Buckskin Sanitary District, Arizona

BUCKSKIN SANITARY DISTRICT
2011 WASTEWATER MASTER PLAN UPDATE

FINAL

August 2011

14615
MARK EVERETT
COURTNEY
Date Signed: 8-30-11
ARIZONA, U.S.A.

EXPIRES 06-30-2014

The undersigned has approved this document for and on behalf of Carollo Engineers, Inc.

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Buckskin Sanitary District, Arizona

**BUCKSKIN SANITARY DISTRICT
2011 WASTEWATER MASTER PLAN UPDATE**

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1.1   BACKGROUND

The Buckskin Sanitary District (BSD) is currently developing plans to expand and upgrade the wastewater collection and treatment system to serve the developed portions of the BSD service area. The BSD currently provides wastewater service to a portion of the customers in its service area. The remaining customers utilize septic tanks and drain fields for wastewater disposal. The BSD had a wastewater master plan created previously (Stanley Consultants – August 2007), which identified alternatives for providing wastewater collection and treatment for the entire BSD service area. The BSD has selected Carollo Engineers, Inc. (Carollo) to develop an update to the master plan.

This master plan update will focus on proposed treatment plant(s) as well as the reclaimed water management issues. More specifically, this master plan update project will include updated flow forecasting, capacity analysis, potential water reclamation plant locations, and Capital Improvement Plan (CIP) development. The BSD has received a Technical Assistance Grant from the Water Infrastructure Finance Authority of Arizona (WIFA) to be used for this master plan update. The update project is further described in the WIFA grant application, dated April 28, 2010.

The BSD owns and operates a collection system and one treatment facility, the Buckskin Wastewater Treatment Plant (WWTP) (formerly the Sandpiper WWTP). The WWTP, which was originally constructed to serve the Sandpiper Condominiums, now receives all of the wastewater flow from the sewered portions of the BSD service area, consisting of Planning Phases I, II, and III. The WWTP is landlocked and, due to the character of the community, receives major fluctuations with influent flow rates.

The BSD recently completed additional collection system construction, which will allow the BSD to provide sewer service to a total of approximately 3,200 people, or approximately 30 percent of the existing population in the BSD planning area.

The BSD faces several significant challenges in its mission to provide wastewater service to the District. The septic systems that are prevalent in the service area are aging and some of the leach fields have failed. A comprehensive collection, treatment, and effluent management system must be developed for the service area in order to protect the health and safety of the community and the surface and groundwater quality in the area.

1.2 BUCKSKIN SANITARY DISTRICT SERVICE AREAS

Figure 1.1 shows the BSD service area. The currently developed service area consists of several villages and parks along the Colorado River. The BSD potential service area covers a larger area of undeveloped State lands. A significant portion of these State lands is likely to remain undeveloped in the future. Currently developed areas are estimated to be approximately 60 percent built out.

The District includes several "villages," each with different wastewater flow amounts and characteristics. Some villages are predominantly retirement communities with a stable population. Other communities consist of vacation homes where residents may visit primarily on weekends, holidays, and during vacations. Other areas consist of parks with seasonal visitors that reside in recreational vehicles. Table 1.1 identifies the names and general characteristics for each of the village areas. Figure 1.2 shows the locations of the village areas.

Table 1.1	Buckskin Sanitary District Village Area Characteristics 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona	
Village	**Existing Developments**	**Type of Development**
1	Moon Ridge	Resort
	Polynesian Shores	Vacation Homes
2	Buckskin Valley	Retirement Community
	Holiday Harbor	Vacation Homes
3	Sundance	Resort
	Rio Lindo	Vacation Homes
	Red Rock	
4	Marina Village	Resort
	Ah Villa Park	R.V. Park
	Branson Resort	Golf Course
		Vacation Homes
5	Moovalya Keys (Phase I – III)	Resort
		Vacation Homes
6	Castle Rock	Planned R.V. Park
		Resort
7	Buckskin Mountain State Park	Park

Legend

- ⬜ Existing Sewered Area (Phases I, II & III)
- ▨ Buckskin Sanitary District Planning Area
- ▬ Highway

Colorado River

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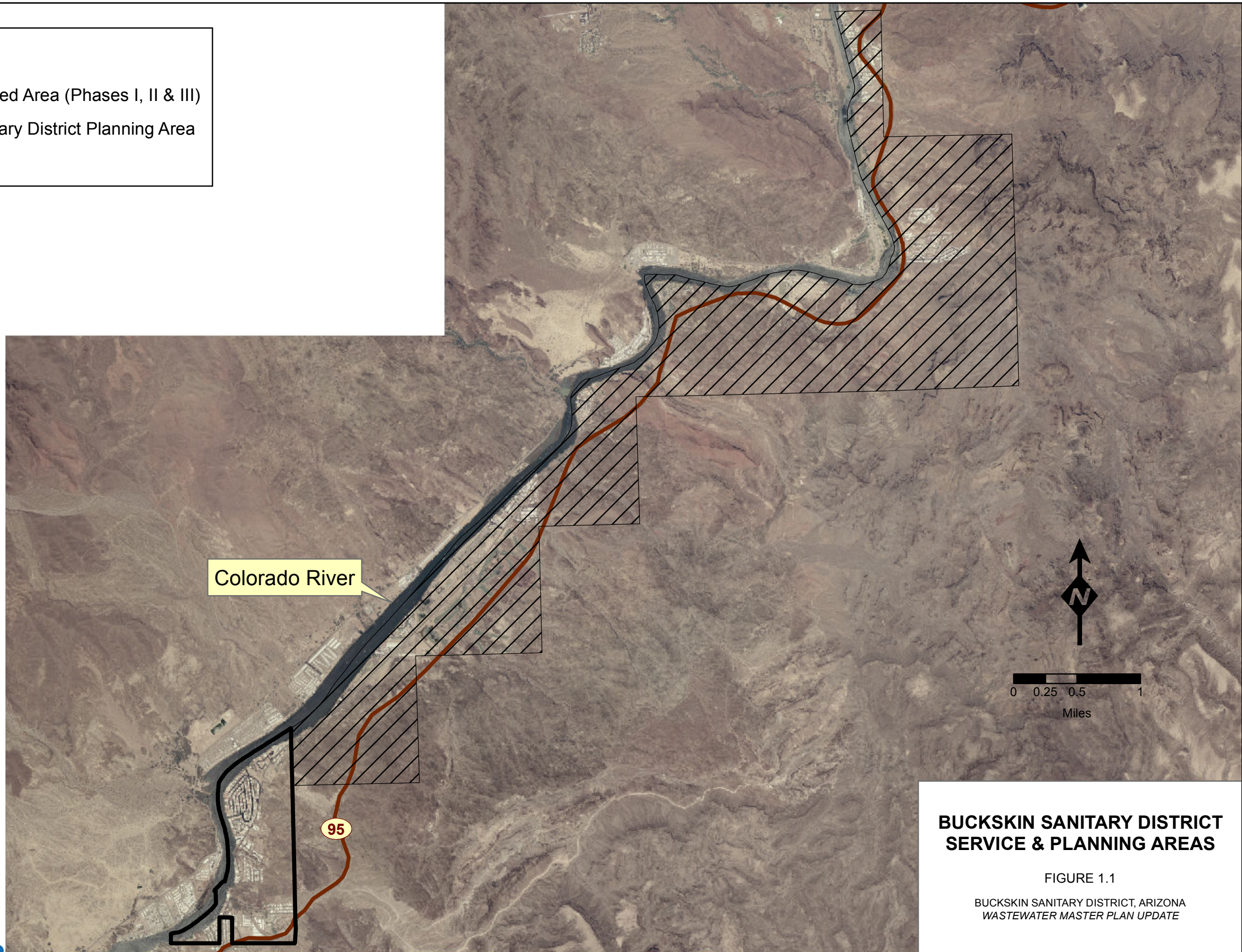
BUCKSKIN SANITARY DISTRICT SERVICE & PLANNING AREAS

