerobic digesters and two sludge drying beds. Currently the District is in the process of installing a new belt filter press for biosolids dewatering. The treatment plant has an average dry weather flow (ADWF) capacity of 170,000 gallons per day (gpd) and the inflow presently averages about 75,000 gpd.

Effluent is disposed of via spray irrigation or subsurface disposal beds. Spray irrigation is used during the dry season for irrigation of up to 25 acres of native grassland, shrubs and trees. In addition, 11 subsurface disposal beds can be used throughout the year for effluent disposal. Potential groundwater impacts are monitored through three onsite monitoring wells. Discharge requirements and key treatment and effluent disposal provisions are discussed in the Regulatory Considerations section of this report.

Purpose and Specific Objectives

This purpose of this report is to describe the conveyance, treatment, storage, and disposal system improvements required to meet the current and future service area needs. In particular, this report provides the following information:

- Delineation of the service area (infill areas and Cedar Ridge). As alternatives, the following revisions to the service area are considered in this report:
  - The potential for providing wastewater treatment and disposal services for the Millwoods sewer system.
  - The potential for providing wastewater treatment and disposal services for the Avery Community Sewer System in addition to the Avery Middle School and Safari Mobile Home Park.

- Characterization of historic wastewater flows, including existing and projected average dry weather, average day, peak month, maximum day, and peak wet weather flows and infiltration and inflow (I&I).
- Projection of future flows.
- Description of the existing facilities and estimated capacities.
- Evaluation of the existing and future options for the conveyance, treatment, storage, and disposal systems.
- Identification of the improvements needed to meet growth, improve operations, comply with current and known future regulations, and correct deficiencies.
- Recommendations for sewer system improvements needed to serve buildout conditions.
- Timelines and cost information for constructing the recommended improvements.

Current and Projected Flows and Wastewater Characteristics

Current and Projected Flows and Wastewater Characteristics

Analyses of service area and treatment plant operating data were conducted to characterize historic influent flows and pollutant loads. Projected future flows were based on the growth anticipated for the service area and the District's standard unit flow rate as described below.

Service Area

The area served by the Arnold WWTP is shown in Figure 1. The service area is composed of approximately 590 acres of the Arnold downtown area. The source of the Arnold WWTP's influent is from primarily domestic and light commercial sources. The Arnold WWTP also receives a relatively small amount of domestic sewage from the Avery Middle School and Safari Mobile Home Park located in Avery, south of the immediate service area boundaries. No industries discharge wastewater to the collection system.

Existing Service Area

The Arnold WWTP currently serves 638 ESFUs. Most connections are single family residences, while some connections serve commercial or multi-family developments. To characterize wastewater flows, the District uses a unit called an equivalent single family unit (ESFU). For single-family residential development, one connection is typically equivalent to one ESFU. Commercial and multi-family connections are assigned a number of ESFUs to represent the flow they contribute to the collection system. In most cases, commercial and multi-family connections each represent more than one ESFU.

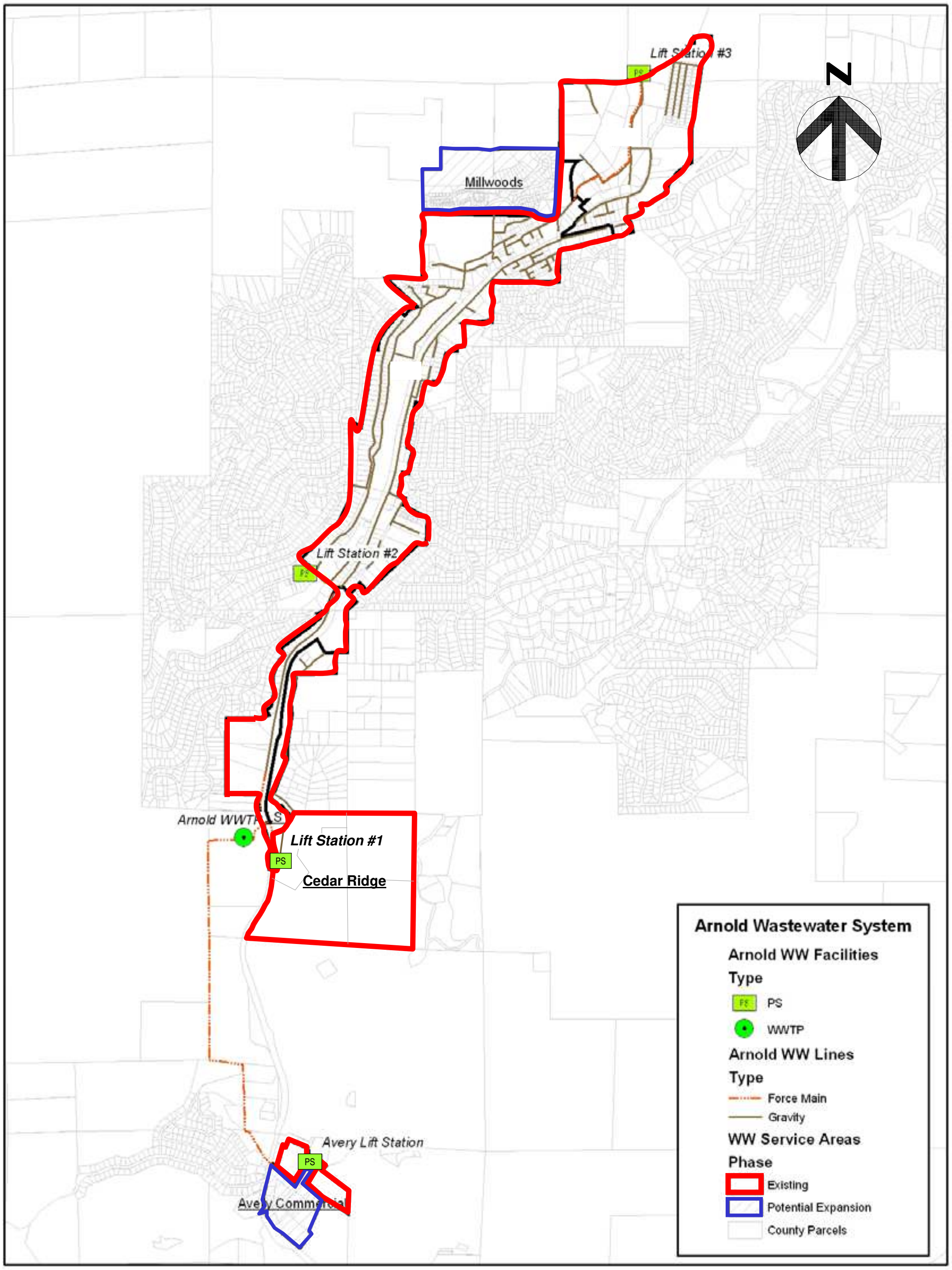
Service Area Growth Scenarios

Future growth within the service area can come in three ways, infill within the service area, service area expansion, and connection of existing septic systems to the Arnold sewer system as described below.

Infill

The 1984 Engineer's Report for the Arnold Wastewater Assessment District (as amended) estimated an ultimate total of 986 equivalent single family units (ESFUs) within the service area. The WWTP currently serves 638 ESFUs.[2] However, 33 of the existing ESFUs are in Avery, outside the original service area. Therefore the current ESFUs in the original service area are estimated as 638 minus 33, or 605 ESFUs. The infill potential in the Arnold service area is estimated as the difference between 986 and 605, or 381 ESFUs.

[2] This value includes the Avery Middle School and Safari Mobile Home Park.



Arnold Sewer Service Area

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Figure 1

The Cedar Ridge development area has been accepted into the Arnold service area. Cedar Ridge is a 169-acre residential and commercial development located east of the Arnold WWTP. The development is expected to represent 213 ESFUs (100 single family units, 120 multi-family units, and 12-acre hotel and conference facilities). The development will be completed over a four-year period, starting in 2005, and is expected to be fully inhabited within the next ten years.

Service Area Expansion

Two areas were considered for potential expansion of the service area in this master plan. The first is the Millwoods subdivision, which represents 177 ESFUs and is considered to be essentially built-out. Properties in the Millwoods subdivision have individual septic tanks, and the septic tank effluent is collected and conveyed to the Millwoods pump station. The pump station directs flow to a disposal field located in the western portion of the subdivision. The pump station could be re-configured to direct flow to the Arnold collection system. The current ADWF from Millwoods is approximately 10,000 gpd, or 56 gpd per ESFU. The annual average flow is 10,800 gpd.

The second area for service expansion is a portion of Avery currently designated as commercial. This is the area that is most likely to be connected to the Arnold WWTP over time due to a planned extension of the sewer line. It is estimated that this expansion area will ultimately serve an estimated 22 ESFUs.[3]

Connection of Existing Septic Systems

The potential for connecting residences to the Arnold sewer system that currently have individual, on-site septic systems was discussed during the January 25, 2005 Public Meeting. Some individuals expressed concern that future on-site septic failures would require implementing a regional sewer solution.

In their response to public comments (see Appendix G), the District explained that implementing a regional solution to eliminate on-site septic systems would be initiated by the county and/or other state agencies, not the District. The District also explained that in the event that a health threat was identified, impacts to the existing sewer system would need to be funded by those directly benefiting from the solution, not by existing customers. Based on this assessment, the District concluded that the scope of this master plan cannot speculate on the need to develop a regional solution to eliminate the septic system. Therefore, connection of existing septic systems to the Arnold sewer system was not considered in this master plan.

Historic and Projected Service Area Contributions

The District provided historical data pertaining to influent flows and connections served by the Arnold WWTP. Historic values for flow and ESFUs are shown in Table 4. ADWF is the average flow from June through September. Between 1991 and 2004, the average geometric

[3] Estimates provided by the District on January 25, 2005.

growth rate has been approximately 1.0 percent per year. The highest growth rates of six percent per year occurred in 2002.

A projection of buildout conditions was made to estimate the ultimate flows that could reach the treatment plant. A total of four potential buildout scenarios were defined based on the possible inclusion of Millwoods and Avery. The projected ESFUs associated with these four scenarios are shown in Table 5.

Table 4. Historic Growth in Influent Flow and ESFUs Served.

Year	Average Dry Weather Flow (gpd)	Annual Average Flow (gpd)	ESFUs	Annual Increase in ESFUs	Percent Increase in ESFUs	ADWF / ESFU (gpd)	Ratio of Annual Average to ADWF
1991	61,300	-	561	-	-	109	-
1992	63,300	-	566	5	1%	112	-
1993	54,800	-	572	6	1%	96	-
1994	71,800	-	577	5	1%	124	-
1995	75,000	81,000	583	6	1%	129	1.08
1996	77,500	83,000	589	6	1%	132	1.07
1997	70,700	72,000	595	6	1%	119	1.02
1998	66,700	70,000	601	6	1%	111	1.05
1999	59,600	64,000	607	6	1%	98	1.07
2000	57,600	61,000	583	(24)	-4%	99	1.06
2001	60,000	61,000	589	6	1%	102	1.02
2002	54,700	59,000	625	36	6%	88	1.08
2003	74,300	70,000	631	6	1%	118	0.94
2004	75,000	-	638	7	1%	118	-

Table 5. Projected Breakdown of Existing and Future Connections.

Location	Scenario	Projected Buildout ESFUs[a]	Notes / Description
Existing Service Area ESFUs	--	638	Historic ESFUs as of 2004, includes Arnold and Avery ESFUs.
Infill -- Outside of Cedar Ridge Development	--	*381*	Infill in existing Arnold service area.
Infill - Cedar Ridge Development	--	*213*	A new development that has been accepted into the service area. All new ESFUs are expected to be connected within the next 10 years.
Base Scenario		1,232	Buildout projections are based on infill development and Cedar Ridge development ESFUs only.
Millwoods Septage System		*177*	Existing septage system outside of the service area. Area is essentially built-out (i.e., no increase in connections)
Existing Service Area Plus Millwoods	1	1,409	Buildout projections are based on the base scenario plus allowing the MIllwoods septage system to be connected to the Arnold Sewer System.
Avery		*22*	Existing septage system outside of the service area. A portion of the system has already been connected to the Arnold system. The 2 ESFUs represent new connections.
Existing Service Area Plus Avery	2	1,254	Buildout projections are based on the base scenario plus allowing Avery to be connected to the Arnold Sewer System
Existing Service Area Plus Millwoods and Avery	3	1,431	Buildout projections are based on the base scenario plus allowing Avery and Millwoods septage systems to be connected to the Arnold Sewer System

a Future ESFUs shown in italics. In 2002 there were 625 ESFUs. Based on a one percent growth rate, the ESFUs in 2004 is estimated at 638.

Historic and Projected Flows

A summary of the flow estimating assumptions is shown in Table 6.

Table 6. Flow Estimating Criteria.

Parameter	2005	2015	2025	Comments
Arnold ADWF (gpd/ESFU)	118	157	195	Phased increase over 20 years
Millwoods ADWF (gpd/ESFU)	56	126	195	Phased increase over 20 years
Cedar Ridge ADWF (gpd/ESFU)	195	195	195	New development at design rate
Avery Expansion ADWF (gpd/ESFU)	195	195	195	New development at design rate
Ratio of Annual Average Flow to ADWF	1.08	1.08	1.08	
Maximum month I&I (gpd/acre)	56	56	56	
Ratio of Maximum Day to Annual Average	1.72	1.6	1.5	Phased decrease over 20 years
Ratio of Peak Hour to Annual Average	3.0	3.0	3.0	Assumed value

The following are descriptions of the methodologies used to characterize historic and project future flows conveyed to the Arnold WWTP.

Average Dry Weather Flow

The District's Board of Directors has adopted a policy to plan for an ADWF of 195 gallons per day (gpd) per ESFU. The existing flow per ESFU is approximately 118 gpd in the existing service area, and only 56 gpd in Millwoods. The increase in flows is expected to occur as more properties are inhabited and used year-round and the number of residents per household increase to values which are typical for California residences. All areas of new development are expected to contribute 195 gpd per ESFU under ADWF conditions. In existing developed areas, the flow per ESFU is expected to transition from the existing value to 195 over a 20-year period, from 2005 to 2025.

Average Annual Flow

The data in Table 4 show that the ratio of average annual flow to ADWF ranges between 0.94 and 1.08. For planning purposes, a ratio of 1.08 will be used for estimating current and future average annual flows.

Maximum Month

The District provided daily influent flow data for 2001 through 2004. Based on a review of this data, the highest average flow for a 30-day period was 80,000 gpd in December 2001. The ADWF during that year was 60,000 gpd, meaning that the maximum month I&I was 20,000 gpd. The approximate active service area at that time was 360 acres, resulting in a maximum month inflow and infiltration (I&I) of 56 gpd/acre. This value of 56 gpd/acre will be used for estimating future I&I in the service area.

Maximum Day

A statistical analysis of the influent flow data was performed to determine the maximum day flow. As shown in Figure 2, the maximum day flow was selected as the 99.7^{th} percentile value of the observed flows. This value was 107,600 gpd. The average flow for the corresponding period was 62,500 gpd. Based on the ratio of these two values, the current peaking factor for maximum day compared to annual average is 1.72. This ratio is assumed to drop from 1.72 to 1.5 over the next 20 years, due to better construction practices that are expected to reduce I&I.

Peak Hour Flow

Hourly flow data were not available at the time this memorandum was prepared. The assumed peak hour flow is 3.0 times the annual average flow based on typical peaking factors and previous factors used for the District's master planning efforts.

Service Area Scenarios and Projected Flows

As previously described, different scenarios were created to represent different buildout conditions with varying degrees of service area expansion. The projected flows under the different buildout scenarios are shown in Table 7.

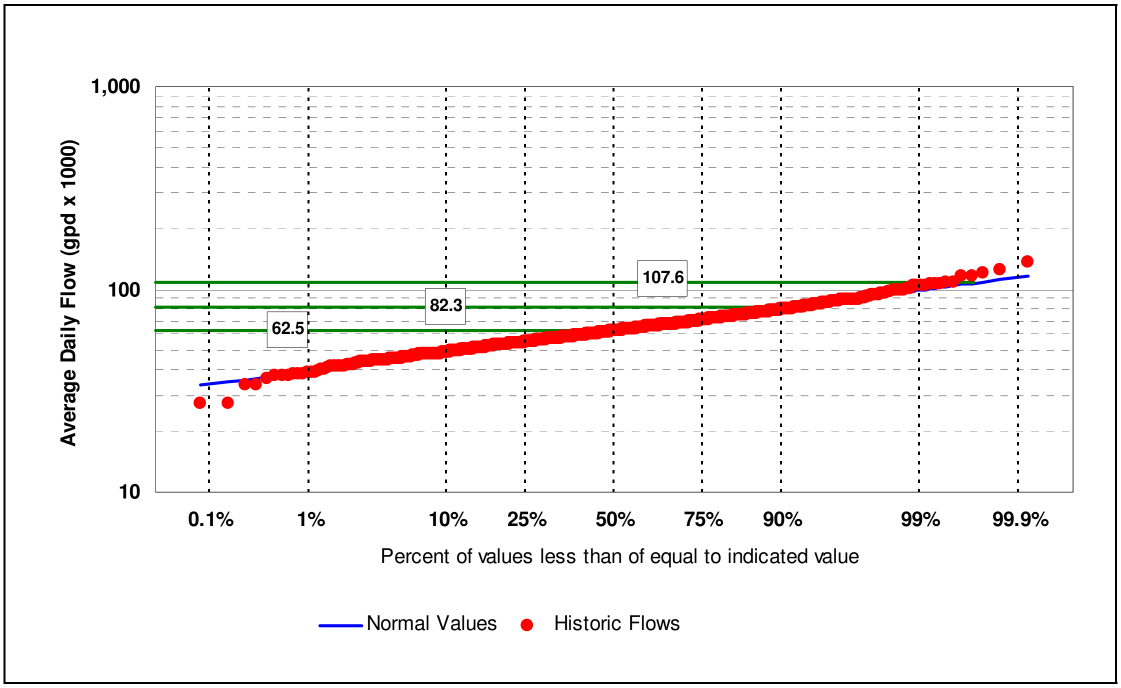


Figure 2. Average Daily Flows and Estimated Maximum Peak Day Flow.

Table 7. Scenarios for Ultimate Buildout.

Location	Currently Connected ESFUs	Ultimate Area (acres)	Additional ESFUs	Ultimate ESFUs	Ultimate ADWF (gpd)	Ultimate Annual Average (gpd)	Ultimate Maximum Month (gpd)	Ultimate Maximum Day (gpd)	Ultimate Peak Hour (gpd)
Existing Service Area	638	740	594	1,232	240,240	259,459	281,680	389,189	778,378
Millwoods	--	78	177	177	34,515	37,276	38,883	55,914	111,829
Avery Expansion	--	29	22	22	4,290	4,633	5,914	6,950	13,900
Base Scenario				1,232	240,240	259,459	281,680	389,189	778,378
Scenario 1 – Existing Service Area Plus Millwoods				1,409	274,755	296,735	320,563	445,103	890,207
Scenario 2 – Existing Service Area Plus Avery				1,254	244,530	264,092	287,594	396,139	792,278
Scenario 3 – Existing Service Area Plus Millwoods and Avery				1,431	279,045	301,368	326,477	452,053	904,107

Notes:

Design ADWF/ESFU (gpd):	195
Ratio of annul average flow to ADWF:	1.08
Maximum month I&I (gpd/acre):	56
Ultimate ratio of maximum day to annual average:	1.5
Ratio of peak hour to annual average:	3.0

Projections of interim growth between 2005 and buildout were made using growth rates developed in cooperation with District staff. For the purposes of this master planning effort, the assumed growth rate for infill in the Arnold service area is 5 ESFUs per year, which is equivalent to a growth rate of 0.8 percent per year. For Cedar Ridge, all 213 ESFUs are assumed to be connected between 2005 and 2015. For Millwoods, all 177 ESFUs are assumed to be connected in the middle or towards the end of 2005. For the Avery expansion area, the 22 new ESFUs are assumed to be connected over a 20-year period (approximately 1 ESFU per year).

Flow projections were developed for 2005, 2015, and 2025. Calculated flows included ADWF, annual average flow, and peak hour flow. To limit the possibility of sewer overflows, the collection system should be sized to handle peak hour flows. Flow projections were developed only for Scenario 3 (infill in the Arnold service area and the addition of Millwoods and the Avery expansion area). Flows for the other development scenarios would be slightly lower. The flow projections are shown in Table 8.

Table 8. Flow Projections for Interim Years – Scenario 3.

Parameter	2005	2015	2025
Arnold ESFU	638	688	738
Arnold ADWF (gpd/ESFU)	118	157	195
Arnold ADWF (gpd)	75,284	107,672	143,910
Arnold Annual Average (gpd)	81,307	116,286	155,423
Arnold Peak Flow (gpd)	243,921	348,858	466,269
Cedar Ridge ESFU	0	213	213
Cedar Ridge ADWF (gpd/ESFU)	195	195	195
Cedar Ridge ADWF (gpd)	0	41,535	41,535
Cedar Ridge Annual Average (gpd)	0	44,858	44,858
Cedar Ridge Peak Hour (gpd)	0	134,573	134,573
Millwoods ESFU	0	177	177
Millwoods ADWF (gpd/ESFU)	56	126	195
Millwoods ADWF (gpd)	0	22,214	34,515
Millwoods Annual Average (gpd)	0	23,991	37,276
Millwoods Peak Hour (gpd)	0	71,973	111,829
Avery Expansion ESFU	0	11	22
Avery Expansion ADWF (gpd/ESFU)	195	195	195
Avery Expansion ADWF (gpd)	0	2,145	4,290
Avery Expansion Annual Average (gpd)	0	2,317	4,633
Avery Expansion Peak Hour (gpd)	0	6,950	13,900
Combined ADWF (gpd)	75,284	173,566	224,250
Combined Annual Average (gpd)	81,307	187,452	242,190
Combined Peak Hour (gpd)	243,921	562,354	726,571

Notes:

Infill growth (ESFU/year) 5
Annual average to ADWF 1.08
Peak hour to annual average 3.0
0 = No ESFUs connected at the beginning of 2005.

Historic and Projected Wastewater Characteristics

Historic wastewater characteristics were estimated by the three methods described below. A copy of the calculations prepared for these analyses are attached in Appendix B for reference.

- **Statistical Analysis of Historic BOD Loads:** Statistical analyses of historic biochemical oxygen demand (BOD) and total suspended solids (TSS) loads between December 2002 and May 2004 were conducted. The overall average annual (50^{th} percentile value) BOD and TSS loads were determined to be 124 lb BOD/d and 121 lb TSS/day, respectively. Based on the current number of connections (638 ESFUs), unit loading rates are estimated to be 0.19 lb BOD per ESFU and 0.19 lb TSS per ESFU.
- **Historic BOD and TSS Concentrations:** A review of historic influent BOD and TSS concentrations between December 2002 and May 2004 was conducted. The overall average BOD and TSS concentrations during this period were determined to be 242 and 237 mg/L, respectively. Based on these concentrations, and the current unit flow rates of 118 gpd, the estimated unit loading rates are estimated to be 0.24 lb BOD per ESFU and 0.23 lb TSS per ESFU.
- **Statistical Analysis of Historic Per Capita Loading Rates:** The District routinely monitors historic unit loading rates entering the Arnold WWTP. Statistical analyses of these historic values were performed for data collected between December 2002 and May 2004. The overall average (50^{th} percentile values) unit loading rates were determined to be 0.22 lb BOD per ESFU and 0.19 lb TSS per ESFU.

Based on a review of the analysis results, loading rates of 0.24 lb BOD per ESFU and 0.20 lb TSS per ESFU will be used as the basis for estimating current BOD and TSS concentrations. Future BOD and TSS concentrations are assumed to be equal to current values.

Similar analyses were prepared to determine the historic peak month BOD and TSS loads. In general, peak month pollutant loads were equal to twice the average annuals loads. Based on these results a load peak factor of 2.0 will be used to project the future peak month wastewater characteristics at buildout.

Summary of Current and Projected Flows and Wastewater Characteristics

Table 9 presents a summary of current and projected flows and loads for the four growth scenarios. Calculations showing how these flow and load projections were developed are shown in Appendix C. The ADWF, average annual, peak month flows and loads will be used to assess the majority of the treatment plant, effluent holding, and disposal facilities. The peak flows will be used to assess the collection system and treatment plant headworks and effluent pumping station.

Table 9. Current and Buildout Wastewater Flows and Characteristics.

Condition	Influent Concentrations (mg/L)		Base Scenario	Scenario 1	Scenario 2	Scenario 3
	BOD	TSS	Flow (gpd)	Flow (gpd)	Flow (gpd)	Flow (gpd)
Current Conditions						
ADWF	--	--	75,284	75,284	75,284	75,284
Average Annual	226	188	81,307	81,307	81,307	81,307
Peak Month	384	322	95,444	95,444	95,444	95,444
Maximum Day	--	--	139,848	139,848	139,848	139,848
Peak Flow	--	--	243,921	243,921	243,921	243,921
Buildout Conditions						
ADWF	--	--	240,240	274,755	244,530	279,045
Average Annual	226	188	259,459	296,735	264,092	301,368
Peak Month	384	322	281,680	320,563	287,594	326,477
Maximum Day	--	--	389,189	445,103	396,139	452,053
Peak Flow	--	--	778,378	890,207	792,278	904,107

Notes:

Scenario 1 includes existing service area plus Millwoods
Scenario 2 includes existing service area plus Avery
Scenario 3 includes existing service area plus Millwoods and Avery

Regulatory Considerations

Regulatory Considerations

A summary of current waste discharge requirements (WDR) for the Arnold Sewer System is presented below. In addition, potential future changes to the WDR are discussed.

Waste Discharge Requirements

The current WDR (Order No. 97-073) for the Arnold WWTP was adopted by the Regional Water Quality Control Board (RWQCB) in April 1997. A copy of the WDR can be found in Appendix A. The WDR covers discharge prohibitions and specifications, effluent limitations, reclamation specifications, solids disposal requirements, groundwater limitations, and other provisions. Portions of the WDR pertinent to wastewater treatment and disposal systems are discussed below.

Discharge Requirements

Treated effluent is permitted to be discharged to either the spray irrigation fields or subsurface disposal beds provided the effluent quality meets the requirements stipulated in the WDR.

Numerical Effluent Limits

Table 10 summarizes the treated effluent requirements listed in the WDR.

Table 10. Effluent Discharge Specifications.

Constituent	Units	Effluent Limitation		
		Average Dry Weather	Monthly Average	Monthly Maximum
Flow	gpd	170,000	--	--
BOD[a]	mg/L	--	40	80
Settable Solids	mg/L	--	0.5	1.0
Total Coliform	MPN/100 mL	--	23[b]	240[c]

a 5-day, 20°C Biochemical Oxygen Demand
b Monthly median value.
c Daily Maximum.

Other Key Requirements

In addition to the limits shown above, the District must comply with the following key specifications:

Discharge Limits and Specifications

- Objectionable odors originating at the facility shall not be perceivable beyond the limits of the wastewater treatment and disposal area.
- The treatment facilities shall be designed, constructed, operated, and maintained to prevent inundation or washout due to floods with a 100-year return frequency.
- Bypass or overflow of untreated or partially treated waste is prohibited.

Irrigation

- Public contact with the reclaimed water shall be precluded through such means as fences, signs, and other acceptable alternatives.
- Areas irrigated with reclaimed water shall be managed to prevent the breeding of mosquitoes.
- Reclaimed water for irrigation shall be managed to minimize erosion, runoff, and movement of aerosols from the disposal area.
- Direct or windblown spray shall be confined to the designated reclamation area and prevented from contacting drinking water facilities.
- Spray irrigated effluent shall not occur during periods of precipitation and for at least 24 hours after cessation of precipitation, or when winds exceed 30 mph.
- Storm water runoff from the irrigation field shall not be discharged to any surface water drainage course within 48-hours of the last application of reclaimed water.
- Reclaimed water for irrigation shall be managed to minimize erosion, runoff, and movement of aerosols from the disposal area.

Ground Water Limitations

- The discharge shall not cause underlying ground water to exceed a most probable number of total coliform organisms of 2.2/100 mL over any seven-day period.
- The discharge shall not contain concentrations of chemical constituents in amounts that adversely affect agricultural use.
- The discharge shall not contain taste or odor-producing substances in concentrations that cause nuisance or adversely affect beneficial uses.
- The discharge shall not contain chemicals, heavy metals or trace elements in concentrations that adversely affect beneficial uses or exceed maximum contaminant levels specified in 22 California Code of Regulations (CCR), Division 4, Chapter 15.

Possible Changes to Permit Requirements and Areas of Concern

The RWQCB was contacted on December 16, 2004 to discuss potential changes and/or additions the District might expect in the future. The RWQCB provided insight about its perceived areas of concern for the Arnold Sewer System. A summary of the information gathered from this effort is described below.

- The current WDR is scheduled to expire in fiscal year 2007. A new Report of Waste Discharge will be required at that time.
- The RWQCB has concerns regarding the underlying groundwater quality at the Arnold WWTP. More groundwater monitoring wells for the subsurface disposal beds and irrigation field will likely be added when the WDR is renewed.