FIGURE 1.1

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE

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Legend

Village Area
Highway

Note: Refer to Table 1.1 for Village Names

Colorado River

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**BUCKSKIN SANITARY DISTRICT
VILLAGE AREAS**

FIGURE 1.2

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE

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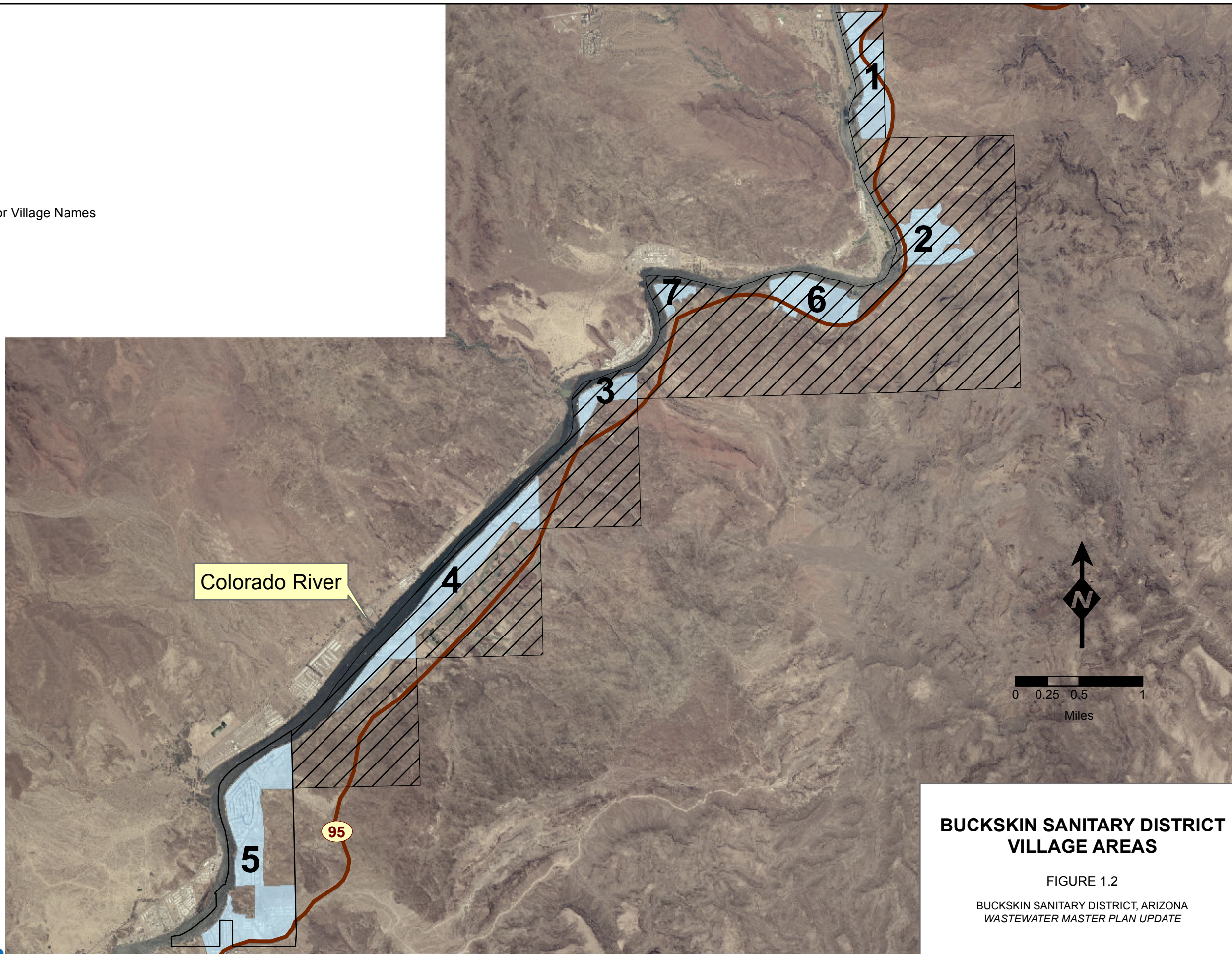
PLANNING FRAMEWORK

2.1 WASTEWATER FLOW PROJECTIONS

2.1.1 Land Use

Land use is one mechanism that can be used to estimate the quantity of wastewater flows that may be generated within the BSD service area. A land use plan consists of a geographic information system (GIS) data layer that defines specific land areas that have a similar type of development. These land areas are assumed to have similar wastewater flows on a per acre basis. Figure 2.1 shows the different types of land use within the BSD service area.

2.1.2 Population

The current population of the BSD was estimated by counting rooftops in each of the village areas, and then estimating the population in each residence. The population per residence can vary dramatically because many residences are a meeting place for larger groups of family and friends on weekends and holidays, then the residence may remain vacant during significant portions of the year. Other residences are primarily retirement homes that could have a seasonal or full-time population, but do not experience the weekend population swings. Park areas populated with residents in RVs will experience high populations during vacation times, and very low populations during other time periods. Ultimate population projections were based on the undeveloped land area in each village, estimated housing density per acre, and estimated population density per house.

Table 2.1 shows the estimated current population and buildout population for each village area.

Table 2.1	Buckskin Sanitary District Village Area Populations 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona	
Village	**Estimated 2010 Population**	**Estimated Buildout or Seasonal Population**
1	1,112	2,061
2	1,603	1,756
3	1,760	2,471
4	1,678	2,044
5	2,287	4,052
6	624	3,120
7	689	689
Total	**9,752**	**16,194**

Legend

Land Use Type

- Low Density Residential
- Transitional Residential
- Moderate Density Residential
- High Density Residential
- Recreational Vehicle Park
- Local Commercial
- Regional Commercial
- Park
- Planned Development
- Highway

Colorado River

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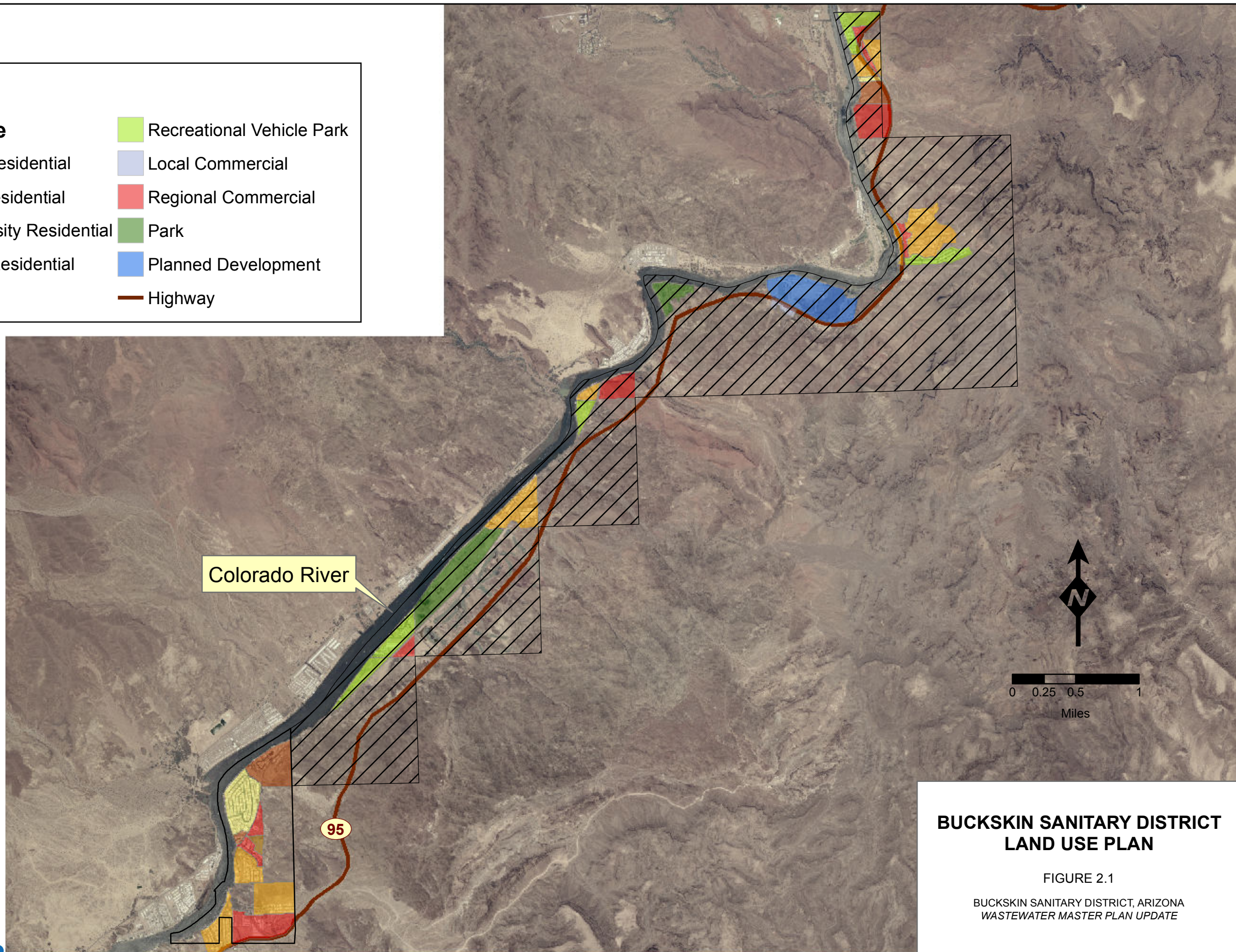
**BUCKSKIN SANITARY DISTRICT
LAND USE PLAN**

FIGURE 2.1

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE

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2.1.3 Historical Wastewater Flows and Peaking Factors

The information available to estimate wastewater flows is limited because the flows into the Existing Buckskin WWTP provide the only mechanism to measure wastewater flows, and the WWTP services only a small portion of the District area. Table 2.2 shows the average wastewater flows for year 2005.

Table 2.2	Historical (2005) Wastewater Flows to the Buckskin WWTP 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona
Month	**Average Daily Flow (gal/day)**
January	40,730
February	42,120
March	37,733
April	38,210
May	45,839
June	42,055
July	52,287
August	43,931
September	46,680
October	45,867
November	45,227
December	37,516
Annual Average Daily Flow (AADF)	**43,183**

Figure 2.2 shows the daily flows that were measured into the existing WWTP. This data gives a graphical representation of the peak flows that can be expected during weekends and holiday periods. Note that the peak flows are not only significantly higher than the average flows, but also often last up to a week in duration. Collection system, treatment, and reclaimed water storage facilities will need to be sized to handle these extended peak flows. Based on this flow data, the average flow is 43,200 gallons/day and the peak daily flow can be as high as 142,000 gallons/day. The population of the existing sewered area is estimated to be 2,287 people, which suggests an average occupancy of approximately 60 percent based on typical wastewater flows of 65 gallons/capita/day.

2.1.4 Estimated Wastewater Flows

The wastewater flow projections for the BSD for both current and buildout conditions, for average daily flows, and for seasonal periods are shown in Table 2.3. The wastewater flows are separated into northern and southern service areas that may correspond with future northern and southern WWTP service areas.
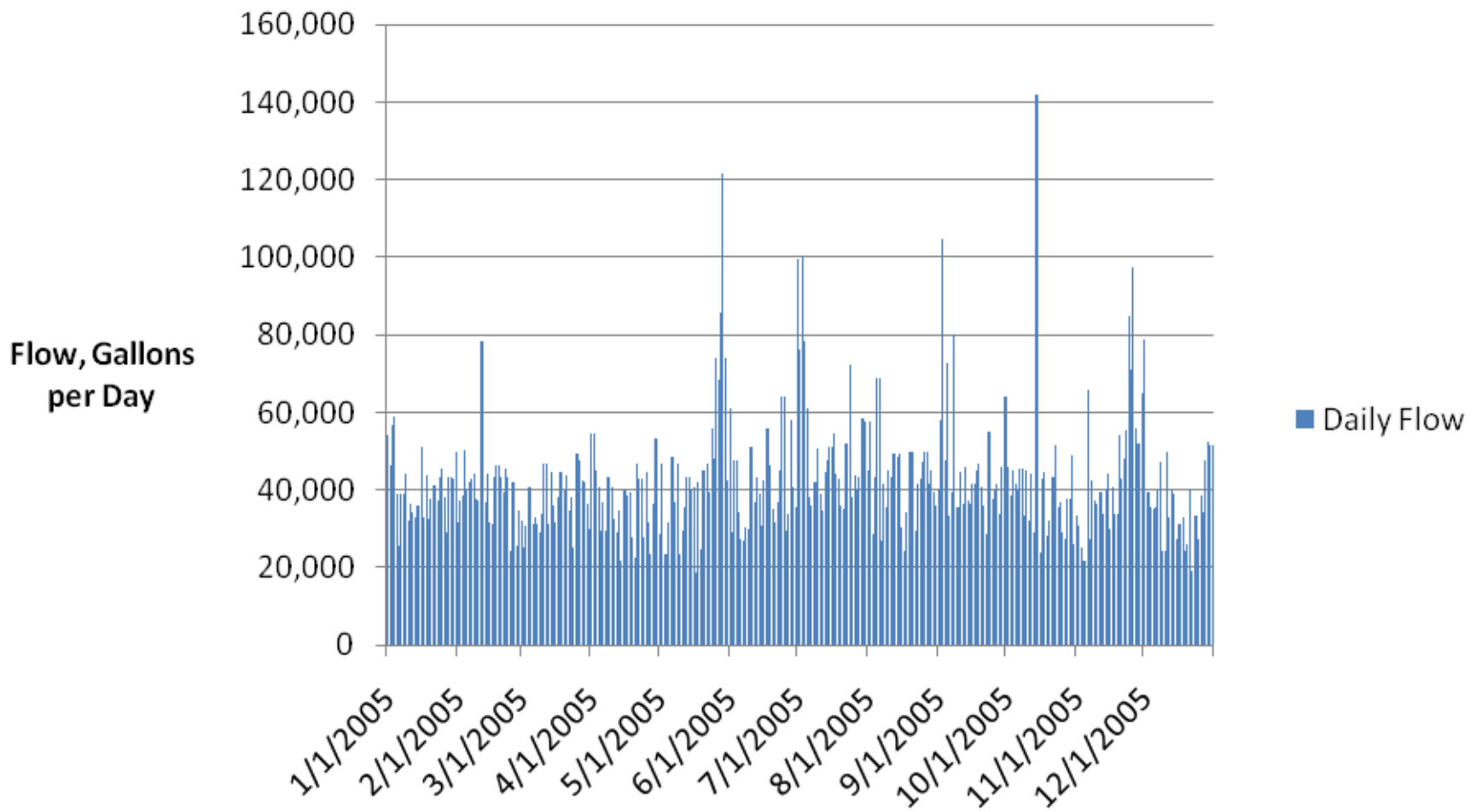
DAILY WRP FLOWS IN TO THE
SANDPIPER WRP IN 2005