In addition, based on past experience with similar wastewater facilities, the following additional changes/requirements may be incorporated into the next WDR:

- **Disinfection By-Products:** Research has shown that chlorine disinfection results in the formation of disinfection byproducts, primarily trihalomethanes (THMs) and haloacetic acids (HAAs), which are know human carcinogens. To minimize the impact on groundwater quality, the District should consider installing ultraviolet light (UV) disinfection when the existing disinfection system requires substantial maintenance or replacement.

Description and Evaluation of Existing Facilities

Description and Evaluation of Existing Facilities

The existing wastewater facilities serving Arnold consist of a conveyance system, treatment plant, effluent holding and disposal facilities. The attributes of each facility and a summary of the results of a capacity evaluation are described below.

Sewer Conveyance System

A schematic of the Arnold collection system in shown in Figure 3. The existing collection system includes approximately 15 miles of pipe and four lift stations.

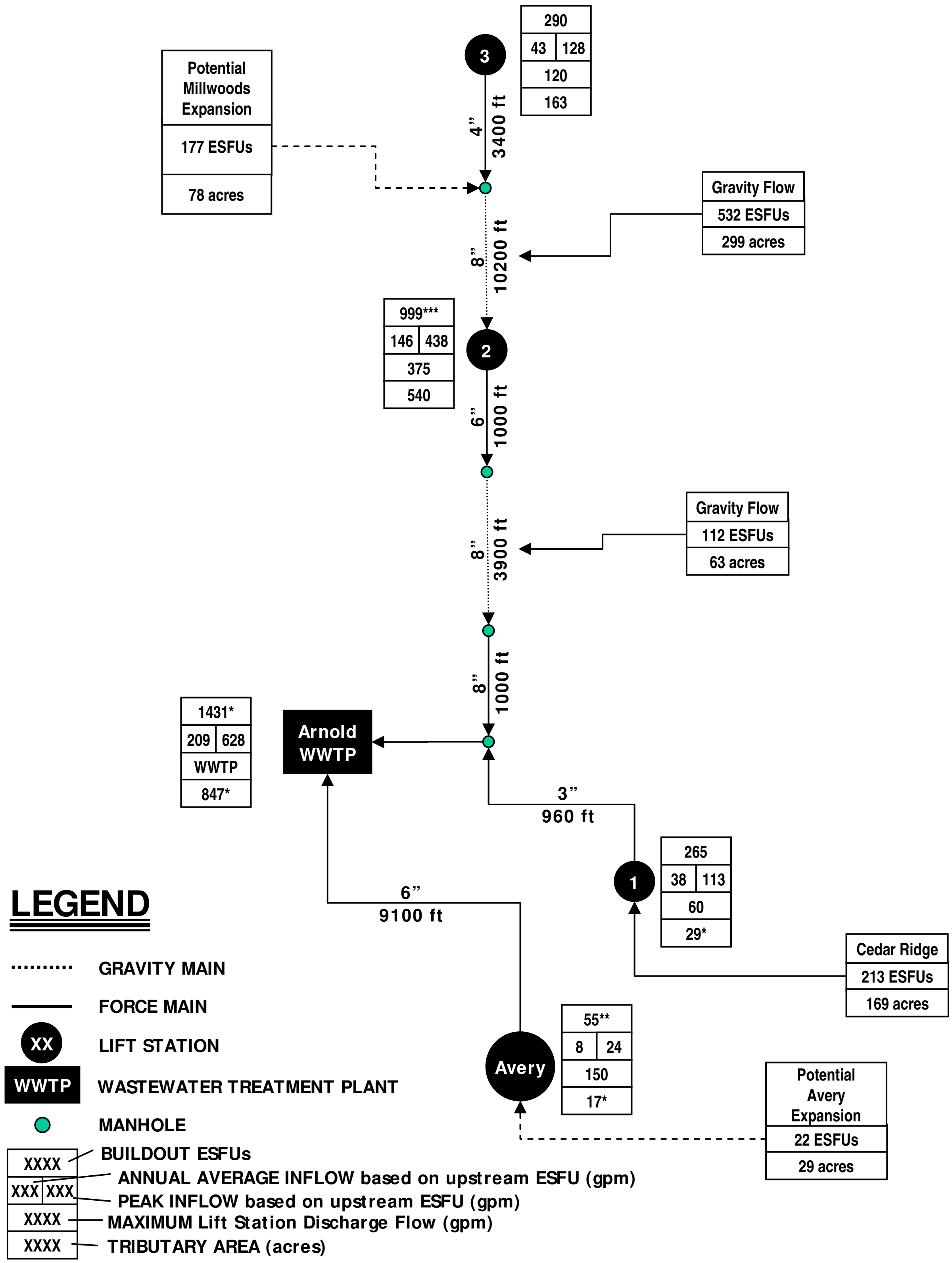
The northernmost portion of the service area drains to Lift Station 3 on Dunbar Road. Lift Station 3 pumps into the 4-inch diameter White Pines Force Main, which runs south into the White Pines Interceptor. The White Pines Interceptor is an 8-inch diameter gravity line running south parallel to Highway 4, collecting gravity flow from both sides of the service area. The White Pines Interceptor terminates at Lift Station 2, on Pines Drive. Lift Station 2 pumps into the 6-inch diameter Meadowmont Force Main, which runs east to Highway 4 and then south approximately 300 feet along Highway 4. At this point the Meadowmont Force Main empties into an 8-inch gravity line called Lateral MM. Lateral MM runs south along Highway 4 for approximately 3,900 feet, to a drop manhole beside Highway 4. At the drop manhole, the line drops in elevation and becomes a pressure line, called the Lakemont Force Main. The 8-inch diameter Lakemont Force Main then flows to the Arnold WWTP.

In the southernmost portion of the collection system, areas east of Highway 4 are lower than the Highway. A gravity lateral on the eastern service area boundary gathers flow from these properties and conveys it to Lift Station 1, which is located near Highway 4 at the southern edge of the Arnold service area. Lift Station 1 pumps into the 3-inch diameter Arnold Force Main, which flows north along Highway 4 to the Arnold WWTP entrance. At this point the Arnold Force Main meets the Lakemont Force Main, and flow enters the plant.

The fourth lift station is located in Avery, a small community located approximately one mile south of the treatment plant. The Avery Pump Station (APS) collects flow from the Avery Middle School and the Safari Mobile Home Park. The flow is then pumped through the 6-inch diameter Avery Force Main directly to the headworks of the Arnold WWTP.

The entire gravity collection system consists of 51,200 feet of 6-inch pipe and 14,000 feet of 8-inch pipe. The force mains, ranging from 3-inch to 8-inch, have a total length of 16,100 feet.

The available data for the four lift stations is shown in Table 11.



ARNOLD TRUNK SEWER SYSTEM WITH BUILDOUT ESFU AND PEAK FLOWS

Notes:
* Total for Scenario 3.
** 33 without Avery Expansion.
*** 822 without Millwoods Expansion.

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Figure 3

Table 11. Pump Station Data.

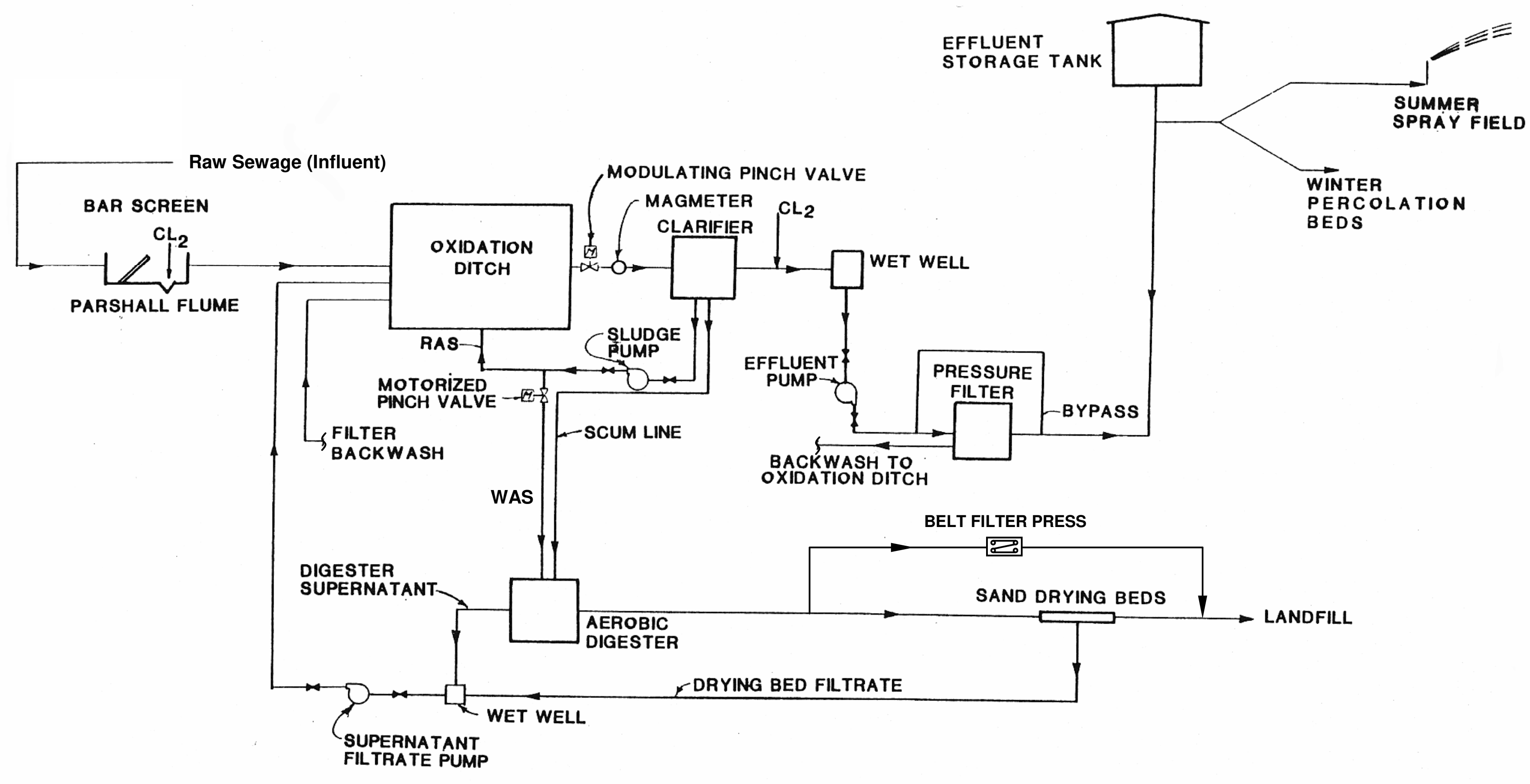
	Avery Lift Station	Lift Station 1	Lift Station 2	Lift Station 3
Pumps	Two 15-hp submersible turbine	Two 5-hp submersible grinder	Two 10-hp submersible non-clog	Two 5 hp submersible grinder followed by two 5-hp dry pit non-clog
Average Design Inflow (gpm)	N/A	12	165	29
Peak Design Inflow (gpm)	N/A	30	350	105
Capacity – one pump				
Pumping Rate (gpm)	110	40	275	81
Head (ft)	400	103	62	181
Capacity – both pumps				
Pumping Rate (gpm)	150	60	375	120
Head (ft)	500	111	68	198
Capacity with Both Pumps (gpd)	**216,000**	**86,400**	**540,000**	**172,800**

N/A = Not Available

Wastewater Treatment Plant

The Arnold WWTP consists of an extended oxidation ditch followed by clarification, chlorination, sand filtration, an enclosed storage tank, eleven subsurface disposal beds, and a 25-acre spray irrigation field. Additionally, there are two aerobic digesters and two sludge drying beds for solids treatment. The District is currently in the process of installing a belt filter press for solids dewatering. According to the WDR, the treatment facility, holding tank, and disposal beds have a design ADWF capacity of 170,000 gpd.

A process schematic and site plan of the Arnold WWTP are shown in Figure 4 and Figure 5, respectively. A summary of key design criteria and operating parameters for the major unit processes is presented in Table 12.



Notes:

RAS = Return Activated Sludge
WAS = Waste Activated Sludge

Arnold WWTP Process Schematic

Figure 4

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ARNOLD WWTP SITE PLAN
SCALE: 1" = 150'

HDR

Figure 5

Table 12. Key Design and Operating Criteria.

	Headworks	
Comminutor	Number	1
	Capacity	626,000 gpd @ Peak Hour
Parshall Flume	Number	1
	Throat Size	3 inches
	Flow Range	19,000 gpd to 777,000 gpd
Chlorine Diffuser	Number	1
	Capacity	500 lbs chlorine/day
Bypass Bar Screen	Number	1
	Bar Spacing	2-inch
	Secondary Treatment	
Oxidation Ditch	Number	1
	Maximum Side Water Depth	11 feet
	Volume	175,000 gallons @ maximum depth
	Detention Time	24 hours @ ADWF
	Mixed Liquor Suspended Solids	3,000 to 6,000 mg/L
	Mean Cell Residence Time	20 to 30 days
	Organic Loading Rate (Maximum)	20 lbs BOD/day/1,000cf @ peak month
	Dissolved Oxygen Concentration	1 to 3 mg/L
Air Diffuser	Number of Aeration Head	9
	Number of Diffusers per Head	8
	Type	Fine Bubble
	Capacity	500 cfm
Low Speed Mixer	Number	2
	Horsepower, each	1.5 hp
	Flow Control Vault	
Pinch Valve	Number	1
	Maximum Flow Regulating Capacity	180,000 gpd
Clarifier	Number	1
	Type	Center Feed
	Diameter	26 feet
	Side Water Depth	10 feet
	Volume	40,000 gallons
	Hydraulic Loading Rate	330 gpd/sf @ ADWF
	Solids Loading Rate	25 lbs/day/sf @ ADWF
Sludge Pump	Number	2
	Type	Variable Frequency Belt Drive
	Capacity, each	60 to 125 gpm
	Recycling Rate	100 percent at average annual flow with 1 pump out of service
Effluent Pump	Number	2
	Type	Vertical Turbine

	Capacity, ea	125 gpm
	Total Dynamic Head	200 feet
Pressure Filter	Number	2
	Type	Single Media Sand
	Volume	2, 500 gallons each
	Hydraulic Capacity	125 gpm
	Hydraulic Loading Rate	3.8 gpm/sf @ ADWF of 170,000 gpd
	Maximum Loading Rate	10 gpm/sf
	Backwash Flow Rate	500 gpm
	Backwash Duration	10 to 15 minutes
	Backwash Hydraulic Loading	15 gpm/sf
	Backwash Air	5 cfm/sf
Blower[a]	Number	3 + 1 standby
	Type	Positive Displacement
	Horsepower	3 - 15 hp; 1 - 10 hp
Disinfection		
Feed Tank	Number	1
Metering Pump (Chlorination)	Number	1
	Type	Peristaltic
	Capacity	30 gpd
	Minimum Residual	0.2 mg/L
	Contact Time	30 minutes through Filters @ peak hour flow
Sludge Treatment		
Aerobic Digester	Number	1
	Compartments	2
	Volume, ea	9,050 gallons
	Sludge Age	15 days
Mixer	Number	2, one each compartment
	Horsepower	1 hp
Supernatant/ Filtrate Pump	Number	2
	Type	Submersible, non-clog
	Capacity, ea	50 gpm
	Total Dynamic Head	31 feet
Sludge Drying Bed	Number	3
	Surface Area, ea	1,000 sf
	Maximum Solids Loading Rate	25 lb/sf/yr
Belt Filter Press[b]	Number of Units	1
	Belt Width	0.7 m
	Rated Capacity, ea	50 gpm

[a] Blowers are shared between the oxidation ditch, digesters, and filters.
[b] Scheduled to be installed by July 2005

Effluent Holding and Disposal

Following filtration and disinfection, the treated effluent is pumped to a 262,500-gallon enclosed steel holding tank. This tank is located in the northwest corner of the treatment plant site, at the highest elevation in the system. The tank is designed to provide a minimum of one-day holding capacity for the treated effluent. From the tank, effluent can be discharged by gravity to either the spray irrigation area or disposal beds.

The spray irrigation area consists of eight different pressure zones spanning a total area of 25 acres. Each area consists of native grassland, shrubs, and trees. The upper-most pressure zone consists of ten sprinklers with 77-ft diameter spray circles capable of discharging 4.7 gpm per sprinkler. The remaining pressure zones each have between nine and fourteen sprinklers with 100-ft diameter spray circles capable of discharging 11.7 gpm per sprinkler. Overall, the design application rate of the entire spray irrigation system is 1.8 inches per week. When in operation, effluent disposal is accomplished by a combination of plant uptake, evaporation, evapotranspiration, and percolation. The spray irrigation system is used only during the dry weather season, in accordance with the WDR.

Whenever the spray irrigation system cannot be used, effluent is disposed of using the 11 subsurface disposal beds which are located around the periphery of the spray irrigation areas. Each disposal bed consists of sixteen parallel trenches with 100-ft long distribution laterals. Each trench has 10 ft^2 of surface area per lineal foot of length. An observation port (vertical pipe) is installed in every other trench to monitor the water level in the trench. Each disposal bed is sized to accept up to 16,000 gpd at a hydraulic loading rate of one gallon per square foot per day. The disposal beds are rotated for efficiency and monitored through a flow meter at the holding tank and the observation ports to visually inspect the degree of saturation.

Both the subsurface disposal beds and spray irrigation areas are located on the treatment plant site. Ground water is monitored by extracting samples from the three wells located on the plant site. Table 13 provides a summary of the key attributes for the effluent holding and disposal facilities.

Table 13. Design and Operating Criteria of Effluent Holding and Disposal.

Effluent Holding		
Storage Tank	Number	1
	Volume	262,500 gallons
Effluent Disposal		
Spray Irrigation System	Area	25 acres
	Design Application Rate	1.8 inches / week
	Recommended Application Rate	36 inches/yr[a]
Percolation Bed	Number	11
	Application Rate	1 gpd/sf of bed trench area
	Disposal Capacity	16,000 gpd per bed

a. Based on agronomic rates.

Evaluation of Existing Facilities

Hydraulic, process and operational capacities for the existing facilities were determined to identify the capacity bottlenecks and improvements needed to accommodate future flows. The evaluations described below assume that all wastewater will be conveyed, treated, stored, and disposed of using the existing facilities. Potential solutions for overcoming the capacity bottlenecks identified in this evaluation are discussed later in the report. The following are descriptions of the capacity analyses performed for this task:

- **Conveyance System Evaluation:** Hydraulic capacities of the existing lift stations were compared to the projected buildout flow at each station. In turn, these capacities were used to identify the lift station improvements needed to accommodate future flows.
- **Treatment Plant Assessment:** Process capacities of the existing treatment plant facilities were determined using a treatment plant mass balance model. Model results were compared to site-specific and standard design criteria and constraints.
- **Effluent Holding and Disposal Evaluation:** Capacities of the existing holding tank and effluent disposal facilities were developed based on previous capacity assessments and design criteria.

Collection System Evaluation

A hydraulic analysis of the collection system was prepared using a Microsoft Excel spreadsheet. Existing and future flows were distributed around the service area to estimate the flow in each part of the system. For flows within the existing Arnold service area, the existing flow contribution was assumed to be spread uniformly throughout the service area. Similarly, the infill development was expected to occur uniformly throughout the service area. The Millwoods subdivision was assumed to flow directly into the White Pines Interceptor. The Cedar Ridge development was assumed to flow directly to Lift Station 1. The Avery Expansion area was assumed to flow directly to the Avery Pump Station.

The boundaries of the 1984 assessment district were drawn in GIS over the County's parcel base layer. The measured service area was approximately 554 acres. The service area was divided into basins that flowed to major facilities. The northernmost basin, flowing to Lift Station 3, includes 163 acres. All flow from Lift Station 3 is pumped to the White Pines Interceptor, which eventually flows to Lift Station 2. An additional 299 acres contributes flow to the White Pines Interceptor upstream of Lift Station 2. All flow from Lift Station 2 is pumped to Lateral MM, which flows to the Lakemont Force Main and the plant. An additional 63 acres contributes flow to Lateral MM upstream of the plant. The final 29 acres flow to Lift Station 1, which pumps directly to the Arnold WWTP.

The flow to the Avery Pump Station was not calculated based on the acreage of the service area. The District has assigned 3 ESFUs to the Middle School and 30 ESFUs to the Safari Mobile Home Park. Using the standard values for estimating flows from Table 6, the 33 ESFUs correspond to:

- ADWF of 6,435 gpd.
- Annual average flow of 6,950 gpd.
- Peak hour flow of 20,850 gpd.

The estimated flow distribution and collection system evaluation results are shown in Table 14.

Table 14. Collection System Evaluation.

	Avery Lift Station	Lift Station 1	Lift Station 2	Lift Station 3
Contributing Area				
Acres	N/A	29	462	163
2005 Peak Hour Flow (gpd)	20,850	11,677	186,552	65,633
2025 Peak Hour Flow from Existing Area (gpd)	20,850	23,316	371,451	131,053
2025 Peak Hour Flow from Avery Expansion (gpd)	13,900	--	--	--
2025 Peak Hour Flow from Cedar Ridge (gpd)	--	134,573	--	--
2025 Peak Hour Flow from Millwoods (gpd)	--	--	111,829	--
Total 2025 Peak Hour Flow (gpd)	**34,750**	**157,889**	**483,280**	**131,053**
Firm Capacity (gpd)[a]	**158,400**	**57,600**	**396,000**	**116,640**

Notes:

2005 Peak Hour Flow for Arnold (gpd)	243,921
2005 Peak Hour Flow for Arnold excluding Avery (gpd)	223,071
Arnold Service Area (acres)	554
2025 Peak Hour Flow for Arnold (gpd)	466,269
2025 Peak Hour Flow for Arnold excluding Avery (gpd)	445,419

[a] Capacity with largest pump out of service in accordance with District standards.

N/A = Not Applicable.

The Avery Lift Station is projected to have adequate capacity through buildout. However, as shown in Table 14, the evaluation results indicate that Lift Stations 1, 2, and 3 will need to be upgraded to accommodate future flows based on the District standards.[4] The following is the recommended approach for expanding their capacities.

- **Lift Station 1:** This station is already scheduled to be replaced as part of the Cedar Ridge development. All flows currently routed to Lift Station 1 will be conveyed to the new Cedar Ridge Lift Station, which will in turn, pump all flows to the Lift Station 1 force main. It is recommended that the new Cedar Ridge Lift Station be designed based on a minimum firm capacity of 110 gpm.
- **Lift Station 2:** An assessment of the existing pump vault shows that this vault cannot accommodate large pumps. Therefore, this lift station will require replacement in the future by installing a package lift station adjacent to the existing, or demolishing the existing station and building a new lift station in its place. If a new station is installed, a

[4] Sewer lift stations shall be capable of providing the maximum design flow with the largest pumping unit out of service. Section 1108 of the *Calaveras County Water District Improvement Standards*, June 1997.

manhole can be used as a temporary pump vault during construction. It is recommended that the new lift station be designed based on a minimum firm capacity of about 340 gpm.

- **Lift Station 3:** The existing pump vaults at Lift Station 2 and 3 are identical with regard to wet well volume. However, Lift Station 3 currently has significantly smaller pumps than Lift Station 2. Based on this assessment, it is expected that the pumps in Lift Station 3 can be replaced with larger capacity units to accommodate future flows. It is recommended that the replacement pumps be designed based on a minimum firm capacity of about 95 gpm to accommodate 2025 flows.

Treatment Plant Evaluation

A mass balance model of the treatment plant was constructed using HDR's ENVision program. The model incorporates flows and pollutant loads (i.e., BOD and TSS) from both influent and internal recycle streams and calculates loading rates of individual unit processes to assess performance. ENVision provides the ability to calibrate each individual unit process based on historic operating data, or in the absence of operating data, typical performance values. The mass balance model was run for a total of eight scenarios: current and buildout average dry weather, average annual, peak month and maximum day. The ENVision mass balance output is included in Appendix D.

After the mass balance was constructed, loading conditions for each unit process were compared to the site-specific and standard design criteria developed for the Arnold WWTP. This comparison allows one to determine whether a unit process is under or over loaded compared to the design criteria.

Table 15 summarizes the base scenario loading conditions under various flow conditions for all major unit processes within the treatment plant. This table also contains a general description of each process along with the criterion or criteria which limit the overall capacity of each unit process. As shown in Table 15, all the key unit processes will require expansion to accommodate buildout conditions.

A site visit of the Arnold WWTP was conducted on November 12, 2004. The following operation and maintenance improvements were discussed during the visit. The need to:

- Add a dissolved oxygen control system in the oxidation ditch to minimize blower output and energy costs.
- Conduct a more thorough evaluation of the subsurface disposal beds and spray irrigation area during the wet weather season.

Effluent Holding and Disposal Evaluation

A summary of the effluent holding and disposal system evaluation is presented in Table 16 for the four buildout scenarios.

Table 15. Treatment Plant Capacity Assessment – Base Scenario.

Process	Unit	Description	Size or Capacity per Unit	Standard or Site Specific Design/Operating Criteria			Mass Balance Output (Current Conditions)			Mass Balance Output (Buildout Conditions) (Base Scenario)			Mass Balance Output (Buildout Conditions) (Scenario 1)			Mass Balance Output (Buildout Conditions) (Scenario 2)			Mass Balance Output (Buildout Conditions) (Scenario 3)		
				Criteria Description	Value	Units	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed
Headworks	**Screening**	1 Comminutor	626,000 gpd	Peak flow capacity	435	gpm	169	39	NO	526	121	YES	604	139	YES	536	123	YES	613	141	YES
	Flow Measurement	1 Parshall flume	3 inch throat	Peak flow capacity	540	gpm	169	31	NO	526	98	NO	604	112	YES	536	99	NO	613	134	YES
Secondary Treatment	**Oxidation Ditch**	1 Oval channel	180 ft x 12 ft x 11 ft deep Volume = 23,400 cf	Hydraulic Retention Time @ average dry weather flow	24	hours	51	47	NO	17	141	YES	15.4	156	YES	17.3	139	YES	15.2	158	YES
				Mixed Liquor Concentration @ peak month	6,000	mg/L	2,465	41	NO	4,710	79	NO	5,380	90	NO	4,900	82	NO	5,400	90	NO
	Clarifier	1 Circular tank	26 ft Diameter 10 ft side water depth Volume = 40,000 gallons	Hydraulic Loading Rate @ average dry weather flow	330	gpd/sf	150	45	NO	449	136	YES	514	156	YES	457	138	YES	522	158	YES
				Solids Loading Rate @ average dry weather flow	25	lbs/day/sf	4	16	NO	35.5	142	YES	46.5	186	YES	36.8	147	YES	48	192	YES
	RAS Pumps	2 Variable speed sludge pumps	125 gpm each	100% recycling rate @ average annual flow with 1 standby pump	125	gpm	56	45	NO	175	140	YES	201	161	YES	179	143	YES	204	163	YES
Filtration and Disinfection	**Effluent (Filter Feed Pumps**	2 Vertical turbine pumps	125 gpm each	Maximum day flow with 1 standby pump	125	gpm	97	77	NO	263	210	YES	302	242	YES	268	168	YES	307	246	YES
	Pressure Filters	2 Single Media Sand Filters	66 ft² media area per filter; 132 ft² total	Hydraulic loading rate @ average dry weather flow; both filters in service	2	gpm/sf	0.4	20	NO	1.23	62	NO	1.4	70	NO	1.25	63	NO	1.43	72	NO
				Hydraulic loading rate @ average dry weather flow; 1 standby filter (in backwash)	2	gpm/sf	0.8	40	NO	2.5	125	YES	2.8	140	YES	2.5	125	YES	2.86	144	YES
				Hydraulic loading rate @ maximum day; both filters in service	10	gpm/sf	0.7	7	NO	2.0	20	NO	2.3	23	NO	2.0	20	NO	2.3	23	NO
				Hydraulic loading rate @ maximum day; 1 standby filter (in backwash)	10	gpm/sf	1.5	15	NO	4.0	40	NO	4.6	46	NO	4.0	40	NO	4.6	46	NO
	Chlorination	Storage/Feed Tank	1 hypo vat; volume = 350 gal	Storage at Average Annual Flow with 15ppm Chlorine dose	14	days	35	40	NO	11	127	YES	10	127	YES	11	127	YES	9.5	147	YES
		Feed Pump	1 peristaltic pump; 30 gpd	Feed rate at Maximum Day and 15ppm Chlorine dose	30	gpd	17	57	NO	47	157	YES	54	181	YES	48	160	YES	55	183	YES

Process	Unit	Description	Size or Capacity per Unit	Standard or Site Specific Design/Operating Criteria			Mass Balance Output (Current Conditions)			Mass Balance Output (Buildout Conditions) (Base Scenario)			Mass Balance Output (Buildout Conditions) (Scenario 1)			Mass Balance Output (Buildout Conditions) (Scenario 2)			Mass Balance Output (Buildout Conditions) (Scenario 3)		
				Criteria Description	Value	Units	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed
	Chlorine Contact Time	Detention Time Through Pressure Filters	2 pressure filters, 2,500 gallons each	Hydraulic detention time at maximum day flow	30	minutes	52	58	NO	19	158	YES	17	176	YES	19	158	YES	16	188	YES
Sludge Treatment and Dewatering	**Aerobic Digester**	1 Aerobic Digester with 2 compartments	9,050 gallons per compartment ; 18,100 gallons total (at 10,000 mg/L)	Hydraulic retention time @ average annual flow	15	days	22	68	NO	7	214	YES	6.2	242	YES	7	214	YES	6.1	246	YES
	Supernatant Filtrate Pumps	2 Submersible non-clog pumps	50 gpm each	Average annual digester feed flow with 1 standby pump	50	gpm	< 1.0	2	NO	1.8	4	NO	2	4	NO	1.7	3	NO	2.0	4	NO
	Sludge Drying Beds	3 Sand beds	1,000 sf each	Solids loading rate @ average annual	25	lb/sf/yr	5	20	NO	10	40	NO	11	45	NO	10	40	NO	11.3	45	NO
	Belt Filter Press	0.7 meter unit	50 gpm	Hydraulic loading rate @ average annual	Operating Time	Hours per week	1.5	—	NO	3.0	—	NO	3.0	—	NO	3.0	—	NO	3.0	—	NO

Notes:

Scenario 1 includes existing service area plus Millwoods
Scenario 2 includes existing service area plus Avery
Scenario 3 includes existing service area plus Millwoods and Avery

Table 16. Effluent Storage and Disposal System Capacity Assessment.

Process	Unit	Description	Size or Capacity per Unit	Standard or Site Specific Design/Operating Criteria			Mass Balance Output (Current Conditions)			Mass Balance Output (Buildout Conditions) (Base Scenario)			Mass Balance Output (Buildout Conditions) (Scenario 1)			Mass Balance Output (Buildout Conditions) (Scenario 2)			Mass Balance Output (Buildout Conditions) (Scenario 3)		
				Criteria Description	Value	Units	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed	Value	Percent of Rated Capacity	Expansion Needed
Effluent Holding	**Effluent Holding Tank**	One 38 ft diameter tank	262,500 gallons	Storage Capacity @ average dry weather flow	1.0	days	3.2	31	NO	1.1	91	NO	1.0	98	NO	1.0	98	NO	1.0	100	NO
Effluent Disposal	**Disposal Beds**	11 subsurface disposal beds	Trench length total = 17,600 ft; 10 ft^2 per lineal ft of trench length	Disposal capacity of 16,000 gpd per bed @ average dry weather flow (wet season application only)	176,000 total	gpd	81,000	46	NO	252,000	143	YES	268,300	152	YES	238,100	135	YES	272,600	155	YES
				Disposal capacity of 16,000 gpd per bed @ average dry weather flow (year round application)	176,000 total	gpd	40,500	23	NO	126,000	72	NO	134,150	76	NO	119,050	68	NO	136,300	78	NO
				Percolation rate at wet season plant effluent in Water Balance	1.0	gallon/ day/ square foot of trench area	0.5	50	NO	1.5	150	YES	1.5	170	YES	1.5	150	YES	1.8	180	YES
	Spray Irrigation Area	Spray fields	25 acres total	Agronomic rates	36	Inches/ year	21.4	59	NO	67	186	YES	77	214	YES	68	188	YES	79	219	YES

Notes:

Scenario 1 includes existing service area plus Millwoods
Scenario 2 includes existing service area plus Avery
Scenario 3 includes existing service area plus Millwoods and Avery

The effluent holding tank was originally designed to provide a minimum hydraulic detention time of one day at the plant's design capacity of 170,000 gpd. This criterion will be used to determine whether additional effluent holding capacity is needed for buildout.

As previously described, the spray irrigation system relies on soil percolation to dispose of a portion of the treated effluent. Percolation is also the primary means of disposal for the subsurface disposal beds. Information pertaining to soil characteristics or percolation rates was not available. The capacity of the disposal beds and spray irrigation system has been previously assessed by District staff and consultants.[5] Although the studies were inconclusive, both consultants indicated that the District should consider expanding the disposal facilities only when influent flows approach the design flow of 170,000 gpd. Based on these recommendations and the absence of wet weather field testing results, the original design criteria for the percolation beds will be used to determine if additional land is necessary to accommodate the projected buildout flows. The rated capacity of the spray irrigation system is based on agronomic rates as described in Table 13. Copies of the water balances developed for this evaluation are in Appendix E.

Summary of Required Improvements

The following is a list of improvements needed to accommodate the buildout flows for the base scenario.

- **Collection System:** The following improvements are recommended based on the projected 2025 peak hour flows.
  - Lift Station 1: As part of the Cedar Ridge development, this lift station is already scheduled to be replaced. It is recommended that the station be designed to provide a minimum firm capacity[6] of 110 gpm.
  - Lift Station 2: Expand the existing or construct a new lift station to provide a minimum firm capacity of 350 gpm.
  - Lift Station 3: Replace pumps to provide a minimum firm capacity of 95 gpm.
- **Treatment Plant:** The following improvements are recommended based on projected buildout flows.
  - Septage Receiving: A new, stand alone, septage receiving station is recommended. The station should be equipped with an integral screen and grinder.
  - Headworks: Given that the existing septage receiving station requires replacement and the headworks require expansion, the District should consider installing a new headworks.

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[5] *Arnold Wastewater Treatment Plant Capacity* (West Yost & Associates, December 1990) and *Wastewater Treatment Plant Evaluation* (Kennedy/Jenks Consultants, November 2001).
[6] Pumping station capacity with largest pump out of service.

- Oxidation Ditch and Clarifier: An additional ditch and clarifier capacity is not necessarily required to accommodate the projected buildout conditions. However, these units will be over 30 years old when influent flows exceed the plant's rated capacity. Moreover, an additional ditch and clarifier should be added for redundancy and to allow the existing units to be taken out of service for routine maintenance.
- RAS Pumps: One additional pump is recommended to service the new clarifier. The configuration of one of the existing RAS pumps should be modified to serve as standby for both dedicated RAS pumps.
- Effluent (Filter Feed) Pumps: A minimum of two additional pumps are required for buildout based on a rated capacity of 125 gpm each.
- Effluent Filters: Additional filters are not needed to accommodate buildout. However, the District should assess whether replacement of these units is required based on their past performance.
- Disinfection: The contact time associated with the pressure filters is insufficient for the projected buildout flows. Therefore additional contact time is required. The District should consider installing UV disinfection to minimize the formation of disinfection byproducts.
- Aerobic Digester: The addition of one more 9,050 gallon compartment is recommended.
- Supernatant Filtrate Pumps: No additional capacity required.
- Sludge Drying Beds: No additional capacity required.
- Belt Filter Press: No additional capacity is required.

- **Effluent Holding:** An additional tank is not required based on providing storage equal to one day at buildout conditions. However, another tank may be necessary if the spray irrigation system and/or the disposal beds are expanded.
- **Disposal Beds and Spray Irrigation Fields:** The District owns an additional 40 acres of land immediately south of the existing disposal system that can be used for these improvements. The additional spray fields and percolation beds will not require the entire 40 acres. It is recommended that the extra ten acres of land be set aside to accommodate additional disposal beds or to expand the spray irrigation area in the future. Approximately 22 acres of additional spray fields are needed to accommodate buildout. In addition, six more percolation beds are needed.

Alternative Analyses

Alternative Analyses

An alternative analysis was prepared to determine the cost effectiveness for incorporating Millwoods into the Arnold service area. The following is a description of the analysis along with the key findings, results, and recommendations.

Incorporation of Millwoods Service Area

The District currently owns and operates both the Arnold and Millwoods sewer systems. As previously described, the District would like to consider connecting the Millwoods system to the Arnold system to centralize operation and maintenance requirements and reduce costs.

Millwoods Septic Tanks

The Millwoods septic tanks have been operating for the past thirteen years with varying degrees of success. The following is a summary of the problems associated with the existing Millwoods septic tanks and sewer system.

Odors

The original septic tanks are two-compartment tanks with concrete lids. Each tank serves two houses, except for condominium areas, where one tank serves three houses. Newer units have a tank serving each house. The lids on the original septic tanks do not seal properly which allows odors to escape from the septic tanks. To minimize odors, the concrete lids need to be replaced and fastened directly to the concrete tank.

Pipeline Plugging

Septic tank discharge pipelines are 1-1/2 inches in diameter. Due to their small diameter, these pipelines have plugged and subsequently overflowed onto residential property in the past. In addition, the existing check valves do not operate properly and require replacement.

The manufacturer's newer septic tank design does not use check valves and the tanks are equipped with 2-inch discharge pipelines. The District believes that increasing the existing discharge pipeline from 1-1/2 to 2-inches would help reduce plugging. The existing 1-1/2-inch discharge pipeline is located in the middle of the second tank, which greatly reduces the tank's capacity; therefore the discharge pipeline should be relocated to the top of the tank.

Solids

The District performed testing at the Millwoods Lift Station and measured a 2-ft sludge blanket at the bottom of the wet well. The original pumps installed at the lift station were designed for clean water applications. These pumps have been replaced with grinder-type pumps suitable for this application. However, the fact that a considerable amount of solids are being conveyed to the lift station and subsequently to the leachfield is problematic, since the leachfield will eventually plug due to solids accumulation and soil pore blockage.

Adding a settling basin for solids removal adjacent to the existing leachfield and routing the Millwoods septic tank effluent directly to the Arnold Sewer System are two viable alternatives for eliminating the problems associated with septic tank effluent solids. In either of these cases, screens would have to be installed in 39 of the existing septic tanks along with the previously mentioned septic tank improvements.

Alternative 1 – Install Settling Basin

One potential alternative would be to install a new settling basin for solids removal prior to leachfield disposal. The District would continue to operate and maintain the Millwoods Sewer System and leachfield if this alternative was selected. The following is a summary of the improvements associated with this alternative:

- **Install Septic Tank Screens:** Install screens (basket type) in the 39 septic tanks that do not presently have them to reduce solids carryover.
- **Replace the Existing Concrete Lids:** Install sealed risers that are connected directly to the concrete tanks. It is estimated that 23 of the existing septic tanks require this improvement.
- **Increase Septic Tank Discharge Pipeline to 2-inch:** Replace the existing 1-1/2-inch discharge lines with 2-inch piping and relocate the tank discharge pipe. For cost estimating purposes, it is assumed that 40 septic tanks require this modification.
- **Install Settling Basin at the Leachfield:** Provide solids removal prior to effluent disposal to reduce leachfield solids deposition and plugging.
- **Drill Monitoring Well:** A new monitoring well is required at the treatment plant site as the existing upstream monitoring well is dry.
- **Annual Operation and Maintenance Costs:** The total estimated operation and maintenance costs for the Millwoods Sewer System is approximately $20,000 per year.

Alternative 2 – Abandon Millwoods Treatment and Disposal Systems

A second alternative to consider is continuing to operate and maintain the Millwoods collection system and pump the septic tank effluent to the Arnold WWTP for subsequent treatment and disposal. Under this option, the District would no longer need to maintain the Millwoods leachfield. The following is a summary of the improvements associated with this alternative:

- **Install Septic Tank Screens:** Install screens (basket type) in the 39 septic tanks that do not presently have them to reduce solids carryover.
- **Replace the Existing Concrete Lids:** Install sealed risers that are connected directly to the concrete tanks. It is estimated that 23 of the existing septic tanks require this improvement.

- **Increase Septic Tank Discharge Pipeline to 2-inch:** Replace the existing 1-1/2-inch discharge lines with 2-inch piping and relocate the tank discharge pipe. For cost estimating purposes, it is assumed that 40 septic tanks require this modification.
- **Provide Additional Capacity at the Arnold WWTP and Alter Expansion Timeline:** Approximately 35,000 gpd of additional ADWF capacity will be required if septic tank effluent is conveyed from Millwoods to the Arnold WWTP. Based on the current service area, improvements to the Arnold sewer system will be required by year 2020. If Millwooods is added, the timeline for required improvements will occur earlier in year 2014.
- **Millwoods Tie-In to the Arnold Sewer System:** It is estimated that a new 4-inch pipeline, approximately 200 feet in length, will be required to accomplish this tie-in.
- **Annual Operation and Maintenance Costs:** It is estimated that the total operation and maintenance costs for the Millwoods system can be reduced from $20,000 to $6,000 per year.

Cost Comparison and Recommendations

Table 17 presents a summary of the estimated life cycle costs developed for this alternative analysis. As shown in the last row, installing a settling basin and continuing to operate Millwoods as a separate system has a significantly lower life cycle cost. It is estimated that this alternative represents approximately 65 percent of the costs associated with abandoning the Millwoods treatment and disposal systems. Based on this cost comparison, it is recommended that Millwoods continue to operate as a separate system.

Table 17. Incorporation of Millwoods Service Area Cost Comparison.

Cost Component	Estimated Costs ($)	
	Alternative 1 - Install Settling Basins	Alternative 2 – Abandon Millwoods Treatment and Disposal Systems
Septic Tank Screens	90,000	90,000
Replace Existing Concrete Lids	15,000	15,000
Increase and Modify Septic Tank Discharge Pipeline	65,000	65,000
Install Solids Removal at the Leachfield	65,000[a]	--
Drill New Monitoring Well	10,000	--
Provide Additional Capacity at the Arnold WWTP	--	380,000[b]
Millwoods and Arnold Connection	--	15,000
Subtotal A	245,000	565,000
Contingency (30 percent of Subtotal A)	75,000	170,000
Subtotal B (Estimate of Probable Construction Cost)	**320,000**	**735,000**
Regulatory Requirements and Documentation	--	5,000
Administration and Engineering (20 percent of Subtotal B)	65,000	145,000
Total Estimated Project Costs	**385,000**	**885,000**
Estimated Annual Operation and Maintenance Costs	20,000	6,000
Estimated Life Cycle Costs[c]	**615,000**	**955,000**

a Cost based on installing a 10,000 gallon concrete tank adjacent to the existing leachfield.

b Incremental costs for treatment plant expansion with and without Millwoods are not expected to be significant. However, the cost associated with moving the expansion timeline from 2020 to 2014 is estimated to be $380,000 based on an interest rate of six percent and the total estimated project cost of $2,185,000 for the treatment plant expansion.

c Life Cycle Costs based on total project costs and annual operation and maintenance costs. A 20-year time period and interest rate of six percent were used in the analysis.

Incorporation of Avery Commercial Area

As previously described, the Arnold WWTP currently receives a small amount of domestic sewage from the Avery Middle School and Safari Mobile Home Park. The District is considering expanding this service by providing sewer service to the Avery commercial area. It is estimated that this area would represent 22 ESFUs. To provide this service, the Avery sewer pipeline would have to be extended. It is estimated that this extension would cost approximately $470,000.[7]

Providing service to this area is not expected to significantly alter the costs or timeline requirements for Arnold sewer system improvements described later in this technical memorandum. In addition, the Avery force main and pumping station have adequate capacity to serve this expansion. Assuming that the Avery sewer pipeline expansion will be paid for by the Avery commercial area, connecting the Avery commercial area to the Arnold sewer system is attractive since it will provide added customers at no additional cost.

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[7] Costs obtained from the 2002 Preliminary Avery Sewer Line Cost Allocation provided by the District.

Recommended Improvements and Timeline

Recommended Improvements and Timeline

Recommended improvements for upgrading the wastewater collection, treatment, and effluent holding and disposal facilities were developed based on the results and information presented in this report. The recommended improvements and timeline requirements described in this section are for the Base Scenario. Improvements and timelines for the other scenarios are presented in Appendix F.

Cost Estimate Development

Costs shown in the tables presented in this section represent total project cost and include administration and engineering costs. Project costs are presented in terms of 2004 U.S dollars according to Engineering News Record's (ENR's) cost indexes, currently equal to 7,115 (see 20 City Construction Cost Index, 1913 = 100 base).

Construction costs are based on equipment costs obtained from equipment manufacturers, past project experience, and quantity and standard unit cost estimates. A 30 percent contingency is included to account for change orders and items not included in the cost breakdowns. Administration and engineering costs are based on 20 percent of the construction costs (with contingency).

Improvements and Project Phasing

Timeline requirements for specific improvements were based on evaluating projected influent flows and specific system capacities. No improvements are required for the collection system piping or the effluent holding and disposal facilities.

- **Collection System – Lift Station:**
  - Lift Station 1: This station is scheduled to be replaced as part of the Cedar Ridge development. All flows currently routed to Lift Station 1 will be conveyed to the new Cedar Ridge Lift Station, which will in turn, pump all flows to the Lift Station 1 force main. It is recommended that the new Cedar Ridge Lift Station be designed based on a minimum firm capacity of 110 gpm.
  - Lift Station 2: The existing lift station wet well cannot accommodate larger capacity pumps. Therefore, this lift station requires replacement to accommodate future flows. The lift station's capacity will be exceeded in year 2019. However, due to its critical location, this lift station should be replaced immediately. The new lift station should be designed to provide a minimum firm capacity of 350 gpm.
  - Lift Station 3: Higher capacity pumps can be installed in the existing wet well to accommodate future flows. The lift station's capacity will be exceeded in year 2020. Replacement pumps should be designed to provide a minimum firm capacity of 95 gpm.
- **Collection System – Septic Tank (Millwoods):** Improvements include installing septic tank screens, replacing the existing concrete lids, and replacing the existing discharge

piping with larger diameter pipe. These improvements should be implemented immediately to minimize odors and maintenance requirements associated with the septic tanks located in the Millwoods service area.

It is recommended that the Arnold and Millwoods service areas be maintained and operated separately. Therefore, the total estimated project cost associated with the Millwoods improvements of $385,000 was not included in the cost estimates presented later in this report (i.e., Table 18 and Table 19).

- **Treatment Plant:** Two phases of improvements are required for the treatment plant. The following is a summary of the major improvements for both phases:
  - Immediate Improvements:
    1. Secondary Clarifier: A second clarifier is needed for redundancy and to allow the existing unit to be taken out of service for routine maintenance. The installation of one additional Return Activated Sludge (RAS) pump will be required to serve the new clarifier. The configuration of one of the existing RAS pumps should be modified to serve as standby for both dedicated RAS pumps.
    2. Dissolved Oxygen (DO) Control System: The addition of an automatic DO control system is recommended to minimize blower output and energy costs.
    3. Effluent (Filter Feed) Pumps: A minimum of two additional pumps are required for buildout. One additional effluent pump will be required by 2008. The second effluent pump is needed after 2060 and has been added to the Phase II improvements.
    4. Effluent Disposal Evaluation: The capacity of the disposal beds and spray irrigation system has been assessed by District staff and consultants in the past. However, these assessments were conducted during the dry season and were inconclusive. A more thorough evaluation of the disposal beds and spray irrigation area should be conducted during the wet weather season to assess their performance and capacity.
  - Plant Expansion: The capacity of the existing treatment plant is estimated to be exceeded by year 2020 and the influent ADWF is projected to approach 170,000 gpd. At that time, the following major unit processes will require expansion to accommodate future flows.
    1. Headworks and Septage Receiving Station: Install a new headworks and a new, stand alone septage receiving station. The new headworks should have a minimum peak flow capacity of 525 gpm.
    2. Oxidation Ditch: An additional oxidation ditch is not necessarily required to accommodate the projected buildout flows. However, the ditch will be over 35 years old when the plant expansion is completed and nearing the end of its useful life. Moreover, an additional ditch, similar in size to the existing, should be

added for redundancy and to allow the existing unit to be taken out of service for routine maintenance.

3. Effluent Pumps: A second additional effluent pump is projected to be required after 2060. It is recommended that this pump be added as part of this expansion phase.
4. Disinfection: Additional contact time is needed to accommodate future flows. At that time, it is recommended that the existing chlorine disinfection system be replaced with UV disinfection to minimize the formation of disinfection byproducts.
5. Aerobic Digester: One additional 9,050 gallon compartment is recommended to serve flows through buildout.

- **Effluent Holding and Disposal:** Approximately 22 acres of additional spray field area and six percolation beds are required to serve buildout. These improvements should be in service by 2011 to accommodate the additional flows.

Figure 6 and Figure 7 show the recommended collection and treatment plant improvements and phasing requirements.

Phase I Improvements (Immediate Improvements)

A summary of the Phase I Improvements is shown in Table 18 along with estimated costs. As shown, the total estimated project cost for the Phase I Improvements is $1,190,000. It is recommended that these improvements be implemented immediately to improve operations and maintenance and provide adequate capacity to accommodate future flows.

Table 18. Phase I Improvements (Base Scenario)

Cost Component	Estimated Costs ($)[a]
Collection System	
Lift Station 1	60,000[b]
Lift Station 2	250,000
Treatment Plant	
Secondary Clarifier and RAS Pump	300,000
DO Control System	40,000
Effluent Pump	35,000
Site Piping	40,000
Effluent Disposal Evaluation	35,000
Subtotal A	760,000
Contingency (30 percent of Subtotal A)	230,000
Subtotal B[c]	990,000
Administration and Engineering (20 percent of Subtotal B)	200,000
Total Estimated Project Cost	**1,190,000**

a Estimated costs presented in terms of 2004 US dollars.

b Cost represents the District's contribution to this lift station and not the total estimated project cost.

c Estimate of probable construction cost.

Phase II Improvements

Approximately 22 acres of additional spray field area and six percolation beds are required to serve buildout. As previously described, these improvement should be in service no later than 2011, or when the ADWF reaches 130,000 gpd. The total estimated project cost for the Phase II improvements is $865.000, which includes an additional effluent holding tank similar in size to the existing.

Phase III Improvements

A summary of the Phase III Improvements is shown in Table 19 along with estimated costs. As shown, the total estimated project cost for the Phase III Improvements is $2,380,000. These improvements are needed to be in service by 2020 when the ADWF approaches 170,000 gpd. The total number of ESFUs served in 2020 is estimated to be 940. Once these improvements are completed, the sewer system will have adequate capacity through buildout.

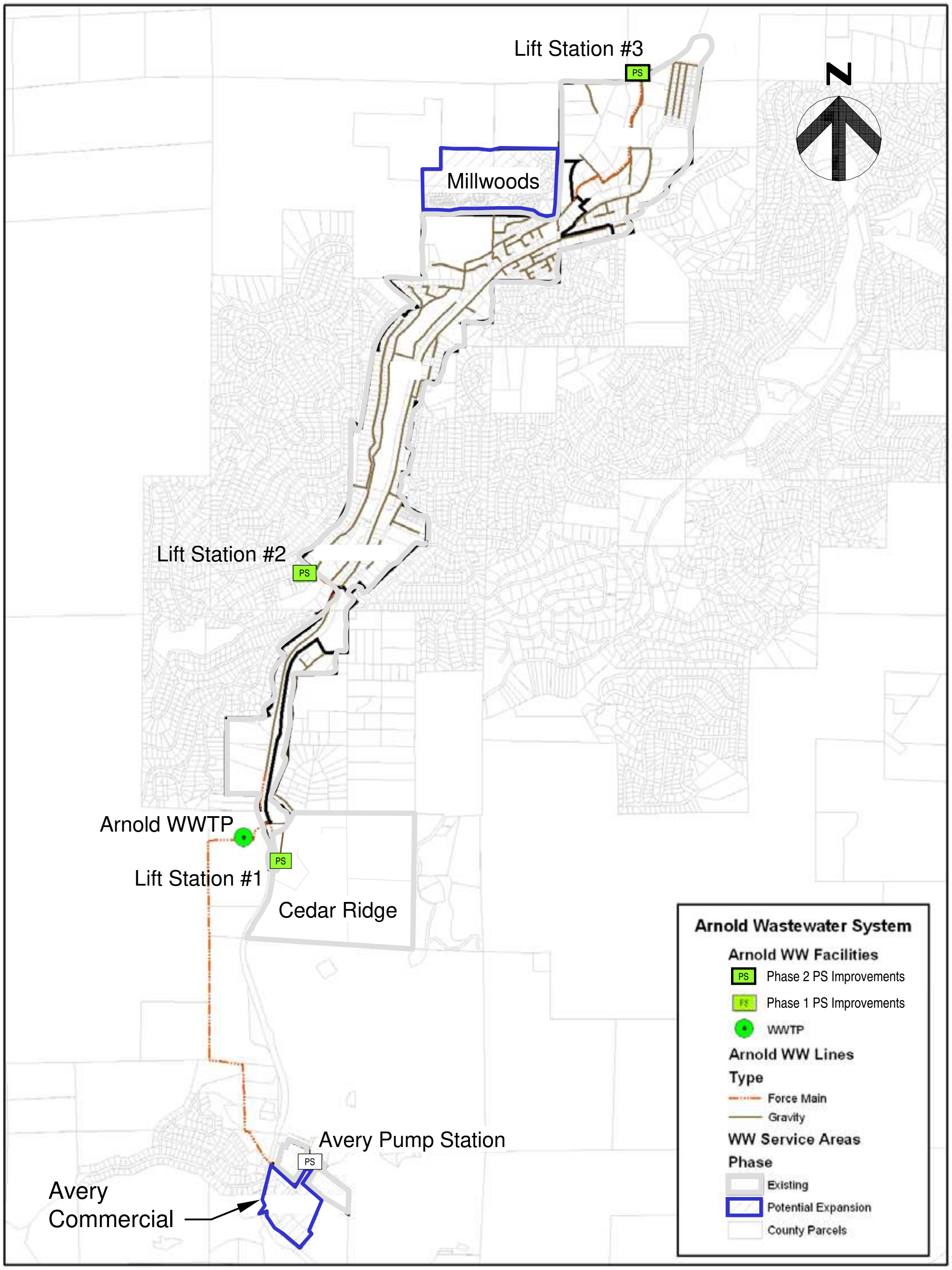
Table 19. Phase III Improvements (Base Scenario).

Cost Component	Estimated Costs ($)[a]
Collection System – Lift Station 3	125,000
Treatment Plant Expansion[b]	1,400,000
Subtotal A	1,525,000
Contingency (30 percent of Subtotal A)	460,000
Subtotal B[c]	1,985,000
Administration and Engineering (20 percent of Subtotal B)	395,000
Total Estimated Project Cost	**2,380,000**

a Estimated costs presented in 2004 US dollars.

b Treatment plant expansion includes headworks and septage receiving station, oxidation ditch, effluent pumping, disinfection, and aerobic digestion improvements.

c Estimate of probable construction cost.



Recommended Lift Station Improvements

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Figure 6

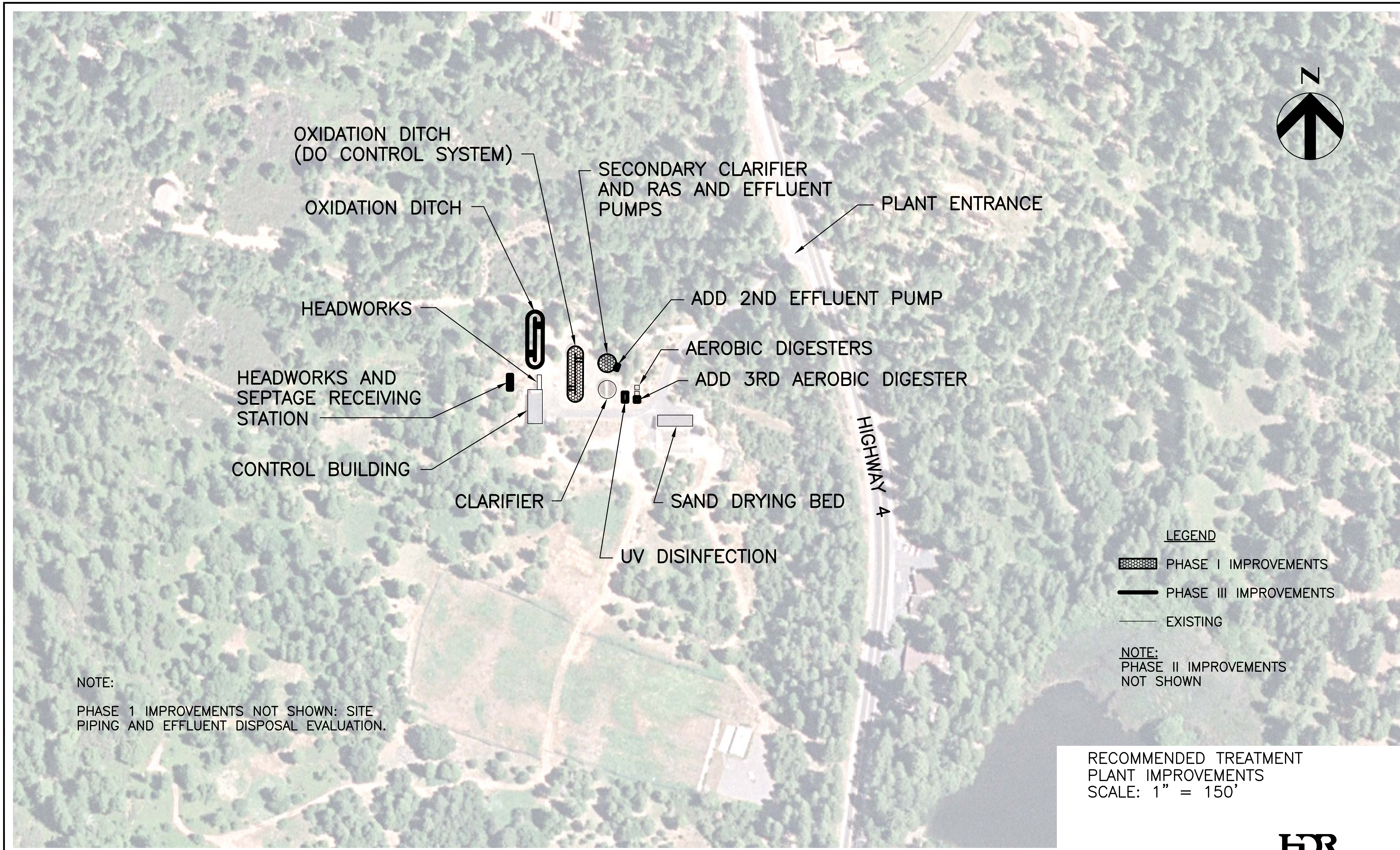

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Figure 7

Appendix A. Waste Discharge Requirements.