FIGURE 2.2

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE

Table 2.3	Estimated Wastewater Flows for the Buckskin Sanitary District 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona				
	2010 Flow (mgd)		Buildout Flow (mgd)		
Village	**Average**	**Peak**	**Average**	**Peak**	**Seasonal Peak**
1	0.07	0.17	0.15	0.38	0.51
2	0.10	0.25	0.11	0.28	0.37
6	0.02	0.05	0.09	0.22	0.29
7	0.05	0.13	0.05	0.13	0.17
Northern Area Total	**0.24**	**0.60**	**0.40**	**1.01**	**1.34**
3	0.13	0.32	0.20	0.49	0.65
4	0.30	0.74	0.33	0.82	1.08
5	0.15	0.37	0.28	0.69	0.91
Southern Area Total	**0.58**	**1.43**	**0.81**	**2.00**	**2.64**
Total	**0.82**	**2.03**	**1.21**	**3.01**	**3.98**

2.2   PERFORMANCE CRITERIA

Appendix A contains the performance criteria that would be appropriate for the BSD.

2.3   PERMITTING AND REGULATORY ISSUES

2.3.1 Applicable Permits

The BSD will be required to secure applicable regulatory permits as listed in Table 2.4.

Table 2.4	Permits Required for WRP 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona
Responsible Agency	**Permit/Approval**
Arizona Department of Environmental Quality (ADEQ)	Aquifer Protection Permit (APP)
Western Arizona Council of Governments	208 Plan Amendment
LaPaz County	Air Quality Permit

WASTEWATER TREATMENT PLANT EVALUATION

3.1 INTRODUCTION

The BSD currently owns and operates a conventional wastewater treatment plant (WWTP) located north of Parker, Arizona. The Buckskin WWTP (formerly called the Sandpiper Wastewater Treatment Plant) was originally constructed to serve the Sandpiper Condominiums, and was first permitted for a maximum average monthly flow treatment capacity of 175,000 gallons per day (gpd). Due to several modifications to the treatment process, the plant currently has a maximum average monthly flow permitted capacity of 228,000 gpd.

Carollo performed a technical evaluation of the existing wastewater treatment plant to: (i) verify the plant rated capacity, and (ii) to determine whether there are potential cost-effective modifications that could increase the rated capacity of the existing facility, maximizing the use of existing facilities and equipment.

The resulting information is intended to support the master planning efforts for the BSD collection system and treatment facilities. Conceptual facility modifications mentioned in this analysis are based on an evaluation of the existing facilities in the context of current and projected regulatory requirements. Following approval of preliminary concepts by applicable regulatory agencies, further analysis and detailed design of facility modifications will be necessary prior to proceeding with construction.

3.2 PERMIT CONSIDERATIONS

The Arizona Department of Environmental Quality (ADEQ) sets forth the regulations pertaining to wastewater treatment effluent quality and effluent management in Arizona. Wastewater treatment plants must comply with two key treatment requirements: one set pertain to the Best Available Demonstrated Control Technologies (BADCT) requirements, and the other set pertain to the requirements for the different reclaimed water quality classes as defined in the Arizona Administrative Code (A.A.C.).

3.2.1 ADEQ BADCT Requirements

The recent ADEQ rules require that wastewater treatment plants in the State of Arizona must meet the conditions of Best Available Demonstrated Control Technology (BADCT). Treated effluent must meet or exceed the current standards set forth in the A.A.C., specifically as defined in R18-9 and R18-11. The BADCT treatment performance requirements are presented in Table 3.1.

Table 3.1   ADEQ BADCT Effluent Requirements
2011 Wastewater Master Plan Update
Buckskin Sanitary District, Arizona

Parameters	Effluent Limits [1]	
	Average Daily Flow < 250,000 gpd	Average Daily Flow > 250,000 gpd
pH	6.0 - 9.0	
BOD_5 (30-day average)	< 30 mg/L	
BOD_5 (7-day average)	< 45 mg/L	
TSS (30-day average)	< 30 mg/L	
TSS (7-day average)	< 45 mg/L	
BOD_5, $CBOD_5$, and TSS Removal Efficiency	85%	
Total nitrogen (as N) [2],[3]	< 10 mg/L	
Fecal coliform [3]		
Single sample maximum	800 cfu/100 mL	23 cfu/100 mL
Four out of last seven daily samples	200 cfu/100 mL	2.2 cfu/100 mL

R18-11-406(B-G) constituents including:
  Inorganic chemicals
  Organic chemicals
  Pesticides and polychlorinated biphenyls
  Radionuclides
  Fecal coliform
  Turbidity
	Numeric water quality standards must be met	

A.R.S. 49-243(I) regulated chemicals including:
  Known carcinogens
  Substances listed in the Resource Conservation and Recovery Act (RCRA)
  Any organic toxic pollutant the Director lists as a substantial short-term and long-term human health threat in minute amounts
	Removal to greatest extent possible without regard to cost	

Trihalomethanes	Minimize THM compounds generated as disinfection byproducts using chlorination, dechlorination	

Notes:
(1)   Reference: A.A.C. R18-9-B204.
(2)   Five-month rolling geometric mean.
(3)   BADCT standards allow for soil aquifer treatment if it can be proven that the required level of treatment is reached prior to effluent interfacing with the groundwater.

BADCT requirements apply not only to new wastewater treatment plants, but also to wastewater treatment plants that undergo major modifications or expand their treatment capacity as defined in A.A.C. R-18-9-B206. There are two conditions that require an existing facility to comply with current BADCT requirements as presented in Table 3.1:

- **An increase in the design flow.** The minimum design flow increase that triggers BADCT requirements depends on the permitted design flow of a wastewater treatment plant (A.A.C. R-18-9-A211). For the Buckskin WWTP, a 10 percent increase in the design flow (flow above 250,800 gpd) will require compliance with current BADCT requirements.

- **Addition of major facilities.** Requirements in A.A.C. R-18-9-B206 state that "An addition of a physically separate process or major piece of production equipment, building, or structure that causes a separate discharge to the extent that the treatment performance requirements for the pollutants addressed in A.A.C. R-18-9-B204 can practicably be achieved by the addition." The pollutants in A.A.C. R-18-9-B204 were presented in Table 3.1.

3.2.1.1 General Site Aesthetic Regulatory Requirements

ADEQ has developed specific criteria relative to setback requirements for the design of water reclamation facilities. These setback requirements have been recently revised per the Arizona Administrative Code Title 18, Chapter 9, Subpart B201 (A.A.C. R18-9-B201), as part of ADEQ's Aquifer Protection Permits (APP) process, with the associated requirements per the Best Available Demonstrated Control Technologies (BADCT) regulations.

For facilities with design flows between 100,000 and 500,000 gpd, such as the Buckskin WWTP, the minimum setback distances from the nearest property line are as follows:

- 500 feet if no odor, noise or aesthetic controls are provided; and

- 100 feet if full noise, odor, and aesthetic controls are provided.

The setbacks are defined by ADEQ in A.A.C. R-18-9-B201 as follows: "setbacks are measured from the treatment and disposal components within the sewage treatment facility to the nearest property line of an adjacent dwelling, workplace, or private property." According to ADEQ, full noise, odor, and aesthetic control means that:

- Noise due to the sewage treatment facility does not exceed 50 decibels at the facility property boundary on the A network of a sound level meter or a level established in a local noise ordinance;

- All odor-producing components of the sewage treatment facility are fully enclosed;

- Odor scrubbers or other odor-control devices are installed on all vents; and

- Fencing aesthetically matched to the area surrounding the facility.

For wastewater treatment plants built before the existence of the required setbacks established in the BADCT requirements, there is a possibility that the minimum setback of 100 feet mentioned above cannot be met. In those cases, ADEQ still requires that full noise, odor, and aesthetic controls be implemented, and that the expanded facilities do not further encroach into setback distances that existed before the modifications (A.A.C. R-18-9-B201).

Setbacks can also be decreased if allowed by local ordinances, or if waivers are obtained from affected property owners. Such waivers should include an acknowledgement by the affected property owner of the potential for noise and odor from the treatment facility.

3.2.2   ADEQ Reuse Applications

The required quality of treated effluent in Arizona is dependent on the intended end use of the reclaimed water. The ADEQ reuse regulations categorize reclaimed water into three main classes: A, B or C effluent. In addition, if nitrogen removal is provided, then the water can be classified as A+ or B+. Class A+ water essentially has unlimited options for water reuse applications (except for potable water supply), while Class B+, though unacceptable for use at schools, parks and recreational lakes, is adequate for golf courses and other restricted-access landscape irrigation uses. The ADEQ Reclaimed Water Quality Standards are presented in Table 3.2.

Table 3.2	ADEQ Reclaimed Water Quality Standards 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona		
	Effluent Limits		
Parameter	**Class A+[1]**	**Class B+[2]**	**Class C[3]**
Secondary treatment	X	X	Stabilization ponds with 20-day detention
Filtration	X	NR	NR
Denitrification	X	X	NR
Disinfection	X	X	With or without
Total Nitrogen (as N)[4, 5]	< 10 mg/L	< 10 mg/L	N/A
Turbidity			
Daily (24-hour) average	2 NTU	N/A	N/A
Single sample maximum	5 NTU	N/A	N/A
Fecal Coliform			
Single sample maximum	< 23 cfu/100 mL	< 800 cfu/100 mL	< 4,000 cfu/100 mL
Four out of last seven daily samples	Non-detect	< 200 cfu/100 mL	< 1,000 cfu/100 mL
Notes: X = Required NR = Not Required (1) Reference: A.A.C. R18-11-303 (2) Reference: A.A.C. R18-11-305 (3) Reference: A.A.C. R18-11-307 (4) Five sample geometric mean			

Table 3.2	ADEQ Reclaimed Water Quality Standards 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona		
	Effluent Limits		
Parameter	**Class A+**[1]	**Class B+**[2]	**Class C**[3]
(5) Not required for Class A and Class B			

3.2.3 Implications to Existing Permit

The existing Buckskin WWTP is currently operating under an Aquifer Protection Permit (APP No. P-100804) that requires meeting Class A reclaimed water quality standards. The permit specifies that all effluent will be consumptively reused under a Type 2 Reclaimed Water Permit. The primary difference between Class A+ and Class A reclaimed water quality, in terms of treatment requirements, is the level of nitrogen removal required.

Modifications to the Buckskin WWTP that result in an increased plant capacity (i.e., re-rating) beyond a capacity of 250,800 gpd will require an application for a significant amendment to the existing APP permit (per AAC R18-9-A211), and will trigger compliance with current BADCT requirements. The more significant impacts of BADCT compliance are:

- The facility would need to meet a total nitrogen limit of less than 10 mg/L. The current permit does not require nitrogen removal. The current process has provisions for nitrogen removal, but the performance of the existing nitrogen removal system has not been verified. Modifications to the secondary process will be required to achieve nitrogen removal in the aeration basins.

- Full odor and noise control will be required at the site. Additional facilities required would not be able to encroach into the existing setbacks.

The treatment plant analysis and associated recommendations outlined below were provided to maintain compliance with the existing permit requirements, to avoid triggering the need for a significant permit amendment and BADCT requirements.

3.3 WASTEWATER FLOW AND CHARACTERISTICS

3.3.1 Wastewater Quantity

The existing Buckskin WWTP was originally designed for 175,000 gpd average daily flow (ADF). However, the facility has been re-rated to a capacity of 228,000 gpd due to some equipment modifications, mainly to the aeration system. Capacity verification analysis was performed at an ADF of 228,000 gpd. A capacity analysis was also performed to evaluate a treatment capacity of 250,000 gpd, which is slightly below the trigger for a significant permit amendment (250,800 gpd).

3.3.2 Wastewater Quality

There was no data available for an analysis of the existing wastewater quality coming into the Buckskin WWTP. Therefore, two scenarios were developed in order to evaluate the plant capacity. Each scenario is defined below:

- **Original Design Scenario** – The original design scenario is based on a biological oxygen demand (BOD) of 200 mg/L, as used in the original plant design (Waste Treatment Plant Specification Data Sheet, Marolf, Inc., 1978). An influent BOD concentration of 200 mg/L could be considered "typical" for domestic wastewater. However, in the last 10 years there has been an increasing trend in wastewater concentrations due to several factors including reduced water usage, larger fraction of commercial flows, among others. Therefore, this scenario is considered to be a lower end for wastewater concentrations in current conditions. Other wastewater components such as Total Suspended Solids (TSS), Volatile Suspended Solids (VSS), Total Kjeldahl Nitrogen (TKN), ammonia (NH_3-N), were estimated based on a typical domestic wastewater composition.

- **High-Strength Scenario** – The high-strength scenario is based on an influent BOD concentration of 320 mg/L. This influent BOD concentration is consistent with values recently observed and used for design in other Arizona communities. Specifically, the criteria utilized for this analysis are based on concentrations recently used for design at the City of Surprise Special Planning Area No. 2 Water Reclamation Facility, which has similar land use and population characteristics.

The wastewater characteristics used in the capacity analysis are presented in Table 3.3.

Table 3.3	Wastewater Characteristics Assumed for Capacity Analysis 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona	
Criteria	**Original Design Scenario**	**High-Strength Scenario**
Biological Oxygen Demand (BOD), mg/L	200	320
Total Suspended Solids (TSS), mg/L	220	352
Total Kjeldahl Nitrogen (TKN), mg/L	32	52
Ammonia (NH_3-N), mg/L	21	34

Both scenarios were modeled and evaluated in an effort to evaluate the plant capacity and perform a sensitivity analysis based on a range of wastewater characteristics. The results of the analyses are presented in the subsequent sections.

3.3.3 Flow and Load Peaking Factors

Based on the preliminary analysis outlined above, varying flows were selected for modeling and subsequent establishment of facility design criteria. Table 3.4 presents the peaking factors used for the analysis and the corresponding flows. The peaking factor used is based

on daily flow data for 2005, obtained from BSD. The actual 2005 peak day factor (peak daily flow divided by annual average daily flow) was 3.3. However, for the purposes of plant evaluation the peaking factor used was reduced to 3.0, based on the assumption that as the service area grows the peaking factor will likely decrease.