Appendix A. Waste Discharge Requirements.

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
CENTRAL VALLEY REGION

ORDER NO. 97-073

WASTE DISCHARGE REQUIREMENTS

FOR
CALAVERAS COUNTY WATER DISTRICT
ARNOLD WASTEWATER TREATMENT FACILITY
CALAVERAS COUNTY

The California Regional Water Quality Control Board, Central Valley Region, (hereafter Board) finds that:

1. Calaveras County Water District (hereafter Discharger) submitted a Report of Waste Discharge and a site evaluation report, dated 4 March 1997, for its wastewater treatment and disposal facility. The property, Assessor's Parcel No. 032-024-01, is owned by Calaveras County Water District.

2. Calaveras County Water District, Arnold Wastewater Treatment Facility is in Sections 6 & 7, T4N, R15E, MDB&M, with surface water drainage to the North Fork Stanislaus River via unnamed ephemeral stream, Mill Creek, and Hunter Reservoir as shown in Attachment A, which is attached hereto and part of the Order by reference. The site lies within the North Fork Stanislaus hydrologic unit/area/subarea No. 534.50, as depicted on interagency hydrologic maps prepared by the Department of Water Resources in August 1986.

3. Waste Discharge Requirements Order No. 85-015, adopted by the Board on 25 January 1985, prescribes requirements for a discharge from the Arnold Wastewater Treatment Facility to land.

4. Order No. 85-015 is neither adequate nor consistent with current plans and policies of the Board.

5. The Calaveras County Water District received a Clean Water Grant from the State Water Resources Control Board in 1984 to construct the Arnold Wastewater Treatment Facility.

6. Arnold Wastewater Treatment Facility is located four miles south of the community of Arnold. The treatment of domestic wastewater consists of an extended oxidation ditch (racetrack) followed by clarification, chlorination, sand filtrations, and holding tank. Additionally, two aerobic sludge digesters and two sludge drying beds are also incorporated into the facility. The treatment facility has a maximum design capacity of 0.170 million gallons per day (mgd) and the inflow presently averages 0.080 mgd. The source of the influent is primarily from domestic and light industries. The facility receives a small amount domestic wastewater from Avery Elementary School south of the facility.

7. The facility during the wet months utilizes 11 subsurface disposal beds. A single disposal bed is sized to accept up to 16,000 gallons per day (gpd) at a hydraulic loading rate of 1 gal/ft^2/day.

The disposal beds are rotated for efficiency. The disposal beds are frequently monitored through a flow meter at the storage tank and visually at observation ports for saturation. A pond in the higher elevation is also used during the dry months for storage of treated wastewater. Spray irrigation is utilized during the dry months for up to 25 acres of native grassland, shrubs, and trees. The sprinkler heads are observed weekly for clogging. Moreover, the subsurface disposal and spray irrigation are both used when conditions are acceptable. Potential impact of ground water is monitored through the three on-site wells.

8. The California Department of Health Services has established statewide reclamation criteria in Title 22, California Code of Regulations, Section 60301, et seq. (hereafter Title 22) for the use of reclaimed water has developed guidelines for specific uses. These uses are consistent with those guidelines.

9. The Board adopted a Water Quality Control Plan, Third Edition, for the Sacramento River Basin and the San Joaquin River Basin (hereafter Basin Plan), which contains water quality objectives for all waters of the Basin. These requirements implement the Basin Plan.

10. The beneficial uses of North Fork Stanislaus River are municipal, industrial, and agricultural supply; recreation; aesthetic enjoyment; navigation; fresh water replenishment; and preservation and enhancement of fish, wildlife, and other aquatic resources.

11. The beneficial uses of underlying ground water are domestic, industrial, and agricultural supply.

12. The action to revise waste discharge requirements for this facility is exempt from the provisions of the California Environmental Quality Act (CEQA), in accordance with Title 14, California Code of Regulations (CCR), Section 15301.

13. This discharge is exempt from the requirements of Title 23, CCR, Section 2510, et seq. (hereafter Chapter 15). The exemption, pursuant to Section 2511(b), is based on the following:

    a. The Board is issuing waste discharge requirements, and
    b. The discharge complies with the Basin Plan, and
    c. The wastewater does not need to be managed according to 22 CCR, Division 4.5, Chapter 11, as a hazardous waste.

14. The Board has notified the Discharger and interested agencies and persons of its intent to prescribe waste discharge requirements for this discharge and has provided them with an opportunity for a public hearing and an opportunity to submit their written views and recommendations.

15. The Board, in a public meeting, heard and considered all comments pertaining to the discharge.

IT IS HEREBY ORDERED that Order No. 85-015 is rescinded and Calaveras County Water District, Arnold Wastewater Treatment Facility, its agents, successors, and assigns, in order to meet the provisions contained in Division 7 of the California Water Code and regulations adopted thereunder, shall comply with the following:

A. Discharge Prohibitions:

1. Discharge of wastes to surface waters or surface water drainage courses is prohibited.
2. Bypass or overflow of untreated or partially treated waste is prohibited.
3. Discharge of waste classified as 'hazardous' or 'designated', as defined in Sections 2521(a) and 2522(a) of Chapter 15, is prohibited.

B. Discharge Specifications:

1. The monthly average dry weather discharge flow shall not exceed 0.170 million gallons/day.
2. Objectionable odors originating at this facility shall not be perceivable beyond the limits of the wastewater treatment and disposal areas.
3. The treatment facilities shall be designed, constructed, operated, and maintained to prevent inundation or washout due to floods with a 100-year return frequency.
4. The following constituent limitations shall apply to wastewater discharge to land:

Constituents	Unit	Monthly Average	Monthly Maximum
BOD_5 [1]	mg/l	40	80
Settleable Solids	ml/l	0.5	1.0

[1] 5-day, 20°C Biochemical Oxygen Demand

C. Reclamation Specifications:

1. Public contact with the reclaimed water shall be precluded through such means as fences, signs, and other acceptable alternatives.

2. Area irrigated with reclaimed water shall be managed to prevent breeding of mosquitos. More specifically,

   a. Tail water must be returned and all applied irrigation water must infiltrate completely within 48-hour period.

   b. Ditches not serving as wildlife habitat should be maintained free of emergent, marginal, and floating vegetation.

   c. Low-pressure and unpressurized pipelines and ditches accessible to mosquitos shall not be used to store reclaimed water

3. Reclaimed water for irrigation shall be managed to minimize erosion, runoff, and movement of aerosols from the disposal area.

4. Direct or windblown spray shall be confined to the designated reclamation area and prevent from contacting drinking water facilities.

5. The Discharger may not spray irrigate effluent during periods of precipitation and for at least 24 hours after cessation of precipitation, or when winds exceed 30 mph.

6. Application of reclaimed wastewater to the reclamation area shall be at reasonable rates considering the crop, soil, climate, and irrigation management system. The nutrient loading of the reclamation area, including the nutritive value of organic and chemical fertilizers and of the reclaimed water, shall not exceed the crop demand.

7. The effluent from the chlorination facility shall not exceed the following limits:

Constituent	Units	Monthly Average	Monthly Median	Daily Maximum
Total Coliform Organisms*	MPN/100 ml	----	23	240

* The limits are established under CCR, Title 22, Division 4, Chapter 3

8. There shall be no irrigation or impoundment of reclaimed water within 500 feet of any domestic water well or within 100 feet of any irrigation well unless it is demonstrated to the satisfaction of the Executive Officer that less distance is justified.

9. Storm water runoff from the irrigation field shall not be discharged to any surface water drainage course within 48-hours of the last application of reclaimed water.

D. Sludge Disposal:

1. Collected screenings, sludge, and other solids removed from liquid wastes shall be disposed of in a manner that is consistent with Chapter 15, Division 3, Title 23, of the California Code of Regulations and approved by the Executive Officer.

2. Any proposed change in sludge use or disposal practice from a previously approved practice shall be reported to the Executive Officer and U.S. Environmental Protection Agency (EPA) Regional Administrator at least 90 days in advance of the change.

3. Use and disposal of sewage shall comply with existing Federal and State laws and regulations, including permitting requirements and technical standards included in 40 CFR 503.

   If the State Water Resources Control Board and the Regional Water Quality Control Boards are given the authority to implement regulations contained in 40 CFR 503, this Order may be reopened to incorporate appropriate time schedules and technical standards. The Discharger must comply with the standards and time schedules contained in 40 CFR 503 whether or not they have been incorporated into this Order.

4. **Thirty days** after the adoption of this Order, the Discharger shall submit a sludge disposal plan describing the annual volume of sludge generated by the plant and specifying the disposal practices.

E. Ground Water Limitations:

The discharge shall not cause underlying ground water to:

1. Be degraded.

2. Contain chemicals, heavy metals, or trace elements in concentrations that adversely affect beneficial uses or exceed maximum contaminant levels specified in 22 CCR, Division 4, Chapter 15.

3. Exceed a most probable number of total coliform organisms of 2.2/100 ml over any seven-day period.

4. Exceed concentrations of radionuclides specified in 22 CCR, Division 4, Chapter 15.

5. Contain taste or odor-producing substances in concentrations that cause nuisance or adversely affect beneficial uses.

6. Contain concentrations of chemical constituents in amounts that adversely affect agricultural use.

F. Provisions:

1. The Discharger shall comply with the Monitoring and Reporting Program No. 97-073, which is part of this Order, and any revisions thereto as ordered by the Executive Officer.

2. The Discharger shall comply with the "Standard Provisions and Reporting Requirements for Waste Discharge Requirements", dated 1 March 1991, which are attached hereto and by reference a part of this Order. This attachment and its individual paragraphs are commonly referenced as "Standard Provision(s)."

3. In the event of any change in control or ownership of land or waste discharge facilities described herein, the Discharger shall notify the succeeding owner or operator of the existence of this Order by letter, a copy of which shall be immediately forwarded to this office.

4. At least **90 days** prior to termination or expiration of any lease, contract, or agreement involving disposal or reclamation areas or off-site reuse of effluent, used to justify the capacity authorized herein and assure compliance with this Order, the Discharger shall notify the Board in writing of the situation and of what measures have been taken or are being taken to assure full compliance with this Order.

5. The Discharger must comply with all conditions of this Order, including timely submittal of technical and monitoring reports as directed by the Executive Officer. Violations may result in enforcement action, including Regional Board or court orders requiring corrective action or imposing civil monetary liability, or in revision or rescission of this Order.

6. A copy of this Order shall be kept at the discharge facility for reference by operating personnel. Key operating personnel shall be familiar with its contents.

7. If reclaimed water is used for construction purposes, it shall comply with the most current edition of "Guidelines for Use of Reclaimed Water for Construction Purposes". Other uses of reclaimed water not specifically authorized herein shall be subject to the approval of the Executive Officer and shall comply with 22 CCR, Division 4.

8. The Board will review this Order periodically and will revise requirements when necessary.

I, JAMES R. BENNETT, Interim Executive Officer, do hereby certify the foregoing is a full, true, and correct copy of an Order adopted by the California Regional Water Quality Control Board, Central Valley Region, on 25 April 1997.

Thomas R Pinkos

for JAMES R. BENNETT, Interim Executive Officer

IK:ik

CALIFORNIA REGIONAL WATER QUALITY CONTROL BOARD
CENTRAL VALLEY REGION

MONITORING AND REPORTING PROGRAM NO. 97-073

FOR
CALAVERAS COUNTY WATER DISTRICT
ARNOLD WASTEWATER TREATMENT FACILITY
CALAVERAS COUNTY

INFLUENT MONITORING

Constituents	Units	Type of Sample	Sampling Frequency
Flow	mgd	Meter	Cumulative

EFFLUENT MONITORING

Effluent samples shall be collected just prior to discharge to the disposal facility. Effluent samples should be representative of the volume and nature of the discharge. Time of collection of a grab sample shall be recorded. Effluent monitoring shall include at least the following:

Constituents	Units	Type of Sample	Sampling Frequency
20°C BOD_5 [1]	mg/l	Grab	Monthly
Suspended Matter	mg/l	Grab	Monthly
Settleable Matter	ml/l	Grab	Monthly
Specific Conductivity	μmhos/cm	Grab	Monthly
pH	pH Units	Grab	Monthly
Total Coliform Organisms [2]	MPN/lOO ml	Grab	Weekly [2]

[1] Biochemical Oxygen Demand, 5 days at 20°C

[2] Shall be monitored twice weekly when irrigating to golf course

3. If requested by staff, copies of laboratory analytical report(s); and

4. A calibration log verifying calibration of all monitoring instruments and devices used to fulfill the prescribed monitoring program.

B. Quarterly Monitoring Reports

Quarterly Monitoring Reports shall be submitted to the Regional Board by the **1st day of the second month** following the end of the quarter (i.e. the January-March quarterly report is due by May 1st). Monthly reports for the months of March, June, September, and December may be submitted as part of the Quarterly Monitoring Report, if desired. The Quarterly Report shall include the following:

1. A narrative description of all preparatory, monitoring, sampling, and analytical testing activities for the groundwater monitoring. The narrative shall be sufficiently detailed to verify compliance with the WDRs, this MRP, and the Standard Provisions and Reporting Requirements. The narrative shall be supported by field logs for each well documenting depth to groundwater; parameters measured before, during, and after purging; method of purging; calculation of the casing volume; and total volume of water purged.

2. Calculation of groundwater elevations, an assessment of the groundwater flow direction and gradient on the date of measurement, comparison to previous flow direction and gradient data, and discussion of seasonal trends, if any.

3. A narrative discussion of the analytical results for all media and locations monitored, including spatial and temporal trends, with reference to summary data tables, graphs, and appended analytical reports (as applicable).

4. A comparison of monitoring data to the discharge specifications, groundwater limitations and surface water limitations, and explanation of any violation of those requirements.

5. Summary data tables of historical and current water table elevations and analytical results.

6. A scaled map showing relevant structures and features of the facility, the locations of monitoring wells and other sampling stations, and groundwater elevation contours referenced to mean sea level datum.

7. Copies of laboratory analytical report(s).

C. Annual Report

An Annual Report shall be prepared as the fourth quarter monitoring report. The Annual Report will include all monitoring data required in the monthly/quarterly schedule. The Annual Report shall be submitted to the Regional Board by **1 February** each year. In addition to the data normally presented, the Annual Report shall include the following:

1. The contents of the regular Quarterly Monitoring Report for the last quarter of the year;
2. The results from annual monitoring of groundwater wells, water supply, and supplemental water supply;
3. If requested by staff, tabular and graphical summaries of all data collected during the year;
4. Data for monitoring and analysis performed on an annual basis (i.e., standard minerals and biosolids);
5. An evaluation of the performance of the wastewater treatment system, as well as a forecast of the flows anticipated in the next year;
6. An evaluation of the groundwater quality beneath the wastewater treatment facility;
7. A discussion of any data gaps and potential deficiencies/redundancies in the monitoring system or reporting program;
8. A discussion of compliance and the corrective actions taken, as well as any planned or proposed actions needed to bring the discharge into full compliance with the waste discharge requirements;
9. The results from any sludge monitoring required by the disposal facility;
10. Summary of information on the disposal of sludge and/or solid waste;
11. A forecast of influent flows, as described in Standard Provision No. E.4; and
12. A copy of the certification for each certified wastewater treatment plant operator working at the facility and a statement about whether the Discharger is in compliance with Title 23, CCR, Division 3, Chapter 26.

The Discharger shall implement the above monitoring program as of **1 March 2003**.

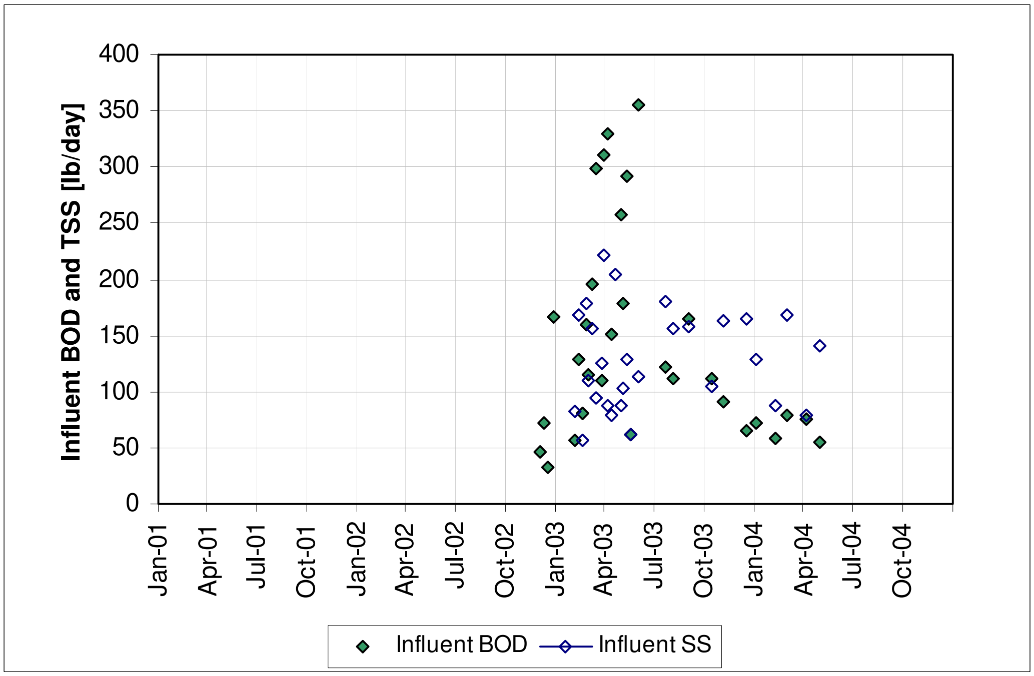
Ordered by: ______________________________
THOMAS R. PINKOS, Executive Officer

6 February 2003
(Date)

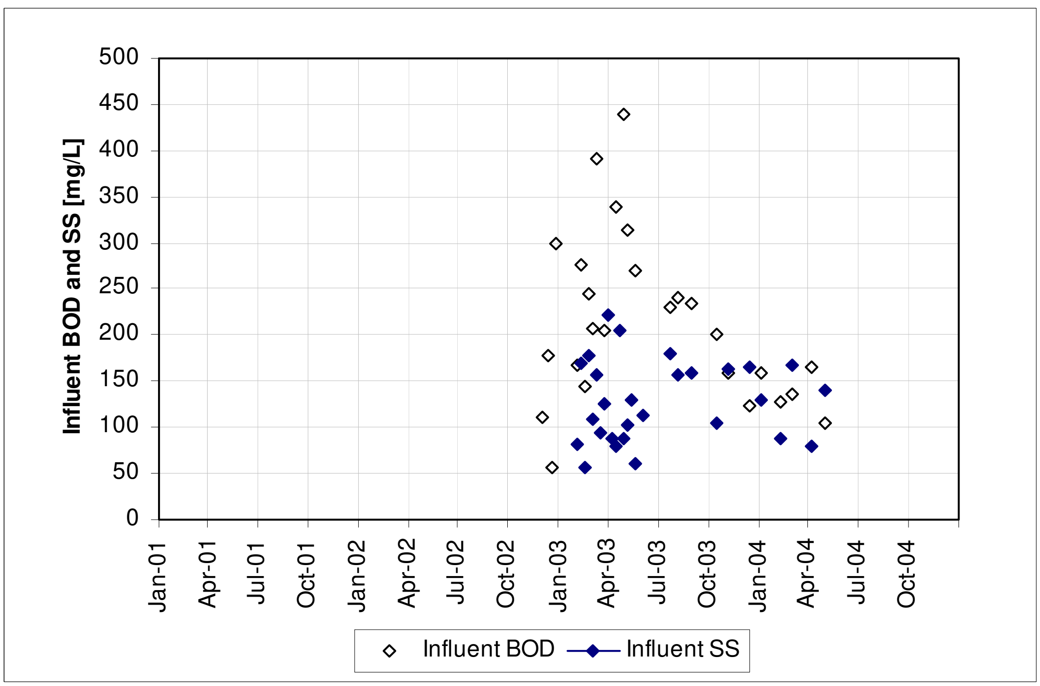
JSK: 2/6/03

Appendix B. Probability Analysis of Historical Plant and Flow Data.

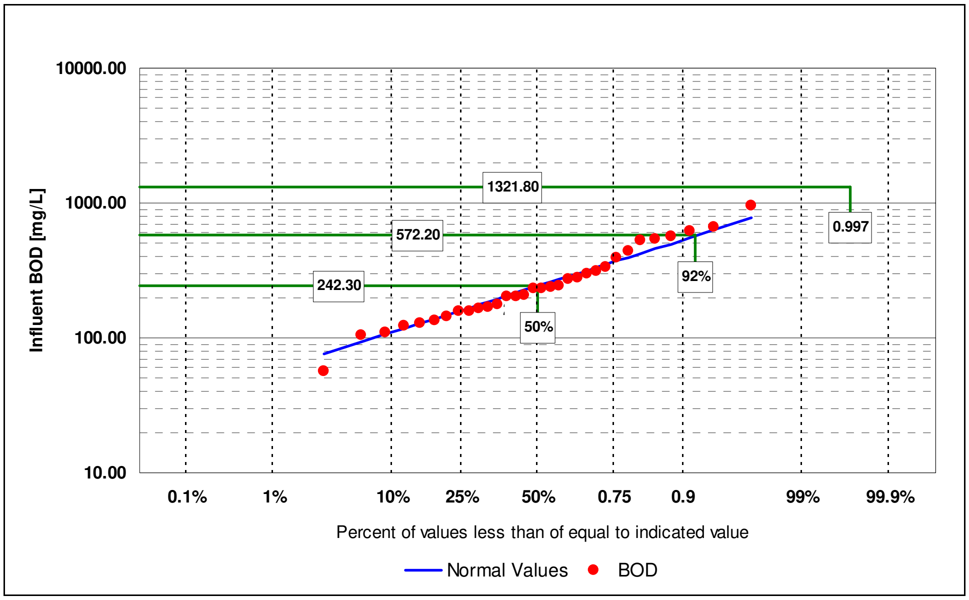
Appendix B. Probability Analysis of Historical Plant & Flow Data.



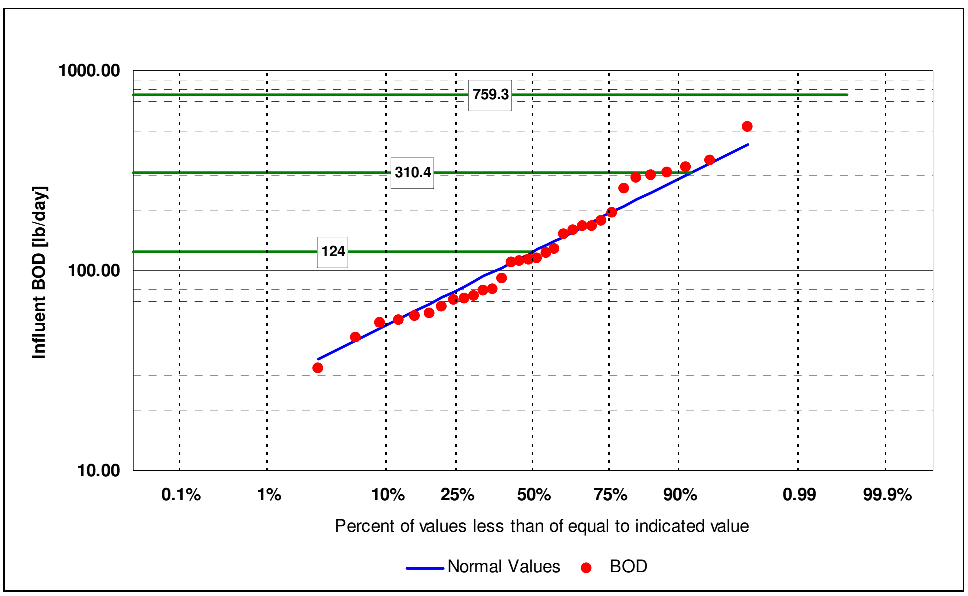
Influent BOD and TSS [lb/day]
400
350
300
250
200
150
100
50
0
Jan-01
Apr-01
Jul-01
Oct-01
Jan-02
Apr-02
Jul-02
Oct-02
Jan-03
Apr-03
Jul-03
Oct-03
Jan-04
Apr-04
Jul-04
Oct-04
Influent BOD
Influent SS



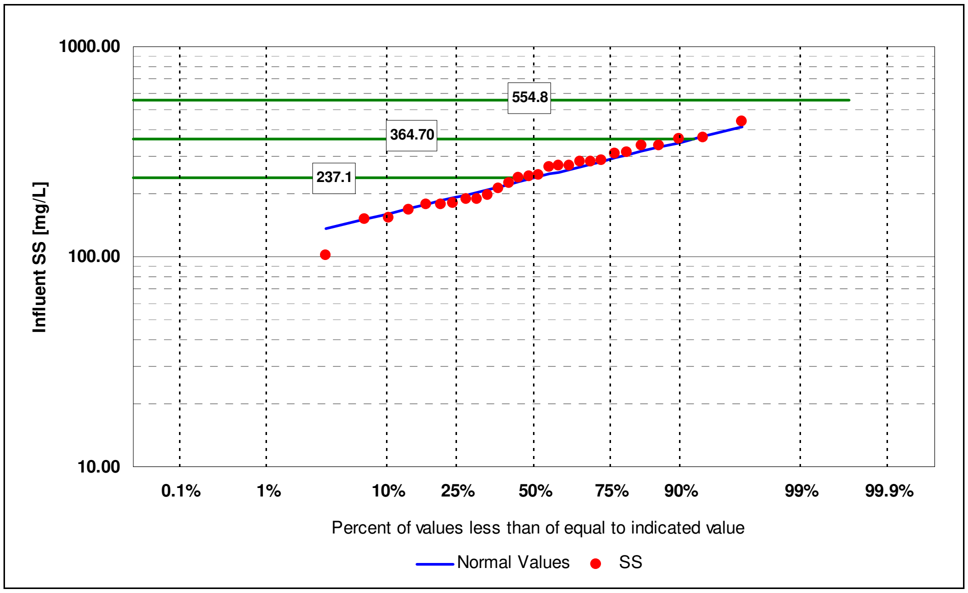
Influent BOD and SS [mg/L]
500
450
400
350
300
250
200
150
100
50
0
Jan-01
Apr-01
Jul-01
Oct-01
Jan-02
Apr-02
Jul-02
Oct-02
Jan-03
Apr-03
Jul-03
Oct-03
Jan-04
Apr-04
Jul-04
Oct-04
Influent BOD
Influent SS

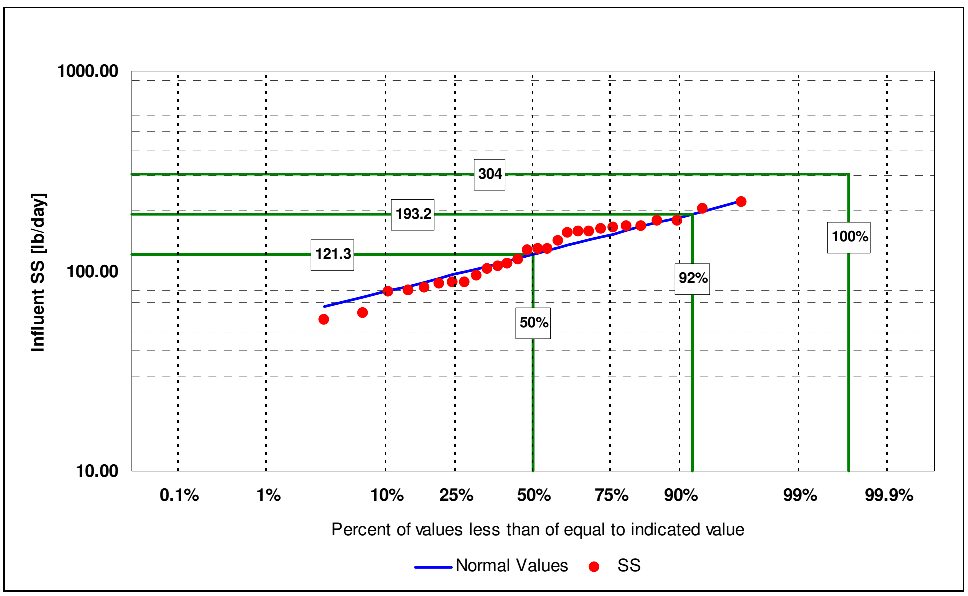


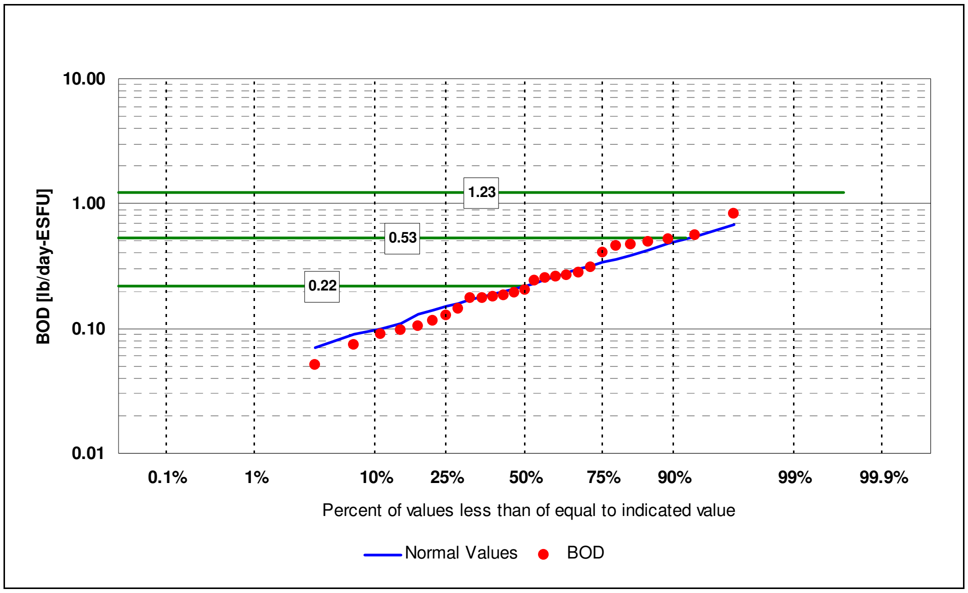
10000.00
1000.00
100.00
10.00
Influent BOD [mg/L]
1321.80
572.20
242.30
0.997
92%
50%
0.1%
1%
10%
25%
50%
0.75
0.9
99%
99.9%
Percent of values less than of equal to indicated value
Normal Values
BOD



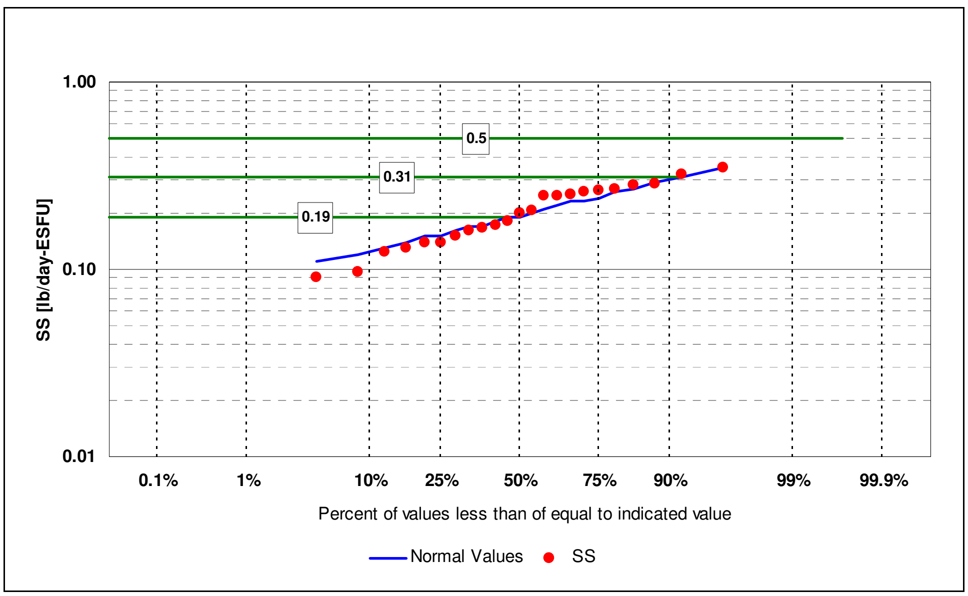
1000.00
100.00
10.00
Influent BOD [lb/day]
759.3
310.4
124
0.1%
1%
10%
25%
50%
75%
90%
0.99
99.9%
Percent of values less than of equal to indicated value
Normal Values
BOD


1000.00
554.8
364.70
237.1
100.00
10.00
Influent SS [mg/L]
0.1%
1%
10%
25%
50%
75%
90%
99%
99.9%
Percent of values less than of equal to indicated value
Normal Values
SS

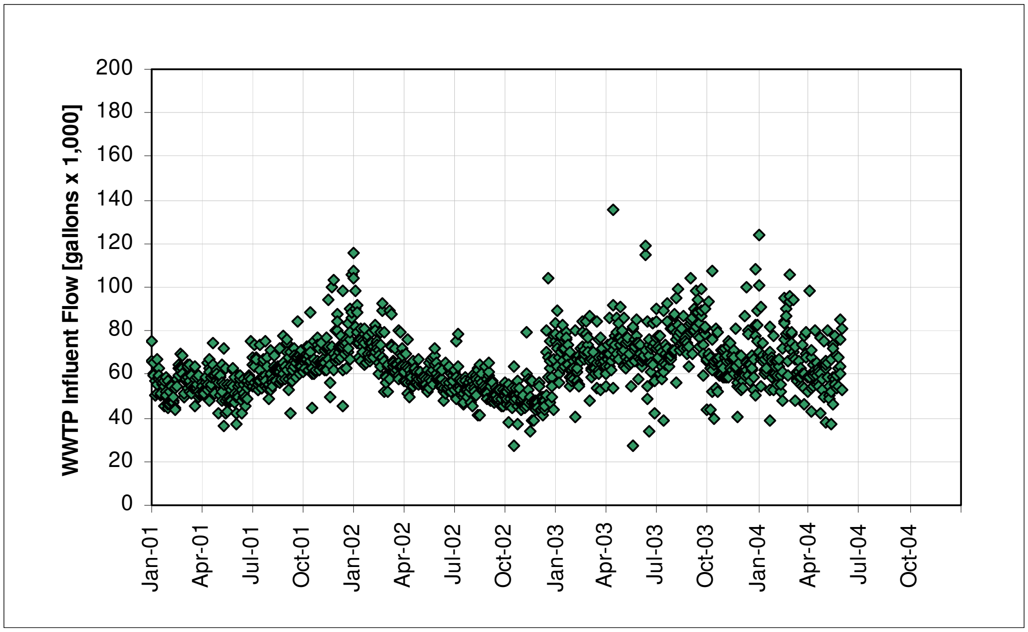

1000.00
304
193.2
121.3
100%
92%
50%
100.00
10.00
Influent SS [lb/day]
0.1%
1%
10%
25%
50%
75%
90%
99%
99.9%
Percent of values less than of equal to indicated value
Normal Values
SS



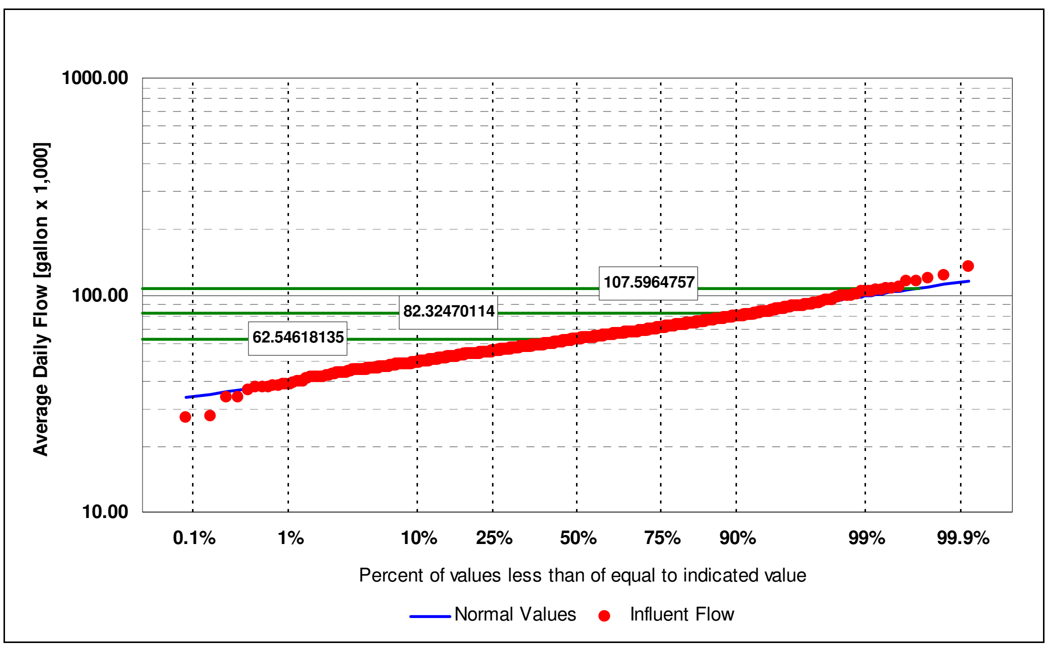
BOD [lb/day-ESFU]
10.00
1.00
0.10
0.01
1.23
0.53
0.22
0.1%
1%
10%
25%
50%
75%
90%
99%
99.9%
Percent of values less than of equal to indicated value
Normal Values
BOD



SS [lb/day-ESFU]
1.00
0.10
0.01
0.5
0.31
0.19
0.1%
1%
10%
25%
50%
75%
90%
99%
99.9%
Percent of values less than of equal to indicated value
Normal Values
SS



WWTP Influent Flow [gallons x 1,000]
200
180
160
140
120
100
80
60
40
20
0
Jan-01
Apr-01
Jul-01
Oct-01
Jan-02
Apr-02
Jul-02
Oct-02
Jan-03
Apr-03
Jul-03
Oct-03
Jan-04
Apr-04
Jul-04
Oct-04



Average Daily Flow [gallon x 1,000]
1000.00
100.00
10.00
107.5964757
82.32470114
62.54618135
0.1%
1%
10%
25%
50%
75%
90%
99%
99.9%
Percent of values less than of equal to indicated value
Normal Values
Influent Flow

Appendix C. Projected Flows and Loads.

Appendix C. Projected Flows and Loads.

Table C-1. Summary of Current and Buildout Wastewater Flows and Characteristics Base Scenario – Infill and Cedar Ridge

Parameter	Units	Wastewater Flows and Characteristics	
		Current (2004)	Buildout
Average Dry Weather Flow (ADWF)			
Connections	ESFUs	638	1,232
Unit Flow Factor	gpd per ESFU	118	195
Flow	**gpd**	**75,284**	**240,240**
Average Annual			
Flow Peaking Factor	ratio to ADWF	1.08	1.08
Flow	**gpd**	**81,307**	**259,459**
BOD per Capita	lb per ESFU	0.24	
BOD Load	**lb BOD/d**	**153**	
BOD Concentration (calculated value)	mg/L	226	226
TSS per Capita	lb per ESFU	0.20	
TSS Load	**lb TSS/d**	**128**	
TSS Concentration	mg/L	188	188
Peak Month			
I&I Rate	gallons per acre	56	56
Service Area	acres	360	740
I&I Flow Rate	gpd	20,160	41,440
Flow	**gpd**	**95,444**	**281,680**
BOD Peaking Factor	Ratio to Average Annual	2.0	
BOD Load	**lb BOD/d**	**306**	
BOD Concentration	mg/L	384	384
TSS Peaking Factor	Ratio to Average Annual	2.0	
TSS Load	**lb TSS/d**	**256**	
TSS Concentration	mg/L	322	322
Maximum Day			
Flow Peaking Factor	Ratio of Average Annual	1.72	1.5
Flow	**Gpd**	**139,848**	**389,189**
Peak Hour Flow			
Flow Peaking Factor	Ratio to Average Annual	3.0	3.0
Flow	**Gpd**	**243,921**	**778,378**

Table C-2. Summary of Current and Buildout Wastewater Flows and Characteristics Scenario 1 – Infill, Cedar Ridge, and Millwooods

Parameter	Units	Wastewater Flows and Characteristics	
		Current (2004)	Buildout
Average Dry Weather Flow (ADWF)			
Connections	ESFUs	638	1,409
Unit Flow Factor	gpd per ESFU	118	195
Flow	**Gpd**	**75,284**	**274,755**
Average Annual			
Flow Peaking Factor	ratio to ADWF	1.08	1.08
Flow	**Gpd**	**81,307**	**296,735**
BOD per Capita	lb per ESFU	0.24	
BOD Load	**lb BOD/d**	**153**	
BOD Concentration (calculated value)	mg/L	226	226
TSS per Capita	lb per ESFU	0.20	
TSS Load	**lb TSS/d**	**128**	
TSS Concentration	mg/L	188	188
Peak Month			
I&I Rate	gallons per acre	56	56
Service Area	acres	360	818
I&I Flow Rate	gpd	20,160	45,808
Flow	**gpd**	**95,444**	**320,563**
BOD Peaking Factor	Ratio to Average Annual	2.0	
BOD Load	**lb BOD/d**	**306**	
BOD Concentration	mg/L	384	384
TSS Peaking Factor	Ratio to Average Annual	2.0	
TSS Load	**lb TSS/d**	**256**	
TSS Concentration	mg/L	322	322
Maximum Day			
Flow Peaking Factor	Ratio of Average Annual	1.72	1.5
Flow	**gpd**	**139,848**	**445,103**
Peak Hour Flow			
Flow Peaking Factor	Ratio to Average Annual	3.0	3.0
Flow	**gpd**	**243,921**	**890,207**

Table C-3. Summary of Current and Buildout Wastewater Flows and Characteristics Scenario 2 – Infill, Cedar Ridge, and Avery

Parameter	Units	Wastewater Flows and Characteristics	
		Current (2004)	Buildout
Average Dry Weather Flow (ADWF)			
Connections	ESFUs	638	1,254
Unit Flow Factor	gpd per ESFU	118	195
Flow	**gpd**	**75,284**	**244,530**
Average Annual			
Flow Peaking Factor	ratio to ADWF	1.08	1.08
Flow	**gpd**	**81,307**	**264,092**
BOD per Capita	lb per ESFU	0.24	
BOD Load	**lb BOD/d**	**153**	
BOD Concentration (calculated value)	mg/L	226	226
TSS per Capita	lb per ESFU	0.20	
TSS Load	**lb TSS/d**	**128**	
TSS Concentration	mg/L	188	188
Peak Month			
I&I Rate	gallons per acre	56	56
Service Area	acres	360	769
I&I Flow Rate	gpd	20,160	43,064
Flow	**gpd**	**95,444**	**287,594**
BOD Peaking Factor	Ratio to Average Annual	2.0	
BOD Load	**lb BOD/d**	**306**	
BOD Concentration	mg/L	384	384
TSS Peaking Factor	Ratio to Average Annual	2.0	
TSS Load	**lb TSS/d**	**256**	
TSS Concentration	mg/L	322	322
Maximum Day			
Flow Peaking Factor	Ratio of Average Annual	1.72	1.5
Flow	**gpd**	**139,848**	**396,139**
Peak Hour Flow			
Flow Peaking Factor	Ratio to Average Annual	3.0	3.0
Flow	**gpd**	**243,921**	**792,278**

Table C-4. Summary of Current and Buildout Wastewater Flows and Characteristics Scenario 3 – Infill, Cedar Ridge, Millwoods, and Avery

Parameter	Units	Wastewater Flows and Characteristics	
		Current (2004)	Buildout
Average Dry Weather Flow (ADWF)			
Connections	ESFUs	638	1,431
Unit Flow Factor	gpd per ESFU	118	195
Flow	**gpd**	**75,284**	**279,045**
Average Annual			
Flow Peaking Factor	ratio to ADWF	1.08	1.08
Flow	**gpd**	**81,307**	**301,369**
BOD per Capita	lb per ESFU	0.24	
BOD Load	**lb BOD/d**	**153**	
BOD Concentration (calculated value)	mg/L	226	226
TSS per Capita	lb per ESFU	0.20	
TSS Load	**lb TSS/d**	**128**	
TSS Concentration	mg/L	188	188
Peak Month			
I&I Rate	gallons per acre	56	56
Service Area	acres	360	847
I&I Flow Rate	gpd	20,160	47,432
Flow	**gpd**	**95,444**	**326,477**
BOD Peaking Factor	Ratio to Average Annual	2.0	
BOD Load	**lb BOD/d**	**306**	
BOD Concentration	mg/L	384	384
TSS Peaking Factor	Ratio to Average Annual	2.0	
TSS Load	**lb TSS/d**	**256**	
TSS Concentration	mg/L	322	322
Maximum Day			
Flow Peaking Factor	Ratio of Average Annual	1.72	1.5
Flow	**gpd**	**139,848**	**452,053**
Peak Hour Flow			
Flow Peaking Factor	Ratio to Average Annual	3.0	3.0
Flow	**gpd**	**243,921**	**904,107**

Appendix D. Mass Balance.

Summary Report For
Arnold WWTP
CCWD

by HDR Engineering, Inc.

HDR

January 26, 2005

Prepared By jleng

Process/Loading	Units	Current Average Dry Weather Flow (Curr	Current Average Annual (Current)	Current Peak Month (Current)	Current Maximum Day (Current)	Base Scenario Average Dry Weather Flow (Buil	Base Scenario Average Annual (Buildout)	Base Scenario Peak Month (Buildout)	Base Scenario Maximum Day (Buildout)
Influent									
Flow	mgd	7.50E-02	8.10E-02	9.52E-02	0.1393	0.2338	0.2525	0.2724	0.3788
Biochemical oxygen demand concentration	mg/L	240	227	386	263	240	137	253	182
Total suspended solids concentration	mg/L	230	189	323	220	230	114	211	152
Percentage of total solids consisiting of vol	%	80	80	80	80	80	80	80	80
Ammonia concentration	mg/L	25	25	25	25	25	25	25	25
Total Kjeldahl Nitrogen	mg/L	35	35	35	35	35	35	35	35
Phosphorous concentration	mg/L	15	15	15	15	15	15	15	15
Alkalinity concentration	mg/L	230	230	230	230	230	230	230	230
Mixing Unit									
Oxidation Ditch									
Flow to AS	mgd	8.34E-02	8.85E-02	0.1035	0.1477	0.2422	0.2603	0.2808	0.3872
BOD in feed	mg/L	230.8	221.4	369.8	262.7	246.6	146.8	260.7	193.2
BOD load	lb/d	160.5	163.5	319.4	323.6	498.1	318.7	610.4	623.7
TKN in feed	mg/L	33.66	33.86	34.93	34.87	35.46	34.9	35.53	35.38
TKN load	lb/d	23.41	25	30.16	42.94	71.62	75.74	83.18	114.2
Number of basins	none	1	1	1	1	1	1	1	1
Length	ft	180	180	180	180	180	180	180	180
Width	ft	12	12	12	12	12	12	12	12
Depth	ft	11	11	11	11	11	11	11	11
Liquid temperature	C	20	20	20	20	20	20	20	20
Oxygen field transfer efficiency as percent	%/ft	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Dissolved oxygen setpoint	mg/L	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Basin volume (Total)	Mgal	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777
Sludge age (w/o clarifier)	days	25	25	25	25	25	25	25	25
Sludge age (w/ clarifier)	days	26.1	25.61	26.37	26.95	28.19	27.06	28.7	30.11
Hydraulic retention time	hr	51.15	48.18	41.2	28.88	17.61	16.39	15.19	11.02
MLSS	mg/L	1,348	1,275	2,467	2,513	4,148	2,505	4,710	4,842
F/M	lb BOD/lb VSS/d	8.34E-02	9.00E-02	9.09E-02	9.04E-02	8.41E-02	8.93E-02	9.10E-02	9.04E-02
Observed growth yield	lb VSS/lb BOD	0.4796	0.4443	0.4398	0.4423	0.4758	0.4481	0.4395	0.4423
Carb 02 required	lb O2/lb BOD	1.239	1.239	1.238	1.236	1.232	1.232	1.231	1.227
Total O2 required	lb O2/lb BOD	1.73	1.775	1.509	1.681	1.716	2.154	1.693	1.902
Oxygen uptake rate	mg/L/h	7.805	8.157	13.54	15.29	24.02	19.3	29.05	33.35
Diurnal OUR peak	mg/L/h	10.54	11.01	18.29	20.64	32.43	26.05	39.21	45.02
Oxygen required	lb/d	277.7	290.2	481.8	543.9	854.5	686.4	1,033	1,186
SOTE	%	15	15	15	15	15	15	15	15
Average required blower capacity	SCFM	2.109	2.205	3.661	4.132	6.492	5.215	7.85	9.013
Average blower energy	hp	0.1004	0.105	0.1743	0.1968	0.3091	0.2483	0.3738	0.4292
Diurnal peak blower capacity	SCFM	2.848	2.976	4.942	5.578	8.764	7.04	10.6	12.17
Peak blower energy	hp	0.1356	0.1417	0.2353	0.2656	0.4173	0.3353	0.5047	0.5794
Nitrogen in VSS	g N/g VSS	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02
Secondary Clarifier									
Number of secondary clarifiers	none	1	1	1	1	1	1	1	1
Diameter	ft	26	26	26	26	26	26	26	26
Depth	ft	10	10	10	10	10	10	10	10
TSS concentration in liquid effluent stream	mg/L	30	30	30	30	30	30	30	30
Maximum MLSS	mg/L	7,000	7,000	7,000	7,000	7,000	7,000	7,000	7,000
Clarifier area (Total)	sq ft	530.9	530.9	530.9	530.9	530.9	530.9	530.9	530.9
Clarifier volume (Total)	cu ft	5,309	5,309	5,309	5,309	5,309	5,309	5,309	5,309
Hydraulic surface loading rate	gpd/sq ft	149.6	164.7	187.6	270.8	448.8	486.8	521.6	722
Solids loading rate	lb/sq ft/d	3.972	2.084	9.027	13.11	35.51	13.83	46.74	66.25
Solids loading rate at set MLSS	lb/sq ft/d	3.972	2.084	9.027	13.11	35.51	13.83	46.74	66.25
Weir loading (single side)	gpd/ft	972.2	1,070	1,219	1,760	2,917	3,164	3,390	4,693
HRT (w/ recycle)	hr	5.079	9.162	4.091	2.869	1.749	2.713	1.509	1.094
HRT (w/o recycle)	hr	12	10.9	9.569	6.629	4	3.688	3.442	2.487
RAS concentration	mg/L	2,314	7,829	4,287	4,406	7,350	9,390	8,363	8,623
Max SVI allowed	mL/g	432.1	127.7	233.3	226.9	136.1	106.5	119.6	116
Filter									
Number of filtration units	none	2	2	2	2	2	2	2	2
Surface area per unit	sq ft	16	16	16	16	16	16	16	16
Depth	in	36	36	36	36	36	36	36	36
Filter run time	hr	24	24	24	24	24	24	24	24
Backwash rate	gpm/sq ft	15	15	15	15	15	15	15	15
Backwash duration	min	15	15	15	15	15	15	15	15
Area (total)	sq ft	32	32	32	32	32	32	32	32
Flow into filter	mgd	7.94E-02	8.74E-02	9.96E-02	0.1438	0.2383	0.2584	0.2769	0.3833
Hydraulic Loading (avg)	gpm/sq ft	1.723	1.897	2.162	3.12	5.171	5.608	6.009	8.318
Hydraulic Loading (1 off line)	gpm/sq ft	3.447	3.794	4.323	6.24	10.34	11.22	12.02	16.64
Solids loading	lb/sq ft/d	0.6209	0.6835	0.7788	1.124	1.863	2.021	2.165	2.997
Backwash Flow (avg)	gpm	5	5	5	5	5	5	5	5
Backwash Flow (instantaneous)	gpm	480	480	480	480	480	480	480	480
Effluent									
RAS Split									
Return activated sludge rate as a percenta	%	125	17.5	125	125	125	35	125	125
RAS stream	mgd	0.1042	1.55E-02	0.1294	0.1846	0.3027	9.11E-02	0.3509	0.4839
WAS stream	mgd	3.99E-03	1.11E-03	3.93E-03	3.90E-03	3.87E-03	1.82E-03	3.85E-03	3.84E-03
Aerobic Digester									
Number of digesters	none	2	2	2	2	2	2	2	2
Diameter	ft	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4
Depth	ft	10	10	10	10	10	10	10	10

Process/Loading	Units	Average Dry Weather Flow (Curr	Average Annual (Current)	Peak Month (Current)	Maximum Day (Current)	Average Dry Weather Flow (Buil	Average Annual (Buildout)	Peak Month (Buildout)	Maximum Day (Buildout)
Digester volume (total)	1000 cu ft	2.415	2.415	2.415	2.415	2.415	2.415	2.415	2.415
Percentage of VSS destroyed during diges	%	45	45	45	45	45	45	45	45
Precent NH4 in VSS	%	9	9	9	9	9	9	9	9
Precent phosphate in VSS	%	2	2	2	2	2	2	2	2
Supernatant flow as a percentage of influer	%	30	30	30	30	30	30	30	30
Detention time	days	4.529	16.24	4.598	4.639	4.673	9.908	4.697	4.71
Flow to disgester	mgd	-	-	-	-	-	-	-	-
VSS in inflow	lb VSS/d	-	-	-	-	-	-	-	-
TSS in inflow	lb TSS/d	3.99E-03	1.11E-03	3.93E-03	3.90E-03	3.87E-03	1.82E-03	3.85E-03	3.84E-03
VSS in outflow	lb VSS/d	74.18	69.85	134.9	137.5	228.3	137.3	257.7	265.1
TSS in outflow	lb TSS/d	76.99	72.69	140.4	143.1	237	142.8	268.2	275.8
Inflow TSS concentration	%	40	38.19	73.38	74.84	124.8	75.16	140.9	145
Outflow TSS concentration	%	40.81	40.48	76.97	78.51	131.5	79.77	149.6	153.9
Sludge Drying Beds									
Flow	gpm	1.939	0.5407	1.91	1.893	1.879	0.8864	1.87	1.865
BOD loading	lb/d	3.493	0.9741	3.441	3.41	3.386	1.597	3.368	3.359
TSS loading	lb/d	40.81	40.48	76.97	78.51	131.5	79.77	149.6	153.9