Table 3.4	Peaking Factors and Hydraulic Flows 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona		
		Flow (gpd)	
Criteria	Peaking Factor [1]	Permitted Capacity	Expanded Capacity
Average Day Flow (ADF)	-	228,000	250,000
Peak Flow (PF)	3.0	684,000	750,000

3.4   PROCESS EVALUATION

Based on the flow projections and peaking factors outlined in Table 3.4, a detailed process evaluation was completed for the Buckskin WWTP to verify the plant capacity and determine whether there are opportunities to increase the permitted capacity of the plant. The aeration basins, aeration system, secondary clarifier, filters, chlorine contact basin, and aerobic digester were evaluated using a process model (*Biotran)*. See Appendix B for detailed process evaluation.

3.5   SUMMARY RECOMMENDATIONS

The existing Buckskin WWTP has sufficient capacity to treat the permitted average flow of 228,000 gpd. Some system deficiencies were found, particularly under peak flow conditions, which require further attention. Because there were no wastewater quality records available for this analysis, the plant evaluation was based on two loading scenarios that encompass the possible range of wastewater characteristics observed at the Buckskin WWTP. For the permitted flow of 228,000 gpd, the following areas that could require further attention are listed below.

- **Aeration System.** The existing blower system is undersized for peak flow conditions at the original design loading scenario, and for average and peak flow conditions at the high-strength loading scenario. The diffuser system is undersized for the high-strength loading scenario. Additional blower capacity is recommended. Additional diffusers may also be necessary depending on confirmation of the actual influent wastewater characteristics.

- **Secondary Clarifier.** The single secondary clarifier does not meet redundancy requirements set by ADEQ Engineering Bulletin No. 11. The secondary clarifier is critical to the operation of the activated sludge system, and permit compliance is at risk if the secondary clarifier presents a major failure.

- **Denitrification Filters.** The carbon feed system was not operational at the time of this study. Carbon feed is necessary for denitrification. However, the plant does not currently have total nitrogen permit limits and does not require denitrification. Should nitrogen limits be imposed in the future, a carbon feed system will be required to operate the denitrification filters. Regular data collection is recommended to verify the performance of the filters.

The Buckskin WWTP plant may be able to treat an average flow of up to 250,000 gpd. The main factor limiting plant capacity is the performance of the secondary clarifier. With regular plant data for mixed liquor settleability parameters, the actual capacity of the secondary clarifier could be verified. The same recommendations regarding the aeration system, secondary clarifier redundancy, and denitrification filters mentioned above apply for an average flow of 250,000 gpd.

Nitrogen removal is currently not required in the plant permit. Nitrogen removal per BADCT requirements could be triggered with a significant permit amendment application, which would be required when re-rating the plant for a flow higher than 250,000 gpd. Nitrogen removal could be achieved either in the denitrification filters, or in the aeration basins. The denitrification filters require field verification of their performance and the installation of the carbon feed system. The aeration basins require retrofitting dedicated anoxic zones into the surge tank and an internal mixed liquor return.

The physical condition of the structure and equipment at the Buckskin WWTP was not evaluated as part of this analysis. The useful life of the equipment and structures need to be taken into account when making decisions regarding possible expansions or modifications to the existing plant. It is worth mentioning that a major infrastructure or equipment failure (e.g., secondary clarifier) at the plant could cause non-compliance with permit requirements.

ALTERNATIVE DEVELOPMENT

4.1   WASTEWATER COLLECTION SYSTEM

A preliminary layout of gravity sewer pipes was developed in the 2007 master plan for the village areas. There have been no revisions to the layout within the neighborhoods in this master plan update. Within each village, an interceptor is planned along the highway or main road within the village to collect wastewater flows and deliver the flows to a wet well where a lift station will gather the flows and send them to an adjacent village or to a water reclamation plant (WRP). In order to develop and evaluate overall collection system concepts, assumptions were made regarding number of WRPs and their general locations. Figure 4.1 shows a conceptual layout of the primary collection system with one WRP. Figure 4.2 shows a conceptual layout of the primary collection system with two WRPs.

4.2   WATER RECLAMATION PLANTS

The existing Buckskin WWTP is currently near the end of its useful life and in need of major upgrades to prolong the life of the plant. The WWTP has significant deferred maintenance that is expected to limit how long the WWTP will be able to operate successfully. Given the existing conditions, the existing WWTP must be operated and maintained until a new WRP can be constructed to treat the wastewater. Then a new lift station would be constructed to transfer wastewater to a new WRP for treatment.

As addressed in the previous master plan, the BSD faces several major challenges in identifying viable locations and sites for proposed WRPs. Those challenges include the spread out, "linear" configuration of the service area along the Colorado River; and the lack of available, private property. As described in Section 4.1 and as shown in Figure 4.1 and Figure 4.2, developing a collection system to serve the District will require significant infrastructure, including multiple lift stations and long runs of gravity sewers and force mains. Developing smaller, more "localized" WRPs would somewhat reduce the linear infrastructure requirements, and the operational challenges associated with the long pipe runs. However, additional physical challenges and costs are associated with developing multiple treatment facilities. One of those challenges is effluent management. For the BSD, effluent management may be the most significant determining factor in locating and developing WRP(s).

A major investment and long-term commitment is required for developing a new WRP site. Site development, permitting, and infrastructure all require significant effort and investment. Flexibility for future improvements and expansion must be accounted for in committing to specific sites. Figure 4.3 shows a typical layout for a WRP with treatment capacity in the 0.5-0.75 mgd range.

Legend

— Highway

⊠ WRP

▪ Lift Station

— Force Main

— Gravity Main Excluding Residential Mains

Colorado River

Proposed Golf Course WRP
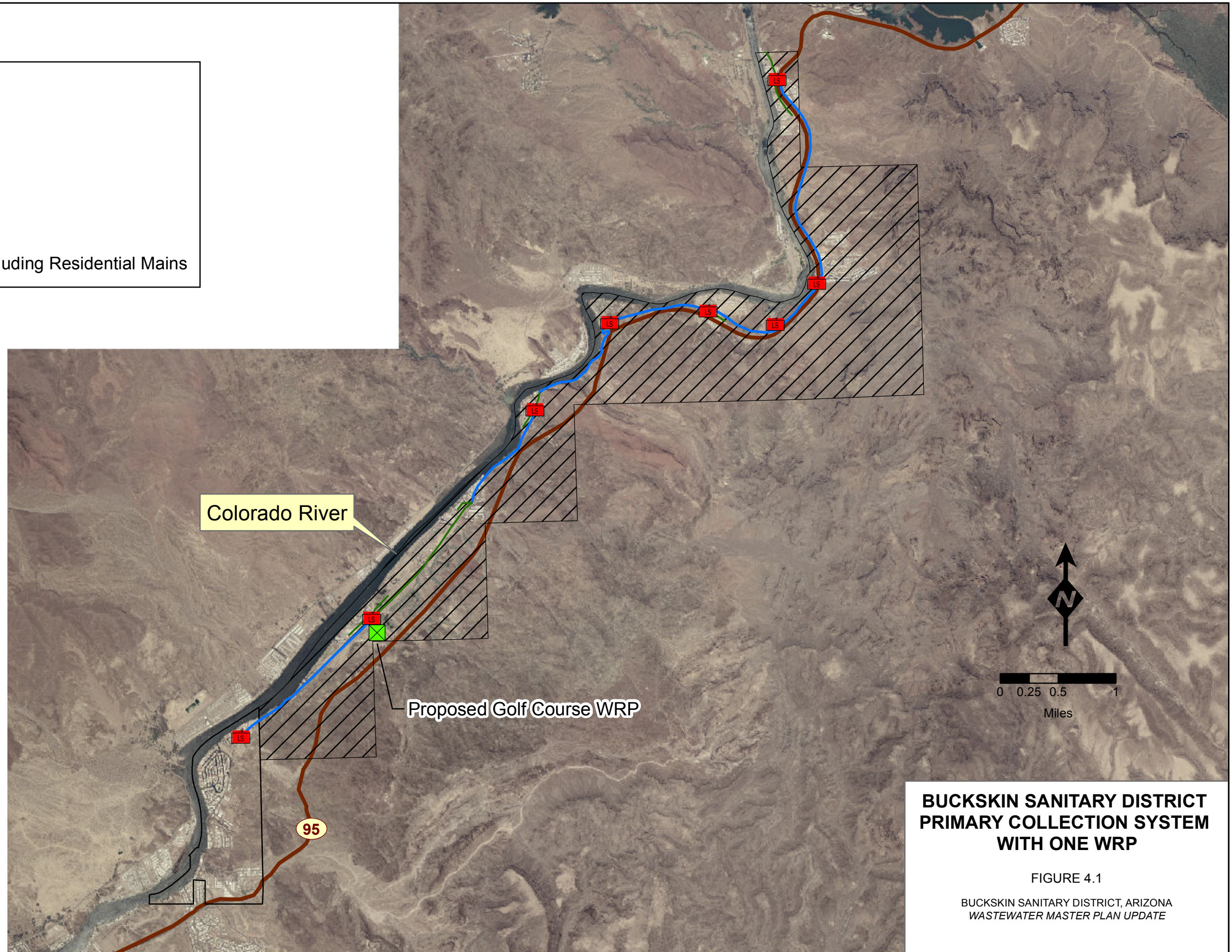
**BUCKSKIN SANITARY DISTRICT
PRIMARY COLLECTION SYSTEM
WITH ONE WRP**

FIGURE 4.1

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE

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0   0.25 0.5   1
Miles

M:\Client\Buckskin SD\Analysis\GIS\MXD\BSD_One_WRP_9-3_rev.mxd

Legend

— Highway
⊠ WRP
🟥 Lift Station
— Force Main
— Gravity Main Excluding Residential Mains

Colorado River

Proposed Castle Rock / State Park WRP

Proposed Golf Course WRP

95

N

0 0.25 0.5 1
Miles
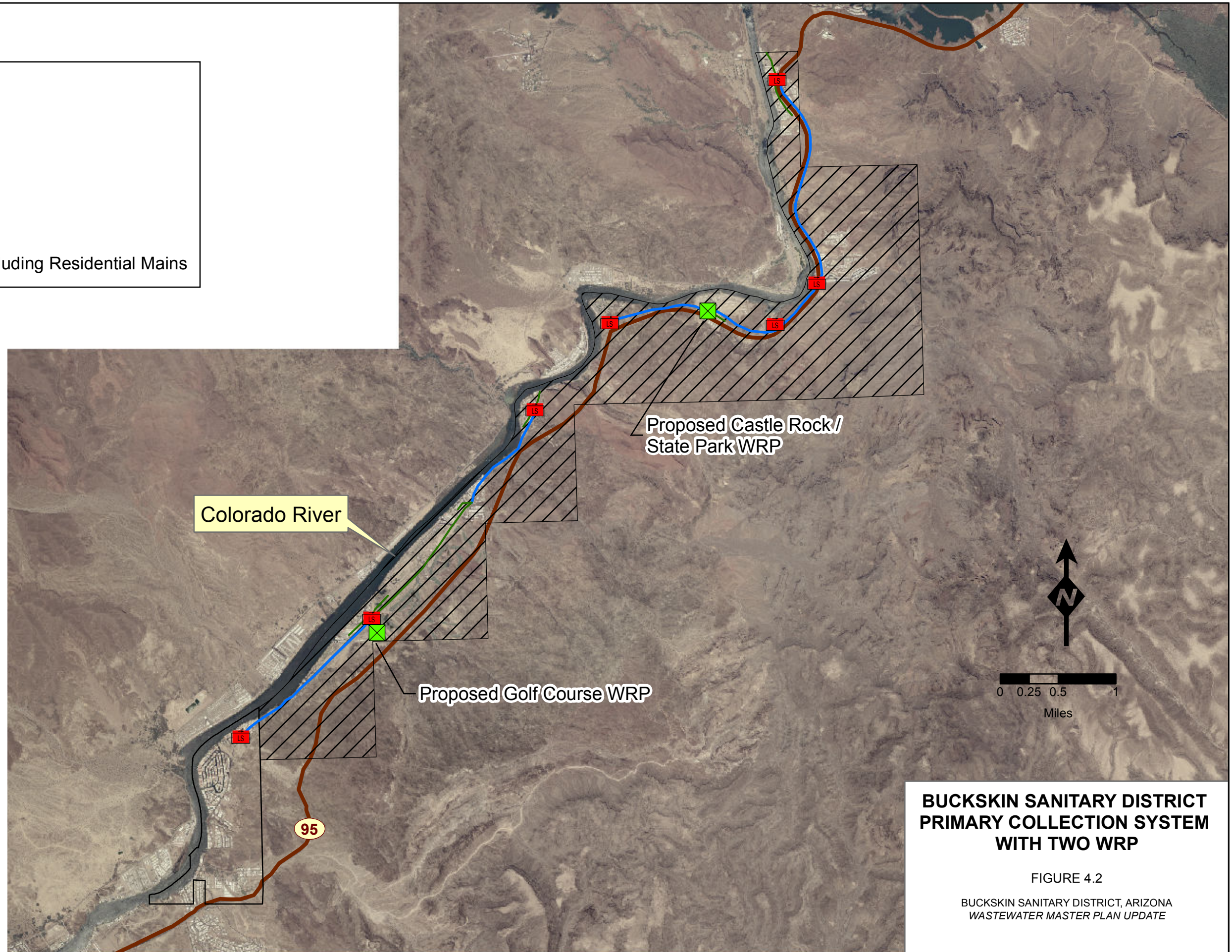
**BUCKSKIN SANITARY DISTRICT
PRIMARY COLLECTION SYSTEM
WITH TWO WRP**

FIGURE 4.2

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE

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M:\Client\Buckskin SD\Analysis\GIS\MXD\BSD_Two_WRP_9-3_rev.mxd

500'

400'