Process/Loading	Units	Current Average Dry Weather Flow (Curr	Current Average Annual (Current)	Current Peak Month (Current)	Current Maximum Day (Current)	Scenario 1 Average Dry Weather Flow (Buil	Scenario 1 Average Annual (Buildout)	Scenario 1 Peak Month (Buildout)	Scenario 1 Maximum Day (Buildout)
Influent									
Flow	mgd	7.50E-02	8.10E-02	9.52E-02	0.1393	0.2683	0.2898	0.3114	0.4347
Biochemical oxygen demand concentration	mg/L	240	227	386	263	240	137	253	182
Total suspended solids concentration	mg/L	230	189	323	220	230	114	211	152
Percentage of total solids consisiting of vol	%	80	80	80	80	80	80	80	80
Ammonia concentration	mg/L	25	25	25	25	25	25	25	25
Total Kjeldahl Nitrogen	mg/L	35	35	35	35	35	35	35	35
Phosphorous concentration	mg/L	15	15	15	15	15	15	15	15
Alkalinity concentration	mg/L	230	230	230	230	230	230	230	230
Mixing Unit									
Oxidation Ditch									
Flow to AS	mgd	8.34E-02	8.85E-02	0.1035	0.1477	0.2767	0.2975	0.3198	0.443
BOD in feed	mg/L	230.8	221.4	369.8	262.7	247.9	147.5	261.9	194
BOD load	lb/d	160.5	163.5	319.4	323.6	571.9	365.9	698.4	716.7
TKN in feed	mg/L	33.65	33.86	34.93	34.87	35.58	35.01	35.63	35.46
TKN load	lb/d	23.4	25	30.16	42.94	82.09	86.87	95.03	131
Number of basins	none	1	1	1	1	1	1	1	1
Length	ft	180	180	180	180	180	180	180	180
Width	ft	12	12	12	12	12	12	12	12
Depth	ft	11	11	11	11	11	11	11	11
Liquid temperature	C	20	20	20	20	20	20	20	20
Oxygen field transfer efficiency as percent	%/ft	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Dissolved oxygen setpoint	mg/L	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Basin volume (Total)	Mgal	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777
Sludge age (w/o clarifier)	days	25	25	25	25	25	25	25	25
Sludge age (w/ clarifier)	days	26.1	25.61	26.37	26.95	28.65	27.35	29.22	30.84
Hydraulic retention time	hr	51.15	48.18	41.2	28.88	15.42	14.34	13.34	9.628
MLSS	mg/L	1,348	1,275	2,467	2,513	4,757	2,873	5,382	5,552
F/M	lb BOD/lb VSS/d	8.34E-02	9.00E-02	9.09E-02	9.04E-02	8.42E-02	8.94E-02	9.11E-02	9.06E-02
Observed growth yield	lb VSS/lb BOD	0.4796	0.4443	0.4398	0.4423	0.4752	0.4474	0.4389	0.4414
Carb 02 required	lb O2/lb BOD	1.239	1.239	1.238	1.236	1.231	1.23	1.229	1.224
Total O2 required	lb O2/lb BOD	1.73	1.775	1.509	1.681	1.713	2.152	1.691	1.899
Oxygen uptake rate	mg/L/h	7.804	8.157	13.54	15.29	27.54	22.13	33.19	38.25
Diurnal OUR peak	mg/L/h	10.54	11.01	18.29	20.64	37.19	29.88	44.81	51.64
Oxygen required	lb/d	277.6	290.2	481.8	543.9	979.9	787.3	1,181	1,361
SOTE	%	15	15	15	15	15	15	15	15
Average required blower capacity	SCFM	2.109	2.205	3.661	4.132	7.444	5.981	8.97	10.34
Average blower energy	hp	0.1004	0.105	0.1743	0.1968	0.3545	0.2848	0.4272	0.4923
Diurnal peak blower capacity	SCFM	2.847	2.976	4.942	5.578	10.05	8.075	12.11	13.96
Peak blower energy	hp	0.1356	0.1417	0.2353	0.2656	0.4786	0.3845	0.5767	0.6645
Nitrogen in VSS	g N/g VSS	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02
Secondary Clarifier									
Number of secondary clarifiers	none	1	1	1	1	1	1	1	1
Diameter	ft	26	26	26	26	26	26	26	26
Depth	ft	10	10	10	10	10	10	10	10
TSS concentration in liquid effluent stream	mg/L	30	30	30	30	30	30	30	30
Maximum MLSS	mg/L	7,000	7,000	7,000	7,000	7,000	7,000	7,000	7,000
Clarifier area (Total)	sq ft	530.9	530.9	530.9	530.9	530.9	530.9	530.9	530.9
Clarifier volume (Total)	cu ft	5,309	5,309	5,309	5,309	5,309	5,309	5,309	5,309
Hydraulic surface loading rate	gpd/sq ft	149.6	164.7	187.6	270.8	513.9	557	595.1	827.2
Solids loading rate	lb/sq ft/d	3.972	2.084	9.027	13.11	46.52	18.12	60.83	86.94
Solids loading rate at set MLSS	lb/sq ft/d	3.972	2.084	9.027	13.11	46.52	18.12	60.83	86.94
Weir loading (single side)	gpd/ft	972.2	1,070	1,219	1,760	3,340	3,620	3,868	5,377
HRT (w/ recycle)	hr	5.079	9.162	4.091	2.869	1.531	2.373	1.325	0.9562
HRT (w/o recycle)	hr	12	10.9	9.569	6.629	3.494	3.223	3.017	2.17
RAS concentration	mg/L	2,314	7,829	4,287	4,406	8,445	10,810	9,571	9,902
Max SVI allowed	mL/g	432.1	127.7	233.3	226.9	118.4	92.54	104.5	101
Filter									
Number of filtration units	none	2	2	2	2	2	2	2	2
Surface area per unit	sq ft	16	16	16	16	16	16	16	16
Depth	in	36	36	36	36	36	36	36	36
Filter run time	hr	24	24	24	24	24	24	24	24
Backwash rate	gpm/sq ft	15	15	15	15	15	15	15	15
Backwash duration	min	15	15	15	15	15	15	15	15
Area (total)	sq ft	32	32	32	32	32	32	32	32
Flow into filter	mgd	7.94E-02	8.74E-02	9.96E-02	0.1438	0.2728	0.2957	0.316	0.4392
Hydraulic Loading (avg)	gpm/sq ft	1.723	1.897	2.162	3.12	5.921	6.417	6.857	9.531
Hydraulic Loading (1 off line)	gpm/sq ft	3.447	3.794	4.323	6.24	11.84	12.83	13.71	19.06
Solids loading	lb/sq ft/d	0.6209	0.6835	0.7788	1.124	2.133	2.312	2.47	3.434
Backwash Flow (avg)	gpm	5	5	5	5	5	5	5	5
Backwash Flow (instantaneous)	gpm	480	480	480	480	480	480	480	480
Effluent									
RAS Split									
Return activated sludge rate as a percenta	%	125	17.5	125	125	125	35	125	125
RAS stream	mgd	0.1042	1.55E-02	0.1294	0.1846	0.3458	0.1041	0.3997	0.5538
WAS stream	mgd	3.99E-03	1.11E-03	3.93E-03	3.90E-03	3.86E-03	1.82E-03	3.84E-03	3.83E-03
Aerobic Digester									
Number of digesters	none	2	2	2	2	2	2	2	2
Diameter	ft	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4
Depth	ft	10	10	10	10	10	10	10	10

Process/Loading	Units	Average Dry Weather Flow (Curr	Average Annual (Current)	Peak Month (Current)	Maximum Day (Current)	Average Dry Weather Flow (Buil	Average Annual (Buildout)	Peak Month (Buildout)	Maximum Day (Buildout)
Digester volume (total)	1000 cu ft	2.415	2.415	2.415	2.415	2.415	2.415	2.415	2.415
Percentage of VSS destroyed during diges	%	45	45	45	45	45	45	45	45
Precent NH4 in VSS	%	9	9	9	9	9	9	9	9
Precent phosphate in VSS	%	2	2	2	2	2	2	2	2
Supernatant flow as a percentage of influe	%	30	30	30	30	30	30	30	30
Detention time	days	4.529	16.24	4.598	4.639	4.682	9.944	4.705	4.716
Flow to disgester	mgd	-	-	-	-	-	-	-	-
VSS in inflow	lb VSS/d	-	-	-	-	-	-	-	-
TSS in inflow	lb TSS/d	3.99E-03	1.11E-03	3.93E-03	3.90E-03	3.86E-03	1.82E-03	3.84E-03	3.83E-03
VSS in outflow	lb VSS/d	74.18	69.85	134.9	137.5	261.8	157.4	294.4	304
TSS in outflow	lb TSS/d	76.99	72.69	140.4	143.1	271.7	163.8	306.5	316.3
Inflow TSS concentration	%	40	38.19	73.38	74.84	143.2	86.23	161.2	166.4
Outflow TSS concentration	%	40.81	40.48	76.97	78.51	151.2	91.65	171.3	176.8
Sludge Drying Beds									
Flow	gpm	1.939	0.5407	1.91	1.893	1.876	0.8831	1.867	1.862
BOD loading	lb/d	3.493	0.9741	3.441	3.41	3.379	1.591	3.363	3.355
TSS loading	lb/d	40.81	40.48	76.97	78.51	151.2	91.65	171.3	176.8

Process/Loading	Units	Current Average Dry Weather Flow (Curr	Current Average Annual (Current)	Current Peak Month (Current)	Current Maximum Day (Current)	Scenario 2 Average Dry Weather Flow (Buil	Scenario 2 Average Annual (Buildout)	Scenario 2 Peak Month (Buildout)	Scenario 2 Maximum Day (Buildout)
Influent									
Flow	mgd	7.50E-02	8.10E-02	9.52E-02	0.1393	0.2381	0.2571	0.2834	0.3857
Biochemical oxygen demand concentration	mg/L	240	227	386	263	240	137	253	182
Total suspended solids concentration	mg/L	230	189	323	220	230	114	211	152
Percentage of total solids consisiting of vol	%	80	80	80	80	80	80	80	80
Ammonia concentration	mg/L	25	25	25	25	25	25	25	25
Total Kjeldahl Nitrogen	mg/L	35	35	35	35	35	35	35	35
Phosphorous concentration	mg/L	15	15	15	15	15	15	15	15
Alkalinity concentration	mg/L	230	230	230	230	230	230	230	230
Mixing Unit									
Oxidation Ditch									
Flow to AS	mgd	8.34E-02	8.85E-02	0.1035	0.1477	0.2465	0.2649	0.2918	0.3941
BOD in feed	mg/L	230.8	221.4	369.8	262.7	246.8	146.9	261	193.3
BOD load	lb/d	160.5	163.5	319.4	323.6	507.3	324.5	635.2	635.2
TKN in feed	mg/L	33.65	33.86	34.93	34.87	35.48	34.91	35.56	35.39
TKN load	lb/d	23.41	25	30.16	42.94	72.92	77.13	86.52	116.3
Number of basins	none	1	1	1	1	1	1	1	1
Length	ft	180	180	180	180	180	180	180	180
Width	ft	12	12	12	12	12	12	12	12
Depth	ft	11	11	11	11	11	11	11	11
Liquid temperature	C	20	20	20	20	20	20	20	20
Oxygen field transfer efficiency as percent	%/ft	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Dissolved oxygen setpoint	mg/L	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Basin volume (Total)	Mgal	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777
Sludge age (w/o clarifier)	days	25	25	25	25	25	25	25	25
Sludge age (w/ clarifier)	days	26.1	25.61	26.37	26.95	28.25	27.1	28.85	30.2
Hydraulic retention time	hr	51.15	48.18	41.2	28.88	17.31	16.1	14.62	10.82
MLSS	mg/L	1,348	1,275	2,467	2,513	4,224	2,551	4,900	4,930
F/M	lb BOD/lb VSS/d	8.34E-02	9.00E-02	9.09E-02	9.04E-02	8.41E-02	8.93E-02	9.10E-02	9.05E-02
Observed growth yield	lb VSS/lb BOD	0.4796	0.4443	0.4398	0.4423	0.4757	0.448	0.4394	0.4422
Carb 02 required	lb O2/lb BOD	1.239	1.239	1.238	1.236	1.232	1.231	1.23	1.226
Total O2 required	lb O2/lb BOD	1.73	1.775	1.509	1.681	1.715	2.154	1.692	1.902
Oxygen uptake rate	mg/L/h	7.805	8.157	13.54	15.29	24.46	19.65	30.22	33.95
Diurnal OUR peak	mg/L/h	10.54	11.01	18.29	20.64	33.02	26.53	40.79	45.84
Oxygen required	lb/d	277.6	290.2	481.8	543.9	870.1	699	1,075	1,208
SOTE	%	15	15	15	15	15	15	15	15
Average required blower capacity	SCFM	2.109	2.205	3.661	4.132	6.61	5.31	8.166	9.177
Average blower energy	hp	0.1004	0.105	0.1743	0.1968	0.3148	0.2529	0.3889	0.437
Diurnal peak blower capacity	SCFM	2.848	2.976	4.942	5.578	8.924	7.169	11.02	12.39
Peak blower energy	hp	0.1356	0.1417	0.2353	0.2656	0.4249	0.3414	0.525	0.5899
Nitrogen in VSS	g N/g VSS	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02
Secondary Clarifier									
Number of secondary clarifiers	none	1	1	1	1	1	1	1	1
Diameter	ft	26	26	26	26	26	26	26	26
Depth	ft	10	10	10	10	10	10	10	10
TSS concentration in liquid effluent stream	mg/L	30	30	30	30	30	30	30	30
Maximum MLSS	mg/L	7,000	7,000	7,000	7,000	7,000	7,000	7,000	7,000
Clarifier area (Total)	sq ft	530.9	530.9	530.9	530.9	530.9	530.9	530.9	530.9
Clarifier volume (Total)	cu ft	5,309	5,309	5,309	5,309	5,309	5,309	5,309	5,309
Hydraulic surface loading rate	gpd/sq ft	149.6	164.7	187.6	270.8	456.9	495.5	542.3	735
Solids loading rate	lb/sq ft/d	3.972	2.084	9.027	13.11	36.8	14.33	50.53	68.66
Solids loading rate at set MLSS	lb/sq ft/d	3.972	2.084	9.027	13.11	36.8	14.33	50.53	68.66
Weir loading (single side)	gpd/ft	972.2	1,070	1,219	1,760	2,970	3,221	3,525	4,777
HRT (w/ recycle)	hr	5.079	9.162	4.091	2.869	1.719	2.665	1.452	1.075
HRT (w/o recycle)	hr	12	10.9	9.569	6.629	3.929	3.623	3.31	2.442
RAS concentration	mg/L	2,314	7,829	4,287	4,406	7,486	9,566	8,704	8,781
Max SVI allowed	mL/g	432.1	127.7	233.3	226.9	133.6	104.5	114.9	113.9
Filter									
Number of filtration units	none	2	2	2	2	2	2	2	2
Surface area per unit	sq ft	16	16	16	16	16	16	16	16
Depth	in	36	36	36	36	36	36	36	36
Filter run time	hr	24	24	24	24	24	24	24	24
Backwash rate	gpm/sq ft	15	15	15	15	15	15	15	15
Backwash duration	min	15	15	15	15	15	15	15	15
Area (total)	sq ft	32	32	32	32	32	32	32	32
Flow into filter	mgd	7.94E-02	8.74E-02	9.96E-02	0.1438	0.2426	0.2631	0.2879	0.3902
Hydraulic Loading (avg)	gpm/sq ft	1.723	1.897	2.162	3.12	5.265	5.709	6.248	8.468
Hydraulic Loading (1 off line)	gpm/sq ft	3.447	3.794	4.323	6.24	10.53	11.42	12.5	16.94
Solids loading	lb/sq ft/d	0.6209	0.6835	0.7788	1.124	1.897	2.057	2.251	3.051
Backwash Flow (avg)	gpm	5	5	5	5	5	5	5	5
Backwash Flow (instantaneous)	gpm	480	480	480	480	480	480	480	480
Effluent									
RAS Split									
Return activated sludge rate as a percenta	%	125	17.5	125	125	125	35	125	125
RAS stream	mgd	0.1042	1.55E-02	0.1294	0.1846	0.3081	9.27E-02	0.3647	0.4926
WAS stream	mgd	3.99E-03	1.11E-03	3.93E-03	3.90E-03	3.87E-03	1.82E-03	3.84E-03	3.84E-03
Aerobic Digester									
Number of digesters	none	2	2	2	2	2	2	2	2
Diameter	ft	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4
Depth	ft	10	10	10	10	10	10	10	10

Process/Loading	Units	Average Dry Weather Flow (Curr	Average Annual (Current)	Peak Month (Current)	Maximum Day (Current)	Average Dry Weather Flow (Buil	Average Annual (Buildout)	Peak Month (Buildout)	Maximum Day (Buildout)
Digester volume (total)	1000 cu ft	2.415	2.415	2.415	2.415	2.415	2.415	2.415	2.415
Percentage of VSS destroyed during diges	%	45	45	45	45	45	45	45	45
Precent NH4 in VSS	%	9	9	9	9	9	9	9	9
Precent phosphate in VSS	%	2	2	2	2	2	2	2	2
Supernatant flow as a percentage of influe	%	30	30	30	30	30	30	30	30
Detention time	days	4.529	16.24	4.598	4.639	4.674	9.913	4.699	4.71
Flow to disgester	mgd	-	-	-	-	-	-	-	-
VSS in inflow	lb VSS/d	-	-	-	-	-	-	-	-
TSS in inflow	lb TSS/d	3.99E-03	1.11E-03	3.93E-03	3.90E-03	3.87E-03	1.82E-03	3.84E-03	3.84E-03
VSS in outflow	lb VSS/d	74.18	69.85	134.9	137.5	232.5	139.8	268	269.9
TSS in outflow	lb TSS/d	76.99	72.69	140.4	143.1	241.3	145.4	279	280.8
Inflow TSS concentration	%	40	38.19	73.38	74.84	127.1	76.54	146.6	147.7
Outflow TSS concentration	%	40.81	40.48	76.97	78.51	134	81.25	155.7	156.7
Sludge Drying Beds									
Flow	gpm	1.939	0.5407	1.91	1.893	1.879	0.8859	1.869	1.864
BOD loading	lb/d	3.493	0.9741	3.441	3.41	3.385	1.596	3.367	3.359
TSS loading	lb/d	40.81	40.48	76.97	78.51	134	81.25	155.7	156.7

Process/Loading	Units	Current Average Dry Weather Flow (Curr	Current Average Annual (Current)	Current Peak Month (Current)	Current Maximum Day (Current)	Scenario 3 Average Dry Weather Flow (Buil	Scenario 3 Average Annual (Buildout)	Scenario 3 Peak Month (Buildout)	Scenario 3 Maximum Day (Buildout)
Influent									
Flow	mgd	7.50E-02	8.10E-02	9.52E-02	0.1393	0.2726	0.2944	0.3124	0.4416
Biochemical oxygen demand concentration	mg/L	240	227	386	263	240	137	253	182
Total suspended solids concentration	mg/L	230	189	323	220	230	114	211	152
Percentage of total solids consisiting of vol	%	80	80	80	80	80	80	80	80
Ammonia concentration	mg/L	25	25	25	25	25	25	25	25
Total Kjeldahl Nitrogen	mg/L	35	35	35	35	35	35	35	35
Phosphorous concentration	mg/L	15	15	15	15	15	15	15	15
Alkalinity concentration	mg/L	230	230	230	230	230	230	230	230
Mixing Unit									
Oxidation Ditch									
Flow to AS	mgd	8.34E-02	8.85E-02	0.1035	0.1477	0.281	0.3022	0.3208	0.45
BOD in feed	mg/L	230.8	221.4	369.8	262.7	248	147.5	261.9	194.1
BOD load	lb/d	160.5	163.5	319.4	323.6	581.1	371.8	700.6	728.2
TKN in feed	mg/L	33.65	33.86	34.93	34.87	35.59	35.02	35.63	35.47
TKN load	lb/d	23.41	25	30.16	42.94	83.39	88.25	95.32	133.1
Number of basins	none	1	1	1	1	1	1	1	1
Length	ft	180	180	180	180	180	180	180	180
Width	ft	12	12	12	12	12	12	12	12
Depth	ft	11	11	11	11	11	11	11	11
Liquid temperature	C	20	20	20	20	20	20	20	20
Oxygen field transfer efficiency as percent	%/ft	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Dissolved oxygen setpoint	mg/L	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
Basin volume (Total)	Mgal	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777	0.1777
Sludge age (w/o clarifier)	days	25	25	25	25	25	25	25	25
Sludge age (w/ clarifier)	days	26.1	25.61	26.37	26.95	28.71	27.39	29.23	30.93
Hydraulic retention time	hr	51.15	48.18	41.2	28.88	15.18	14.12	13.3	9.479
MLSS	mg/L	1,348	1,275	2,467	2,513	4,833	2,918	5,398	5,641
F/M	lb BOD/lb VSS/d	8.34E-02	9.00E-02	9.09E-02	9.04E-02	8.42E-02	8.94E-02	9.11E-02	9.06E-02
Observed growth yield	lb VSS/lb BOD	0.4796	0.4443	0.4398	0.4423	0.4751	0.4474	0.4389	0.4413
Carb 02 required	lb O2/lb BOD	1.239	1.239	1.238	1.236	1.231	1.23	1.229	1.224
Total O2 required	lb O2/lb BOD	1.73	1.775	1.509	1.681	1.713	2.151	1.691	1.898
Oxygen uptake rate	mg/L/h	7.805	8.157	13.54	15.29	27.98	22.48	33.29	38.86
Diurnal OUR peak	mg/L/h	10.54	11.01	18.29	20.64	37.78	30.35	44.95	52.46
Oxygen required	lb/d	277.6	290.2	481.8	543.9	995.4	799.8	1,184	1,382
SOTE	%	15	15	15	15	15	15	15	15
Average required blower capacity	SCFM	2.109	2.205	3.661	4.132	7.563	6.076	8.998	10.5
Average blower energy	hp	0.1004	0.105	0.1743	0.1968	0.3601	0.2894	0.4285	0.5001
Diurnal peak blower capacity	SCFM	2.848	2.976	4.942	5.578	10.21	8.203	12.15	14.18
Peak blower energy	hp	0.1356	0.1417	0.2353	0.2656	0.4862	0.3906	0.5785	0.6751
Nitrogen in VSS	g N/g VSS	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02	8.00E-02
Secondary Clarifier									
Number of secondary clarifiers	none	1	1	1	1	1	1	1	1
Diameter	ft	26	26	26	26	26	26	26	26
Depth	ft	10	10	10	10	10	10	10	10
TSS concentration in liquid effluent stream	mg/L	30	30	30	30	30	30	30	30
Maximum MLSS	mg/L	7,000	7,000	7,000	7,000	7,000	7,000	7,000	7,000
Clarifier area (Total)	sq ft	530.9	530.9	530.9	530.9	530.9	530.9	530.9	530.9
Clarifier volume (Total)	cu ft	5,309	5,309	5,309	5,309	5,309	5,309	5,309	5,309
Hydraulic surface loading rate	gpd/sq ft	149.6	164.7	187.6	270.8	521.9	565.7	596.9	840.3
Solids loading rate	lb/sq ft/d	3.972	2.084	9.027	13.11	47.99	18.7	61.2	89.7
Solids loading rate at set MLSS	lb/sq ft/d	3.972	2.084	9.027	13.11	47.99	18.7	61.2	89.7
Weir loading (single side)	gpd/ft	972.2	1,070	1,219	1,760	3,393	3,677	3,880	5,462
HRT (w/ recycle)	hr	5.079	9.162	4.091	2.869	1.508	2.337	1.321	0.9414
HRT (w/o recycle)	hr	12	10.9	9.569	6.629	3.44	3.173	3.007	2.136
RAS concentration	mg/L	2,314	7,829	4,287	4,406	8,582	10,980	9,601	10,060
Max SVI allowed	mL/g	432.1	127.7	233.3	226.9	116.5	91.06	104.2	99.4
Filter									
Number of filtration units	none	2	2	2	2	2	2	2	2
Surface area per unit	sq ft	16	16	16	16	16	16	16	16
Depth	in	36	36	36	36	36	36	36	36
Filter run time	hr	24	24	24	24	24	24	24	24
Backwash rate	gpm/sq ft	15	15	15	15	15	15	15	15
Backwash duration	min	15	15	15	15	15	15	15	15
Area (total)	sq ft	32	32	32	32	32	32	32	32
Flow into filter	mgd	7.94E-02	8.74E-02	9.96E-02	0.1438	0.2771	0.3003	0.3169	0.4461
Hydraulic Loading (avg)	gpm/sq ft	1.723	1.897	2.162	3.12	6.014	6.518	6.878	9.682
Hydraulic Loading (1 off line)	gpm/sq ft	3.447	3.794	4.323	6.24	12.03	13.04	13.76	19.36
Solids loading	lb/sq ft/d	0.6209	0.6835	0.7788	1.124	2.167	2.348	2.478	3.488
Backwash Flow (avg)	gpm	5	5	5	5	5	5	5	5
Backwash Flow (instantaneous)	gpm	480	480	480	480	480	480	480	480
Effluent									
RAS Split									
Return activated sludge rate as a percenta	%	125	17.5	125	125	125	35	125	125
RAS stream	mgd	0.1042	1.55E-02	0.1294	0.1846	0.3512	0.1058	0.401	0.5625
WAS stream	mgd	3.99E-03	1.11E-03	3.93E-03	3.90E-03	3.86E-03	1.82E-03	3.84E-03	3.83E-03
Aerobic Digester									
Number of digesters	none	2	2	2	2	2	2	2	2
Diameter	ft	12.4	12.4	12.4	12.4	12.4	12.4	12.4	12.4
Depth	ft	10	10	10	10	10	10	10	10

Process/Loading	Units	Average Dry Weather Flow (Curr	Average Annual (Current)	Peak Month (Current)	Maximum Day (Current)	Average Dry Weather Flow (Buil	Average Annual (Buildout)	Peak Month (Buildout)	Maximum Day (Buildout)
Digester volume (total)	1000 cu ft	2.415	2.415	2.415	2.415	2.415	2.415	2.415	2.415
Percentage of VSS destroyed during diges	%	45	45	45	45	45	45	45	45
Precent NH4 in VSS	%	9	9	9	9	9	9	9	9
Precent phosphate in VSS	%	2	2	2	2	2	2	2	2
Supernatant flow as a percentage of influe	%	30	30	30	30	30	30	30	30
Detention time	days	4.529	16.24	4.598	4.639	4.683	9.948	4.705	4.717
Flow to disgester	mgd	-	-	-	-	-	-	-	-
VSS in inflow	lb VSS/d	-	-	-	-	-	-	-	-
TSS in inflow	lb TSS/d	3.99E-03	1.11E-03	3.93E-03	3.90E-03	3.86E-03	1.82E-03	3.84E-03	3.83E-03
VSS in outflow	lb VSS/d	74.18	69.85	134.9	137.5	266	159.9	295.3	308.8
TSS in outflow	lb TSS/d	76.99	72.69	140.4	143.1	276.1	166.4	307.4	321.3
Inflow TSS concentration	%	40	38.19	73.38	74.84	145.5	87.6	161.7	169.1
Outflow TSS concentration	%	40.81	40.48	76.97	78.51	153.7	93.12	171.8	179.7
Sludge Drying Beds									
Flow	gpm	1.939	0.5407	1.91	1.893	1.875	0.8828	1.867	1.862
BOD loading	lb/d	3.493	0.9741	3.441	3.41	3.378	1.59	3.362	3.354
TSS loading	lb/d	40.81	40.48	76.97	78.51	153.7	93.12	171.8	179.7

Appendix E. Water Balances.

Table E1 - Water Balance (Current Conditions)
Arnold WWTP Facility - Water Balance

Month (1)	Days	EFFLUENT PRODUCTION ADWF gpd	ADWF gal/month	ADWF ac-ft/month (2)	I/I ac-ft/month (3)	Total Effluent ac-ft (4)	WET SEASON PERCOLATION Application Rate ac-ft/yr (5)	DRY SEASON IRRIGATION Application Rate ac-ft/yr (6)
Oct	31	75,284	2,333,804	7.16	0.79	8.0	4.0	4.0
Nov	30	75,284	2,258,520	6.93	1.47	8.4	8.4	0.0
Dec	31	75,284	2,333,804	7.16	1.92	9.1	9.1	0.0
Jan	31	75,284	2,333,804	7.16	1.92	9.1	9.1	0.0
Feb	28	75,284	2,107,952	6.47	1.43	7.9	7.9	0.0
Mar	31	75,284	2,333,804	7.16	1.46	8.6	8.6	0.0
Apr	30	75,284	2,258,520	6.93	0.96	7.9	3.9	3.9
May	31	75,284	2,333,804	7.16	0.53	7.7	0.0	7.7
Jun	30	75,284	2,258,520	6.93	0.19	7.1	0.0	7.1
Jul	31	75,284	2,333,804	7.16	0.13	7.3	0.0	7.3
Aug	31	75,284	2,333,804	7.16	0.20	7.4	0.0	7.4
Sep	30	75,284	2,258,520	6.93	0.32	7.3	0.0	7.3
Total				84.32	11.33	95.6	51.0	44.6

Average Dry Weather Flow, gal/d: **75,284** Current ADWF

DRY SEASON PERCOLATION

Irrigation Area (Dry Season), acres:	25
Maximum Application Rate (inches per year)	36.0
Irrigation Duration	April 15 through October 15
Number of Irrigation Days	183
Actual Application Rate (inches per year)	21.4
Over Irrigating ?	**No**

WET SEASON PERCOLATION

Percolation Area (sf trench area per bed)	16,000
Number of Percolation Beds	11
Maximum Percolation Rate (gallon/sf trench area day)	1.0
Number of Percolation Days	182
Applied Percolation Rate (gallon/sf trench area day)	0.5
Over Percolate?	**No**

Note:
(1) Month
(2) ADWF converted to acre-ft/month
(3) Calculated I/I flows. ADWF and influent flows obtained from CCWD 2001 to 2004 monitoring reports.
(4) Total effluent flow is equal to the sum of the ADWF plus I/I. Column (2) + Column (3)
(5) Percolation is practiced during wet season which is estimated to be between October 16 through April 14. Percolation water obtained from Column (4)
(6) Irrigation is practiced during dry season which is estimated to be between April 15 and October 15. Irrigation water obtained from Column (4)

Table E2 - Water Balance (Base Scenario - Existing Service Area)
Arnold WWTP Facility - Water Balance

Month	Days	EFFLUENT PRODUCTION ADWF gpd	gal/month	ac-ft/month	I/I ac-ft/month	Total Effluent ac-ft	WET SEASON PERCOLATION Application Rate ac-ft/yr	DRY SEASON IRRIGATION Application Rate ac-ft/yr
(1)				(2)	(3)	(4)	(5)	(6)
Oct	31	240,240	7,447,440	22.85	1.52	24.4	12.2	12.2
Nov	30	240,240	7,207,200	22.11	2.82	24.9	24.9	0.0
Dec	31	240,240	7,447,440	22.85	3.68	26.5	26.5	0.0
Jan	31	240,240	7,447,440	22.85	3.68	26.5	26.5	0.0
Feb	28	240,240	6,726,720	20.64	2.75	23.4	23.4	0.0
Mar	31	240,240	7,447,440	22.85	2.79	25.6	25.6	0.0
Apr	30	240,240	7,207,200	22.11	1.84	24.0	12.0	12.0
May	31	240,240	7,447,440	22.85	1.01	23.9	0.0	23.9
Jun	30	240,240	7,207,200	22.11	0.37	22.5	0.0	22.5
Jul	31	240,240	7,447,440	22.85	0.25	23.1	0.0	23.1
Aug	31	240,240	7,447,440	22.85	0.38	23.2	0.0	23.2
Sep	30	240,240	7,207,200	22.11	0.61	22.7	0.0	22.7
Total				269.06	21.71	290.8	151.2	139.6

Average Dry Weather Flow, gal/d: **240,240** Current ADWF

DRY SEASON PERCOLATION

Irrigation Area (Dry Season), acres:	46.5	Increase to reduced rate to 36 in/yr
Maximum Application Rate (inches per year)	36.0	
Irrigation Duration	April 15 through October 15	
Number of Irrigation Days	183	
Actual Application Rate (inches per year)	36.0	
Over Irrigating ?	**No**	

WET SEASON PERCOLATION

Percolation Area (sf trench area per bed)	16,000	
Number of Percolation Beds	17	Increased to reduce rate to 1.0
Maximum Percolation Rate (gallon/sf trench area day)	1.0	
Number of Percolation Days	182	
Applied Percolation Rate (gallon/sf trench area day)	1.0	
Over Percolate?	**No**	

Note:
(1) Month
(2) ADWF converted to acre-ft/month
(3) Calculated I/I flows. ADWF and influent flows obtained from CCWD 2001 to 2004 monitoring reports.
(4) Total effluent flow is equal to the sum of the ADWF plus I/I. Column (2) + Column (3)
(5) Percolation is practiced during wet season which is estimated to be between October 16 through April 14. Percolation water obtained from Column (4)
(6) Irrigation is practiced during dry season which is estimated to be between April 15 and October 15. Irrigation water obtained from Column (4)

Table E3 - Water Balance (Scenario 1 - Existing Service Area Plus Millwoods)
Arnold WWTP Facility - Water Balance

Month	Days	EFFLUENT PRODUCTION ADWF gpd	ADWF gal/month	ADWF ac-ft/month	I/I ac-ft/month	Total Effluent ac-ft	WET SEASON PERCOLATION Application Rate ac-ft/yr	DRY SEASON IRRIGATION Application Rate ac-ft/yr
(1)				(2)	(3)	(4)	(5)	(6)
Oct	31	274,755	8,517,405	26.14	1.70	27.8	13.9	13.9
Nov	30	274,755	8,242,650	25.29	3.15	28.4	28.4	0.0
Dec	31	274,755	8,517,405	26.14	4.10	30.2	30.2	0.0
Jan	31	274,755	8,517,405	26.14	4.10	30.2	30.2	0.0
Feb	28	274,755	7,693,140	23.61	3.07	26.7	26.7	0.0
Mar	31	274,755	8,517,405	26.14	3.11	29.2	29.2	0.0
Apr	30	274,755	8,242,650	25.29	2.05	27.3	13.7	13.7
May	31	274,755	8,517,405	26.14	1.13	27.3	0.0	27.3
Jun	30	274,755	8,242,650	25.29	0.41	25.7	0.0	25.7
Jul	31	274,755	8,517,405	26.14	0.28	26.4	0.0	26.4
Aug	31	274,755	8,517,405	26.14	0.42	26.6	0.0	26.6
Sep	30	274,755	8,242,650	25.29	0.68	26.0	0.0	26.0
Total				307.72	24.23	331.9	172.4	159.5

Average Dry Weather Flow, gal/d: **274,755** Current ADWF

DRY SEASON PERCOLATION

Irrigation Area (Dry Season), acres: 53.2 Increase to reduced rate to 36 in/yr
Maximum Application Rate (inches per year) 36.0
Irrigation Duration April 15 through October 15
Number of Irrigation Days 183
Actual Application Rate (inches per year) 36.0
Over Irrigating ? **No**

WET SEASON PERCOLATION

Percolation Area (sf trench area per bed) 16,000
Number of Percolation Beds 19 Increased to reduce rate to 1.0
Maximum Percolation Rate (gallon/sf trench area day) 1.0
Number of Percolation Days 182
Applied Percolation Rate (gallon/sf trench area day) 1.0
Over Percolate? **No**

Note:
(1) Month
(2) ADWF converted to acre-ft/month
(3) Calculated I/I flows. ADWF and influent flows obtained from CCWD 2001 to 2004 monitoring reports.
(4) Total effluent flow is equal to the sum of the ADWF plus I/I. Column (2) + Column (3)
(5) Percolation is practiced during wet season which is estimated to be between October 16 through April 14. Percolation water obtained from Column (4)
(6) Irrigation is practiced during dry season which is estimated to be between April 15 and October 15. Irrigation water obtained from Column (4)

Table E4 - Water Balance (Scenario 2 - Existing Service Area Plus Millwoods)
Arnold WWTP Facility - Water Balance

Month (1)	Days	EFFLUENT PRODUCTION ADWF gpd	ADWF gal/month	ADWF ac-ft/month (2)	I/I ac-ft/month (3)	Total Effluent ac-ft (4)	WET SEASON PERCOLATION Application Rate ac-ft/yr (5)	DRY SEASON IRRIGATION Application Rate ac-ft/yr (6)
Oct	31	244,530	7,580,430	23.26	1.59	24.8	12.4	12.4
Nov	30	244,530	7,335,900	22.51	2.95	25.5	25.5	0.0
Dec	31	244,530	7,580,430	23.26	3.84	27.1	27.1	0.0
Jan	31	244,530	7,580,430	23.26	3.84	27.1	27.1	0.0
Feb	28	244,530	6,846,840	21.01	2.87	23.9	23.9	0.0
Mar	31	244,530	7,580,430	23.26	2.91	26.2	26.2	0.0
Apr	30	244,530	7,335,900	22.51	1.92	24.4	12.2	12.2
May	31	244,530	7,580,430	23.26	1.06	24.3	0.0	24.3
Jun	30	244,530	7,335,900	22.51	0.38	22.9	0.0	22.9
Jul	31	244,530	7,580,430	23.26	0.26	23.5	0.0	23.5
Aug	31	244,530	7,580,430	23.26	0.40	23.7	0.0	23.7
Sep	30	244,530	7,335,900	22.51	0.64	23.1	0.0	23.1
Total				273.87	22.65	296.5	154.3	142.2

Average Dry Weather Flow, gal/d: **244,530** Current ADWF

DRY SEASON PERCOLATION

Irrigation Area (Dry Season), acres:	47.4	Increase to reduced rate to 36 in/yr
Maximum Application Rate (inches per year)	36.0	
Irrigation Duration	April 15 through October 15	
Number of Irrigation Days	183	
Actual Application Rate (inches per year)	36.0	
Over Irrigating ?	**No**	

WET SEASON PERCOLATION

Percolation Area (sf trench area per bed)	16,000	
Number of Percolation Beds	17	Increased to reduce rate to 1.0
Maximum Percolation Rate (gallon/sf trench area day)	1.0	
Number of Percolation Days	182	
Applied Percolation Rate (gallon/sf trench area day)	1.0	
Over Percolate?	**No**	

Note:
(1) Month
(2) ADWF converted to acre-ft/month
(3) Calculated I/I flows. ADWF and influent flows obtained from CCWD 2001 to 2004 monitoring reports.
(4) Total effluent flow is equal to the sum of the ADWF plus I/I. Column (2) + Column (3)
(5) Percolation is practiced during wet season which is estimated to be between October 16 through April 14. Percolation water obtained from Column (4)
(6) Irrigation is practiced during dry season which is estimated to be between April 15 and October 15. Irrigation water obtained from Column (4)

Table E5 - Water Balance (Scenario 3 - Existing Service Area Plus Avery and Millwoods)
Arnold WWTP Facility - Water Balance

Month	Days	EFFLUENT PRODUCTION ADWF gpd	ADWF gal/month	ADWF ac-ft/month	I/I ac-ft/month	Total Effluent ac-ft	WET SEASON PERCOLATION Application Rate ac-ft/yr	DRY SEASON IRRIGATION Application Rate ac-ft/yr
(1)				(2)	(3)	(4)	(5)	(6)
Oct	31	279,045	8,650,395	26.54	1.76	28.3	14.2	14.2
Nov	30	279,045	8,371,350	25.69	3.27	29.0	29.0	0.0
Dec	31	279,045	8,650,395	26.54	4.26	30.8	30.8	0.0
Jan	31	279,045	8,650,395	26.54	4.26	30.8	30.8	0.0
Feb	28	279,045	7,813,260	23.97	3.19	27.2	27.2	0.0
Mar	31	279,045	8,650,395	26.54	3.23	29.8	29.8	0.0
Apr	30	279,045	8,371,350	25.69	2.13	27.8	13.9	13.9
May	31	279,045	8,650,395	26.54	1.18	27.7	0.0	27.7
Jun	30	279,045	8,371,350	25.69	0.43	26.1	0.0	26.1
Jul	31	279,045	8,650,395	26.54	0.29	26.8	0.0	26.8
Aug	31	279,045	8,650,395	26.54	0.44	27.0	0.0	27.0
Sep	30	279,045	8,371,350	25.69	0.71	26.4	0.0	26.4
Total				312.52	25.17	337.7	175.6	162.1

Average Dry Weather Flow, gal/d: **279,045** Current ADWF

DRY SEASON PERCOLATION

Irrigation Area (Dry Season), acres: 54.1 Increase to reduced rate to 36 in/yr
Maximum Application Rate (inches per year) 36.0
Irrigation Duration April 15 through October 15
Number of Irrigation Days 183
Actual Application Rate (inches per year) 35.9
Over Irrigating ? **No**

WET SEASON PERCOLATION

Percolation Area (sf trench area per bed) 16,000
Number of Percolation Beds 20 Increased to reduce rate to 1.0
Maximum Percolation Rate (gallon/sf trench area day) 1.0
Number of Percolation Days 182
Applied Percolation Rate (gallon/sf trench area day) 1.0
Over Percolate? **No**

Note:
(1) Month
(2) ADWF converted to acre-ft/month
(3) Calculated I/I flows. ADWF and influent flows obtained from CCWD 2001 to 2004 monitoring reports.
(4) Total effluent flow is equal to the sum of the ADWF plus I/I. Column (2) + Column (3)
(5) Percolation is practiced during wet season which is estimated to be between October 16 through April 14. Percolation water obtained from Column (4)
(6) Irrigation is practiced during dry season which is estimated to be between April 15 and October 15. Irrigation water obtained from Column (4)

Appendix F. Improvements and Timelines for Scenario 1, Scenario 2 and Scenario 3.

Appendix F

Scenario 1, 2, and 3 Improvement and Timeline Requirements

Projected average dry weather flows (ADWFs) at buildout at estimated to be approximately 240,000, 275,000, 245,000, and 280,000 gallons per day (gpd) for the Base Scenario and Scenarios 1, 2, and 3, respectively.

Currently the plant has a rated ADWF capacity of 170,000 gpd. It is recommended that an additional clarifier and return activated sludge (RAS) pump be installed in the first improvement phase. This clarifier is recommended for redundancy and to allow the existing clarifier to be taken out of service for routine maintenance. Based on this approach, the new clarifier will be 26 ft diameter, with a greater side water depth. This sizing criterion is based on mirroring the existing clarifier as opposed to providing additional clarification capacity based on the difference between the projected ADWF at buildout and the current plant capacity. Once installed, the secondary clarifiers and RAS system will provide adequate capacity through buildout for all growth scenarios. A same sizing criterion is recommended for the oxidation ditch. However a second oxidation ditch is not required until the Phase III improvements.

Overall, the oxidation ditch and secondary clarifier represent a large portion of the overall expansion costs. Moreover, the relative difference in ADWFs between the four scenarios is at most 40,000 gpd, which is relatively small. Based on these considerations, it is expected that the relative costs for expanding the treatment plant are expected to be similar for all four growth scenarios. However, as described in the Millwoods alternative analysis, the expansion timeline will change based on which scenario is implemented.

Table 1 provides a summary of the improvements, estimated costs, and timeline requirements for all four growth scenarios.

Table 1. Summary of Recommended Timeline and Costs for All Growth Scenarios

	Base Scenario	**Scenario 1 – Existing Service Area Plus Millwoods**	**Scenario 2 – Existing Service Area Plus Avery**	**Scenario 3 – Existing Service Area Plus Millwoods and Avery**
	Recommended	**Not Recommended**	**Recommended provided collection system expansion is paid for by Avery commercial area**	**Not Recommended**
Phase I Improvements				
Timeline	Immediately	Immediately	Immediately	Immediately
Improvements	See Table 18	See Table 18	See Table 18	See Table 18
Estimated Project Cost	$1,170,000	$1,170,000	$1,170,000	$1,170,000
Phase II Improvements				
Timeline	2011	2009	2010	2008
Improvements	See Page 40	See Page 40	See Page 40	See Page 40

Estimated Project Cost	$865,000	$865,000	$865,000	$865,000
Phase III Improvements				
Timeline[a]	2020	2014	2019	2014
Improvements	See Table 19	See Table 19	See Table 19	See Table 19
Estimated Project Cost	$2,380,000	$2,380,000	$2,380,000	$2,380,000

[a] Year in which expansion is required to be in service.

Appendix G. Responses to Public Comments.

Arnold Wastewater Master Plan

Calaveras County Water District

PUBLIC PRESENTATION

January 25, 2005

HDR

Purpose

Describe District's Planning Efforts

Present Draft Master Plan Results

Collect Comments and Feedback Prior to Finalizing Plan

Planning Effort Overview

Master Plan
(*Arnold Sewer System*)

Identify specific improvements
- **Regulations**
- **Growth**
- **Facility Age**

Financial Plan
(*District-wide*)

Develop a basis for managed upgrade to meet short and long-term needs

Master Plan Components

Master Plan

Recommend Improvements and Schedule → **Financial Plan**

Compare Alternatives

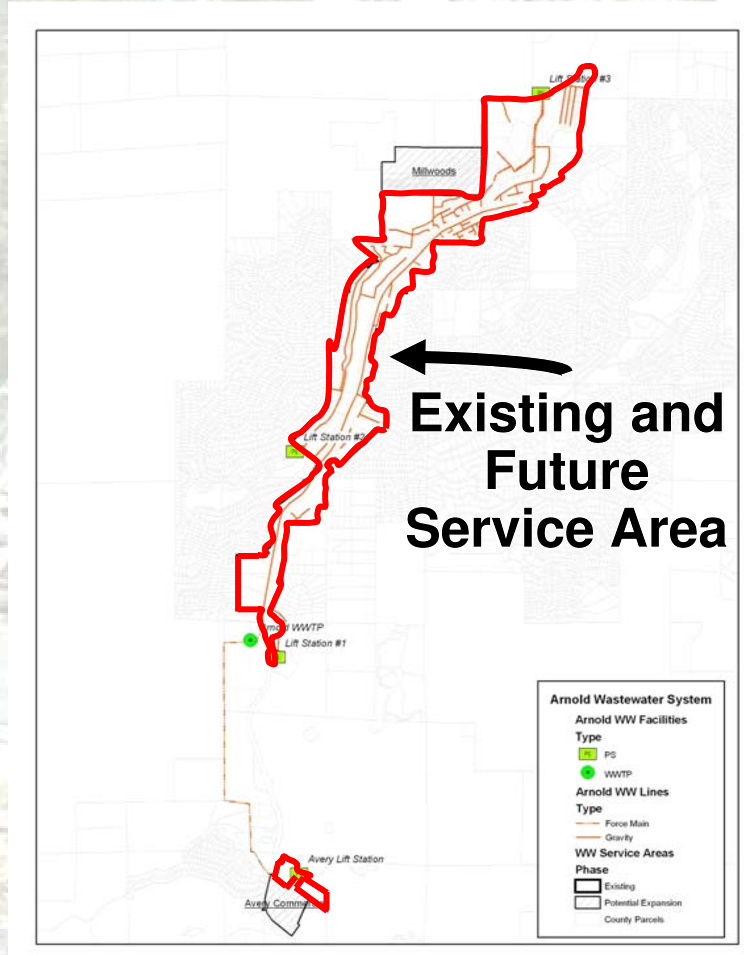
Evaluate Current Facilities

Assess Regulations

Characterize Current and Future Flows

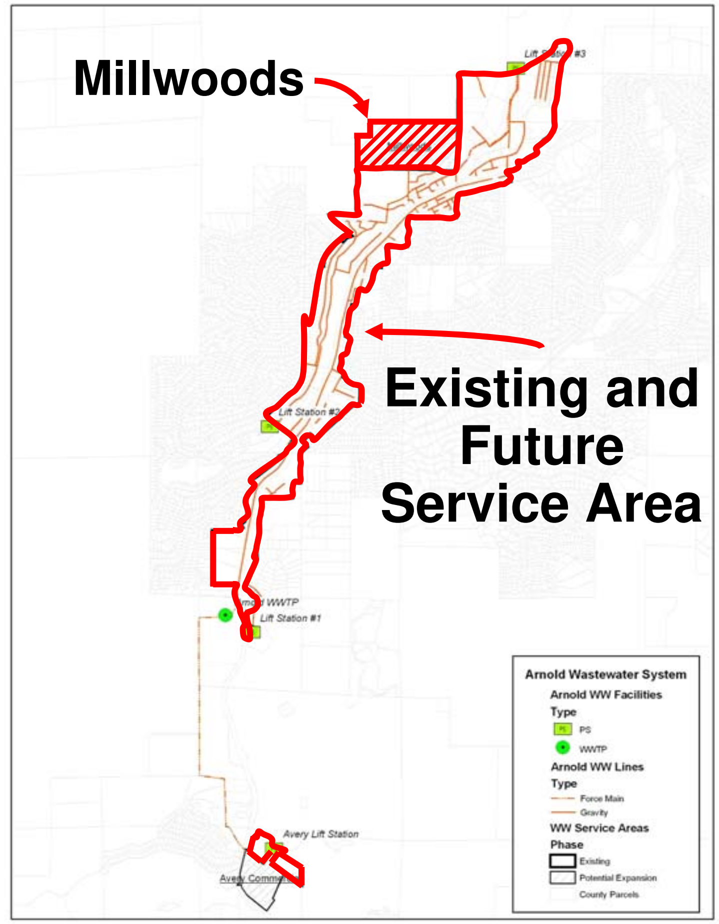
Existing Service Area
Scenario 1

Location	Contributions (ESFUs)
Existing (Current)	638
Infill (Future)	348
Cedar Ridge (Future)	213
Total (Buildout)	1,199



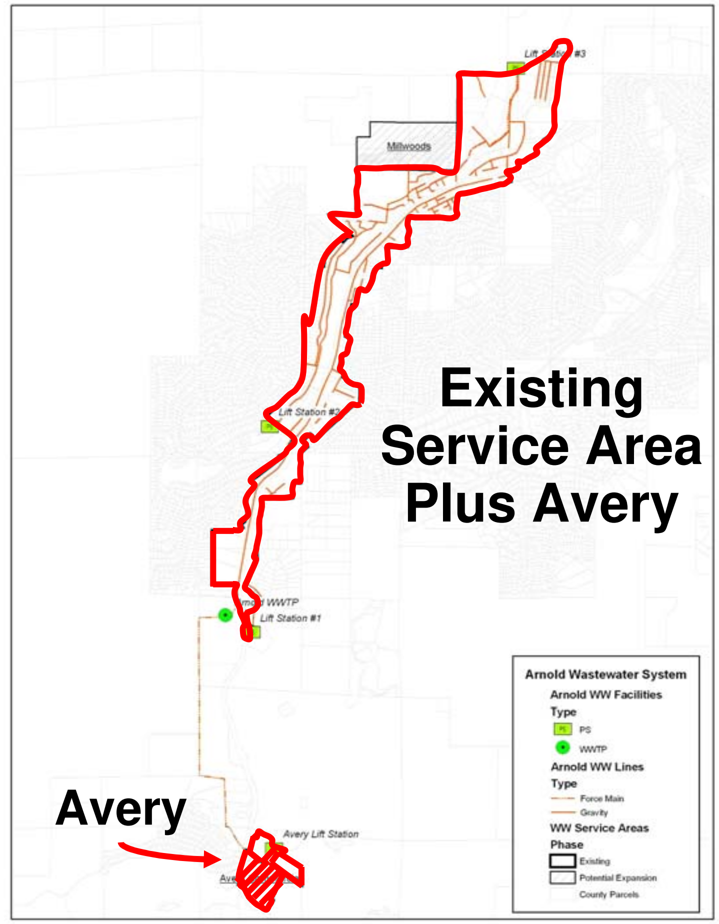
Millwoods Addition
Scenario 2

Location	Contributions (ESFUs)
Existing	638
Infill	348
Cedar Ridge	213
Millwoods	177
Total (Buildout)	1,376



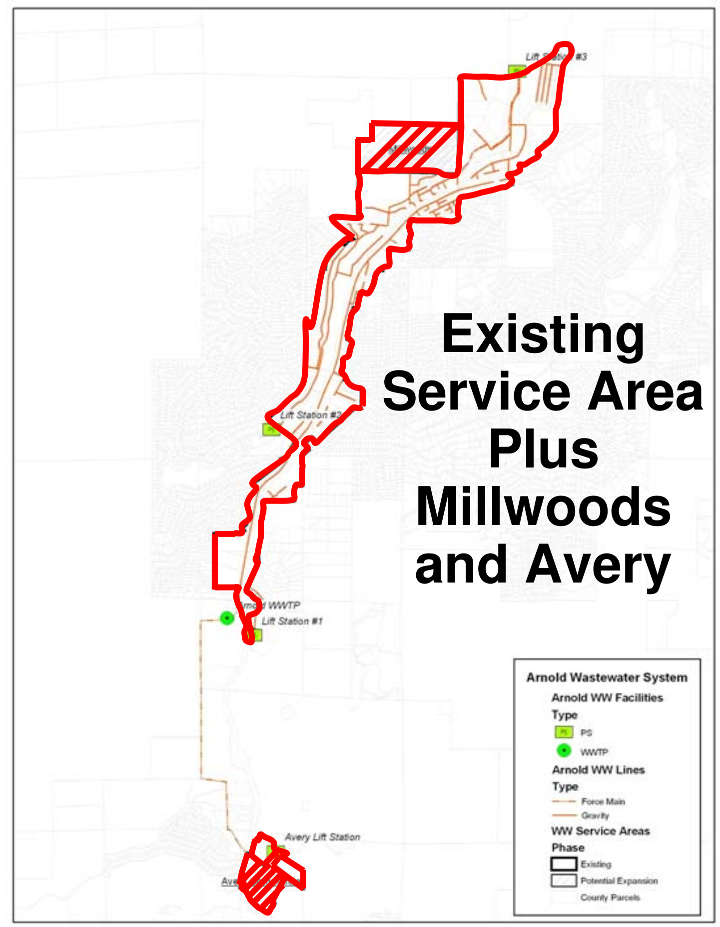
Avery Addition
Scenario 3

Location	Contributions (ESFUs)
Existing	**638**
Infill	**348**
Cedar Ridge	**213**
Avery	**83**
Total (Buildout)	**1,282**



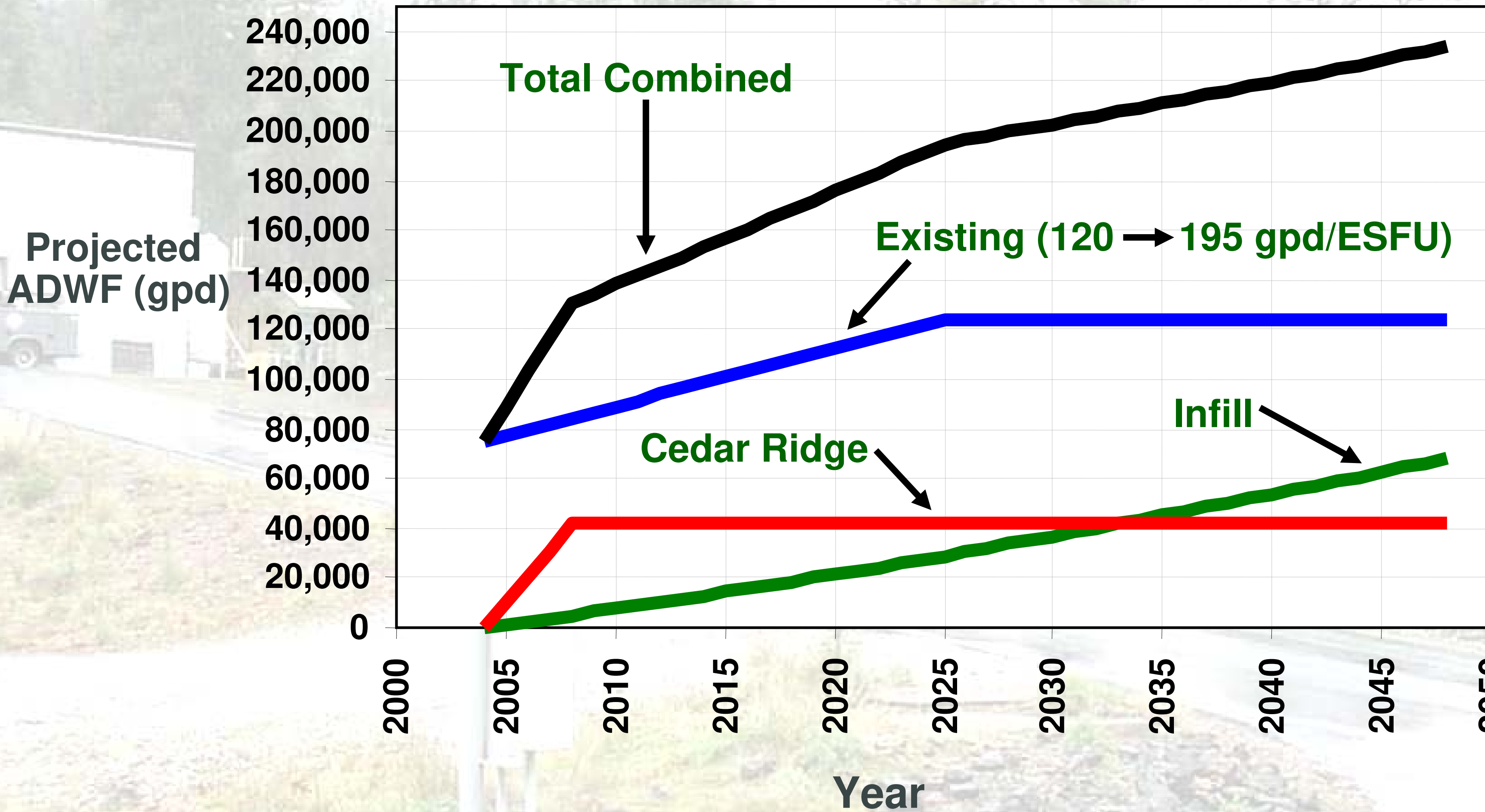
Millwoods and Avery Addition
Scenario 4

Location	Contributions (ESFUs)
Existing	638
Infill	348
Cedar Ridge	213
Millwoods	177
Avery	83
Total (Buildout)	1,459



Current and Future Flows

Scenario 1



Regulatory Considerations

WDR Adopted April 1997

2007 Renew Permit

Future Changes/ Requirements

Groundwater Monitoring

Disinfection By-Products

Facility Evaluation
(accommodate Buildout)

Conveyance System	LS 1, 2, and 3 require additional capacity (*LS 1 scheduled to be replaced*)
Treatment Plant	• Various O & M Improvements – 20 years old • Capacity of most major processes exceeded in 10 to 15 years
Effluent Holding	Additional capacity needed only if existing disposal system is expanded
Effluent Disposal	• Additional capacity not required. • Recommend existing 40 acre site be set aside as fail safe.

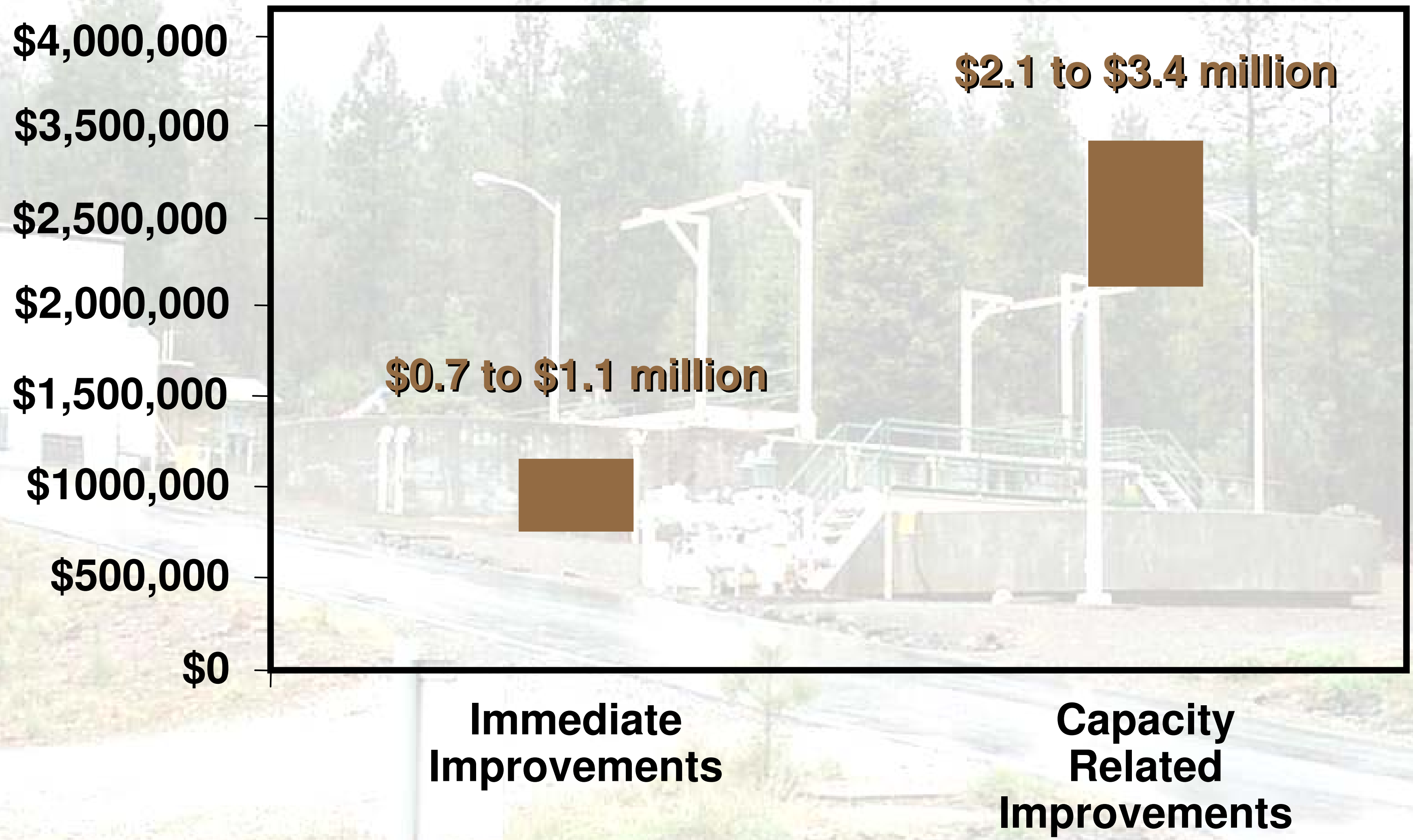
Immediate Improvements
(within the next 5 years)

Conveyance System	Lift Station 1 - replaced as part of Cedar Ridge Development
Treatment Plant	• Septage Receiving Improvements • Ability to Optimize Energy Use (DO control) • Second Clarifier • Effluent Pumps
Effluent Holding	None Required
Effluent Disposal	None Required

Capacity Related Improvements
(required by 2017)

Conveyance System	**Lift Stations 2 and 3**
Treatment Plant	**Expansion of most unit processes**
Effluent Holding	**Not Required**
Effluent Disposal	**Not Required**

Preliminary Project Costs



Next Steps

Obtain and Address District and Public Comments

Refine Improvement Costs and Develop Timelines

Allocate Improvement Costs (Existing, Infill, and New)

Final Master Plan (February 2005) → Financial Plan Input

Future Stakeholder Presentations

- Final Master Plan and Preliminary Financial Plan ~ May 2005
- Final Financial Plan ~ June 2005

Arnold Wastewater Master Plan

Calaveras County Water District

PUBLIC PRESENTATION

January 25, 2005

HDR

Arnold Sewer System Master Plan

Calaveras County Water District

PUBLIC PRESENTATION

May 4, 2005

HDR

Purpose

Present Master Plan Results and Recommendations

Describe Cost Information
Input to Financial Master Plan

Overview of Response to Public Comments

Planning Effort Overview

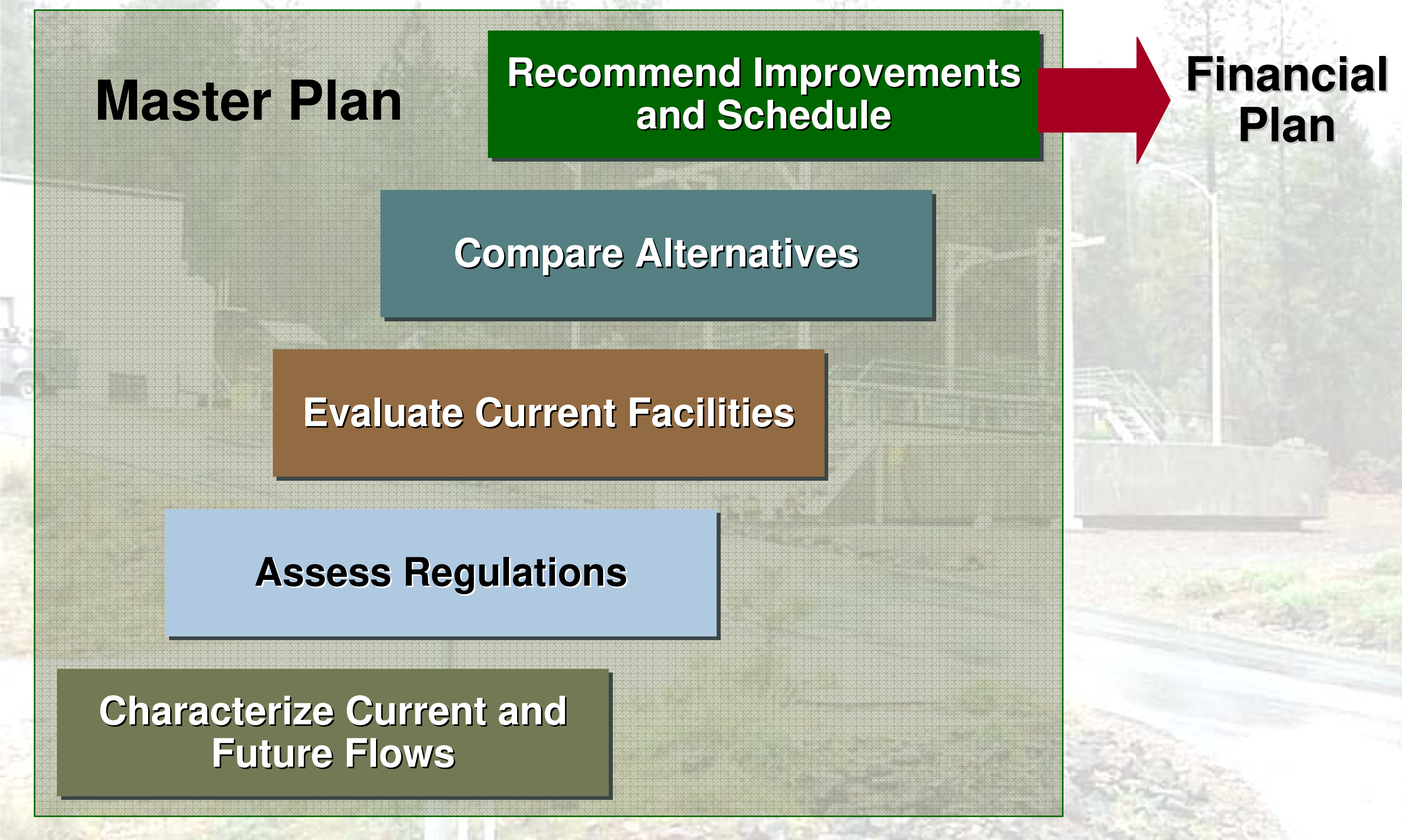
Master Plan
(*Arnold Sewer System*)

Identify specific improvements
- **Regulations**
- **Growth**
- **Facility Age**

Financial Plan
(*District-wide*)

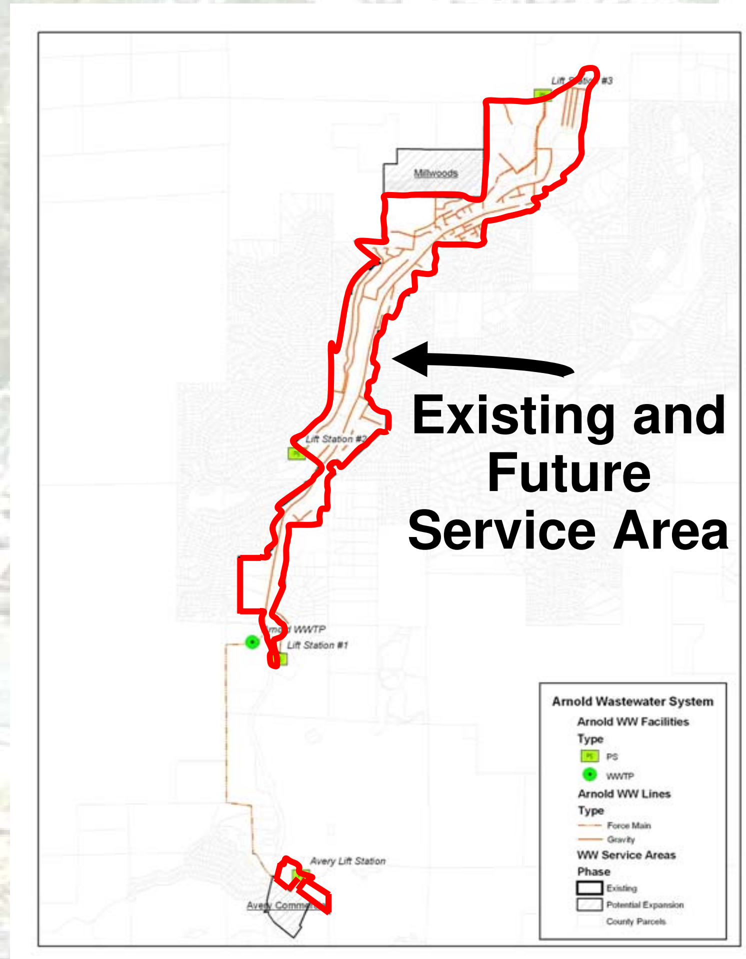
Develop a basis for managed upgrade to meet short and long-term needs

Master Plan Components



Existing Service Area
Base Scenario

Location	Contributions (ESFUs)
Existing (Current)	638
Avery (Current)	33
Infill (Future)	348
Cedar Ridge (Future)	213
Total	1,232

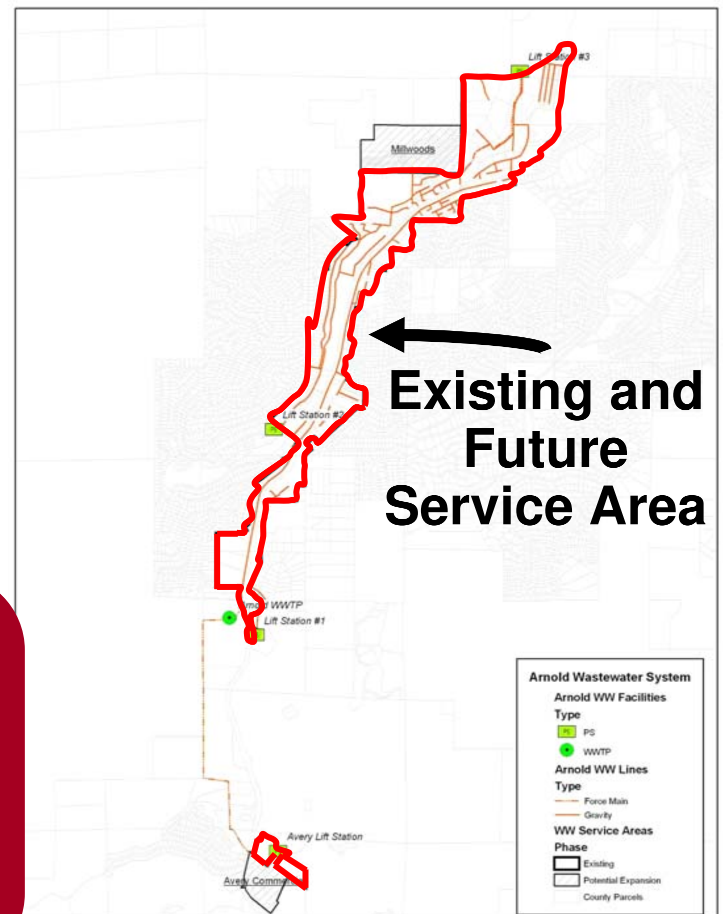


Existing Service Area
Base Scenario

Location	Contributions (ESFUs)
Existing (Current)	638
Avery (Current)	33
Infill (Future)	348
Cedar Ridge (Future)	213
Total	1,232

How was this value determined?

- ***1984 Assessment Project: 986 ESFUs***
- ***2002 + Historic Growth: 638 ESFUs***
- ***Difference = 348 ESFUs***



Millwoods Addition
Scenario 1

Location	Contributions (ESFUs)
Base Scenario	1,232
Millwoods	177
Total	1,409

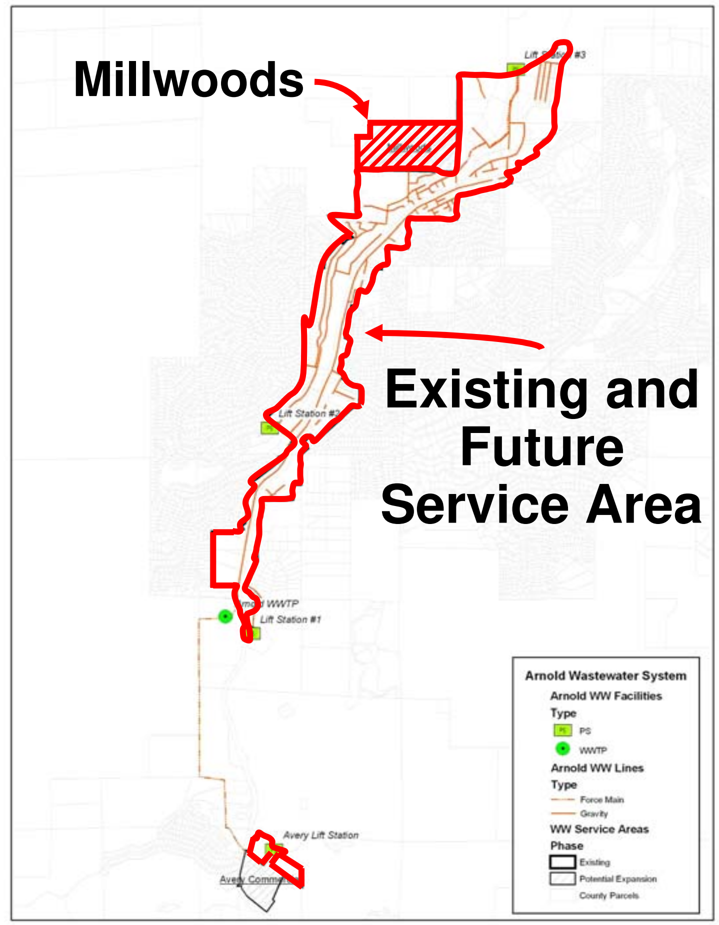
Cost Comparison with/without Millwoods System

Results

- Continued use of Millwoods system: 35% less than combining with Arnold

Recommendations

- Continue operating Millwoods as a separate system



Avery Addition
Scenario 2

Location	Contributions (ESFUs)
Base Scenario	1,232
Avery Commercial	22
Total (Buildout)	1,254

Cost Comparison with/without Millwoods System

Results

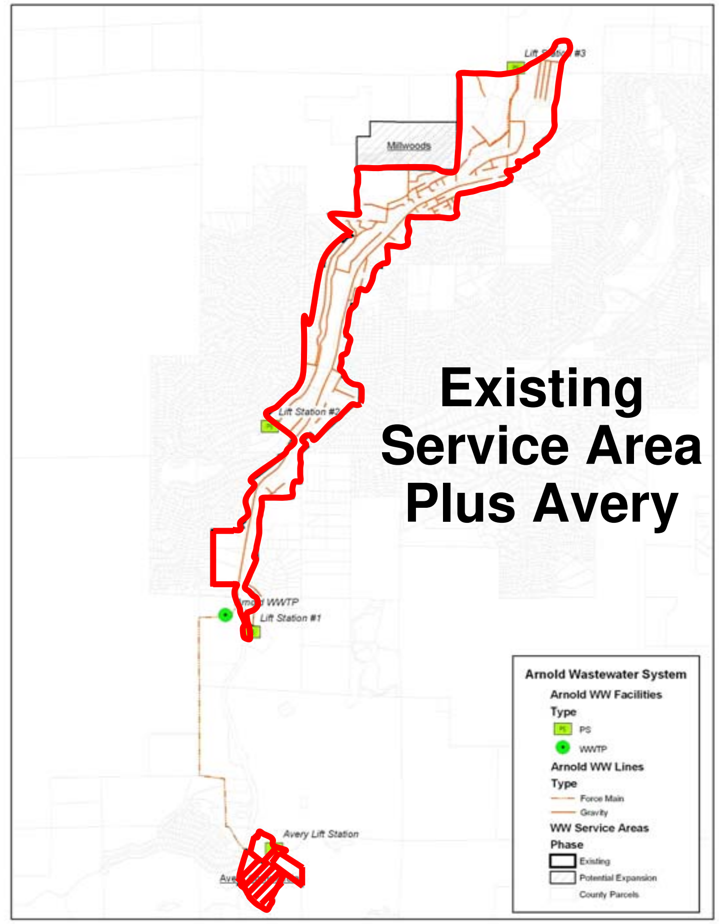
- Negligible impact on cost and timing of Arnold WWTP improvements

Recommendations

- Allow connection

Improvement Needed

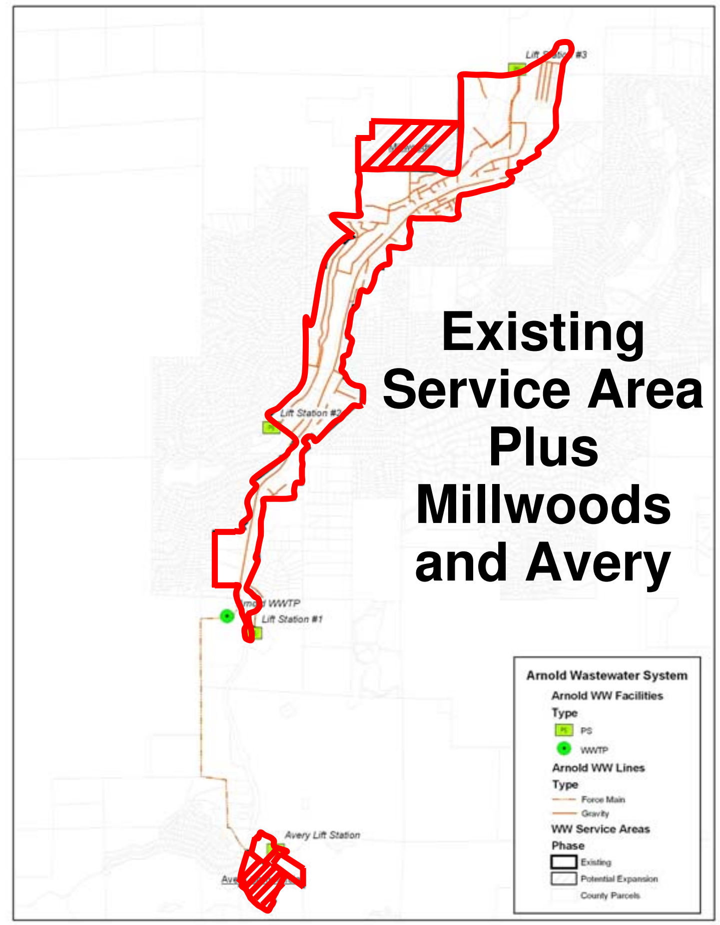
- Collection system – paid for by Avery Commercial Area



Millwoods and Avery Addition
Scenario 3

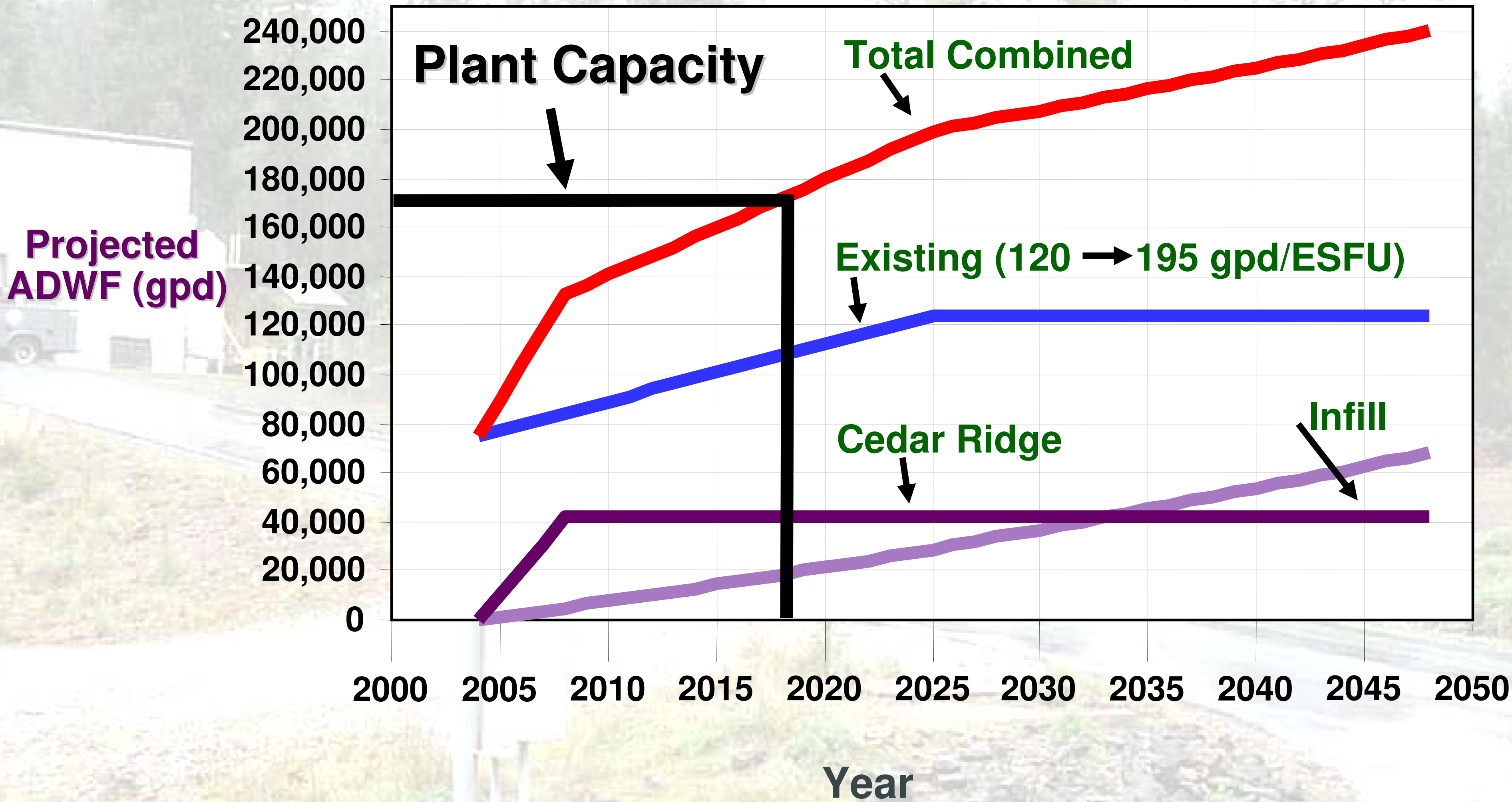
Location	Contributions (ESFUs)
Base Scenario	**1,232**
Millwoods	**177**
Avery	**22**
Total	**1,431**

Not Recommended

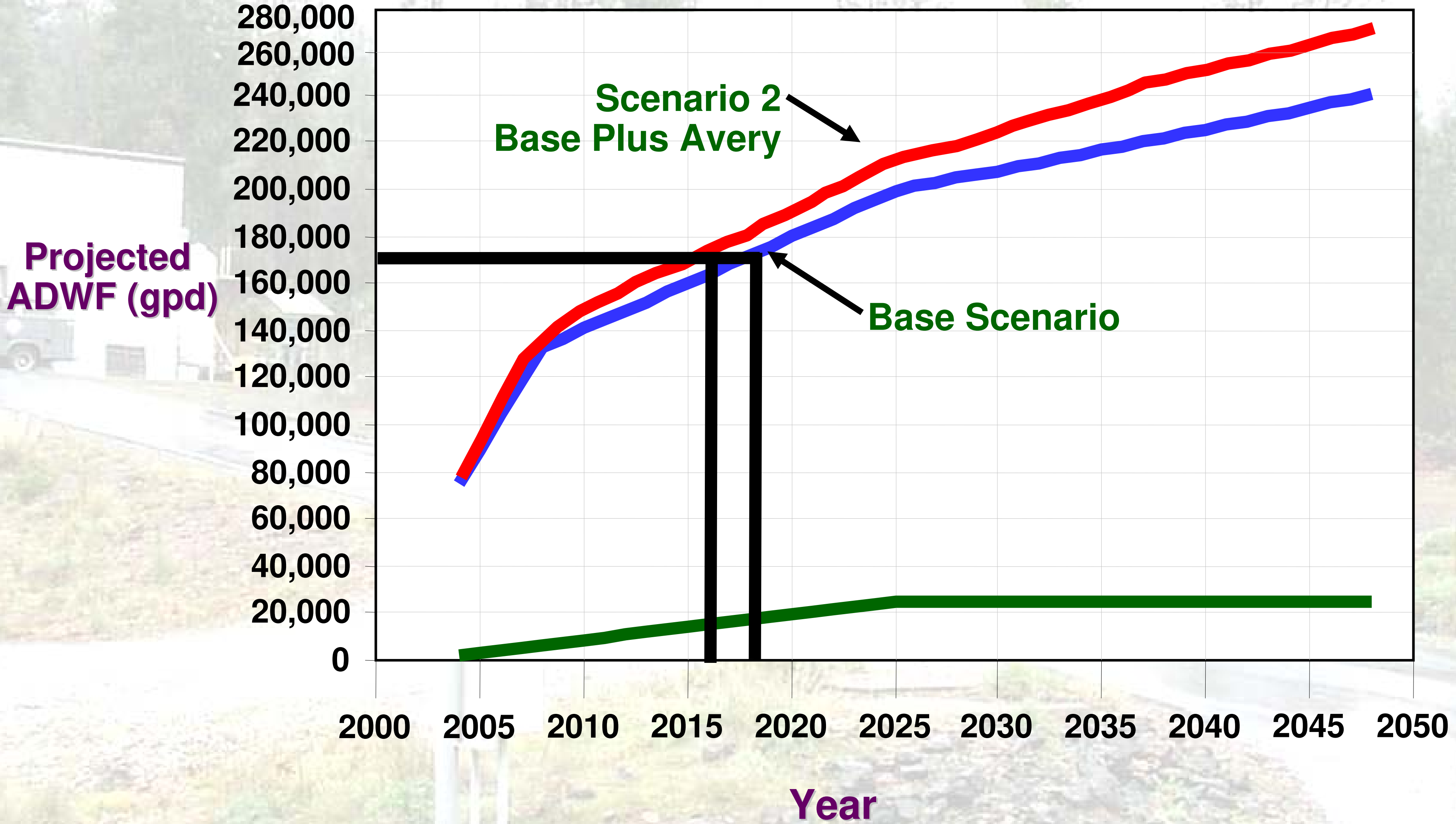


Current and Future Flows

Base Scenario



Current and Future Flows
Base Scenario and Scenario 2



Facility Evaluation
(accommodate Buildout)

Conveyance System	**LS 1, 2, and 3 require additional capacity** **(*LS 1 scheduled to be replaced for Cedar Ridge*)**
Treatment Plant	• **Various O & M Improvements – 20 years old** • **Capacity of most major processes exceeded in 2018**
Effluent Holding	**Additional capacity needed only if existing disposal system is expanded**
Effluent Disposal	• **Develop 25 acres for leachfield and spray field expansion.** • **Remaining 15 acre site to be set aside as fail safe.**

Phase I (Immediate) Improvements

Area	Improvements
Conveyance System	Lift Station 1 – to be replaced as part of Cedar Ridge Development Lift Station 2 – Firm capacity of 350 gpm
Treatment Plant	Ability to Optimize Energy Use (DO control) Second Clarifier Effluent Pumps
Effluent Holding/Disposal	None Required

Estimated Project Cost: $1,190,000

Phase II (Capacity Related) Improvements
(Required by 2018)

Conveyance System	• Lift Station 3 – Firm capacity of 350 gpm
Treatment Plant	• Expansion of most unit processes
Effluent Holding/Disposal	• Leachfield and spray field expansion

Estimated Project Cost: $3,245,000

Other Considerations: Replacement Costs

	Replacement Cost	*Useful Life*
Building	$289,000	50
Improvements other than Buildings	$6,158,000	50
Machinery & Equipment	$556,700	10
Total	$7,004,000	

Facility Plan Input

Compliance Costs	$760,000
Expansion Costs	$3,675,000
Replacement Costs*	$184,600/yr*

Actual value to be determined during Financial MP

* Based on estimated replacement costs of $7.0 million

Next Steps

Develop Draft and Final Financial Master Plans

Future Stakeholder Presentations

- Final Master Plan and Preliminary Financial Plan – Early June 2005
- Final Financial Plan – Late June 2005

Questions

Arnold Sewer System Master Plan

Calaveras County Water District

PUBLIC PRESENTATION

May 4, 2005

HDR