EFFLUENT
PUMP
STATION

DISINFECTION

CLARIFIER
NO. 1

AERATION
BASIN
NO. 1

OPERATIONS
BUILDING

TERTIARY
FILTERS

BLOWER
BUILDING

SPLITTER BOX/
RECYCLE PUMP
STATION

FINE
SCREENS

ODOR
CONTROL

AEROBIC
DIGESTER

CLARIFIER
NO. 2

AERATION
BASIN
NO. 2

0   20'   40'   80'
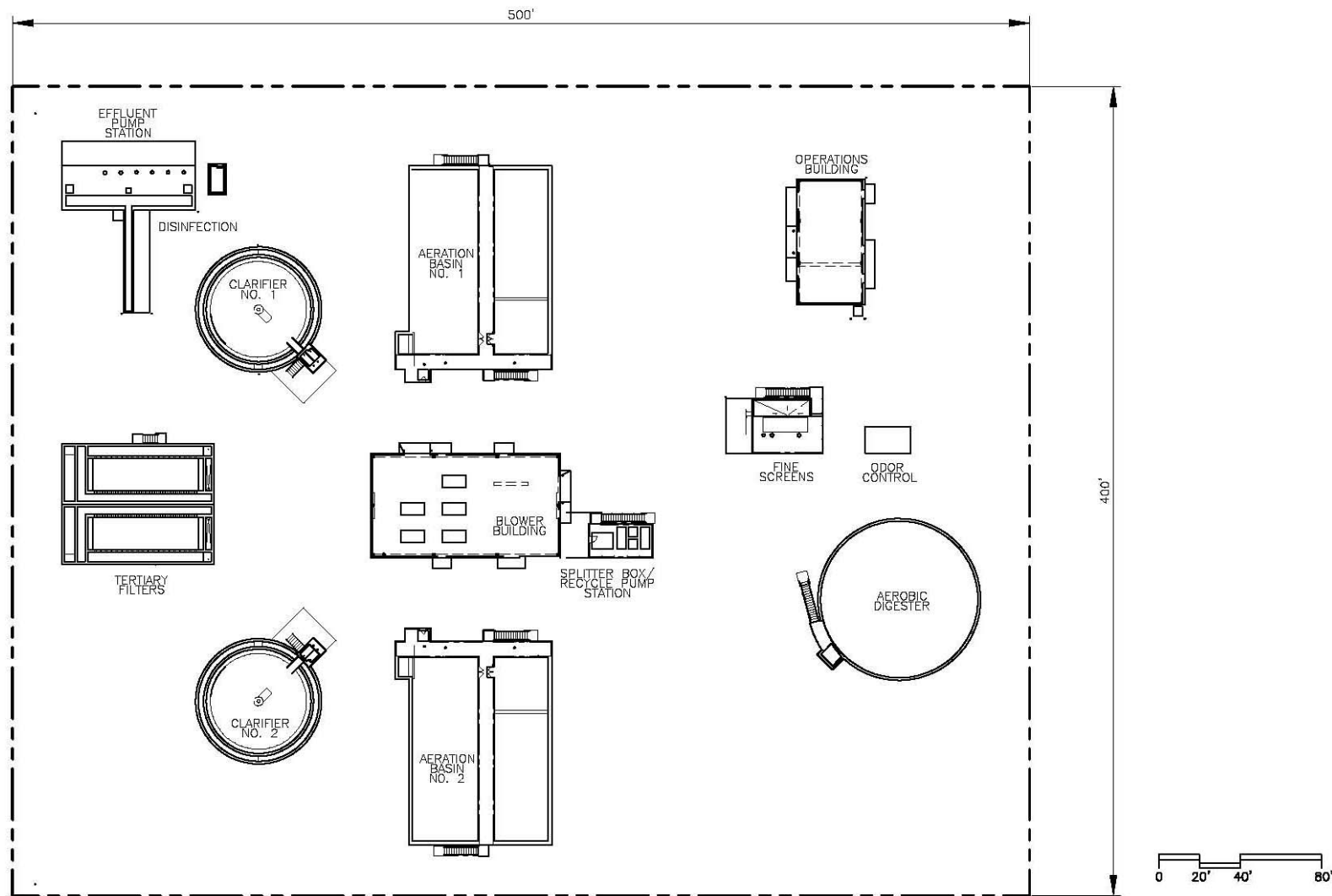
TYPICAL WRP LAYOUT

FIGURE 4.3

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE

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4.3   RECLAIMED WATER STRATEGIES

The location and treatment methods for a WRP depend in a large measure upon how the effluent from the WRP will be managed. The county golf course currently takes reclaimed water from the Buckskin WWTP to use for irrigation at the golf course. Approximately 50,000 gallons per day are delivered to the golf course in 2011, although there are days that the golf course is not able to take the flow. The golf course uses up to 600,000 gallons/day during the peak of the irrigation season.

The estimated average daily flow for all existing development in the BSD (assuming the entire area had sewer service) is approximately 0.81 mgd. Therefore, more wastewater flow would be generated than the golf course could use, even during the peak irrigation season. One option to expand the use of irrigation for effluent disposal would be to irrigate the green areas in Castle Rock and the Buckskin Mountain State Park. To minimize the infrastructure needed to deliver reclaimed water, a WRP would be located near the lands that would be irrigated. With irrigation reuse being such an important component of effluent management for the BSD, high priority WRP sites are identified in the area of the existing golf course irrigation and in the potential irrigation areas of Castle Rock and the State Parks. A new WRP near the golf course would collect wastewater from Villages 3, 4, and 5 under a two-WRP scenario, or the entire BSD service area under a one-WRP scenario. Under the two-WRP scenario, a WRP in the Castle Rock/State Parks area would treat wastewater from Villages 1, 2, 6, and 7.

As previously noted, the BSD is currently permitted for effluent disposal via consumptive use irrigation at the golf course. Although irrigation reuse is a very viable beneficial use of effluent in our arid southwestern climate, a backup plan is normally required for seasonal and other times of reduced irrigation demand. Many Arizona wastewater facilities utilize groundwater recharge as a backup effluent disposal option. However, the BSD is located adjacent to the Colorado River. Within the area of influence of the Colorado River, reclaimed water cannot be recharged into the aquifer. This is due to the shallow groundwater being directly connected to the river. Other options potentially available for effluent disposal in times of low irrigation demand include evaporation basins and/or wetland/riparian areas. Wetland/riparian areas can provide significant benefits for effluent management and as backup to irrigation reuse. Potential benefits include effluent storage, evaporation, and evapotranspiration. In addition, wetland/riparian areas could be used to enhance the golf course, the State Parks, and as an amenity to other public access areas used for passive recreation along the river.

Discharge to the Colorado River could be an option for effluent disposal. The Colorado River has stringent water quality requirements for any water that is discharged. Therefore, obtaining a permit to discharge to the Colorado River is expected to be difficult and costly. It is important to note that permitting for discharge to the Colorado River may require advanced treatment beyond the A+ quality effluent addressed in this report. As previously

noted, the District's existing Aquifer Protection Permit specifies that all effluent will be consumptively reused. Based on the regulatory requirements and costs associated with river discharge, and long-term sustainability of the effluent as a water resource, it is recommended that the District develop other effluent management options in lieu of river discharge.

4.4 RECLAIMED WATER FLOWS

Irrigation demands vary seasonally, and reclaimed water availability is more constant, so the amount of flow that would need to be managed (in excess of irrigation demand) will vary month to month. Figure 4.4 shows the amount of reclaimed water that could be used for golf course irrigation, park irrigation, and the amount of reclaimed water that would need to be managed on a monthly basis, assuming a total wastewater average daily flow of 0.81 mgd (all existing development with sewer service). The irrigation demands were estimated based on land areas taken from aerial photographs. Figure 4.5 shows the seasonal flow allocations assuming occupancy of 60 percent. Figure 4.6 shows the additional reclaimed water flows that could be available in the future under buildout conditions. These figures show that, under estimated buildout conditions, the District will need to develop effluent management options (beyond the assumed irrigation reuse) for utilization approximately 8 to 12 months of each year.

4.5 SHORT TERM STORAGE

Wastewater flows will vary significantly between holiday peak and off peak times. Because peak flows can last as long as a week, the WRP will need to be able to treat wastewater at the peak flows. The BSD wastewater flow patterns are very different from most communities because of these high peaks that last for several days. Most communities have only a daily peak, which is significantly lower than the peaks experienced by the BSD. Consideration was given to providing short-term storage in lift station wet wells as a means of averaging out the peaks, but storage requirements for this wastewater became excessive. Therefore, lift station wet wells will have a typical size for a given flow condition, and the lift stations will have the capability of pumping both low and high flows.

Reclaimed effluent storage was considered as a means of balancing high and low flows to maximize the amount of water available for irrigation. However, as previously noted, overall effluent management will have to balance available flow, storage, peak seasonal irrigation demands, and effluent disposal during low irrigation periods.
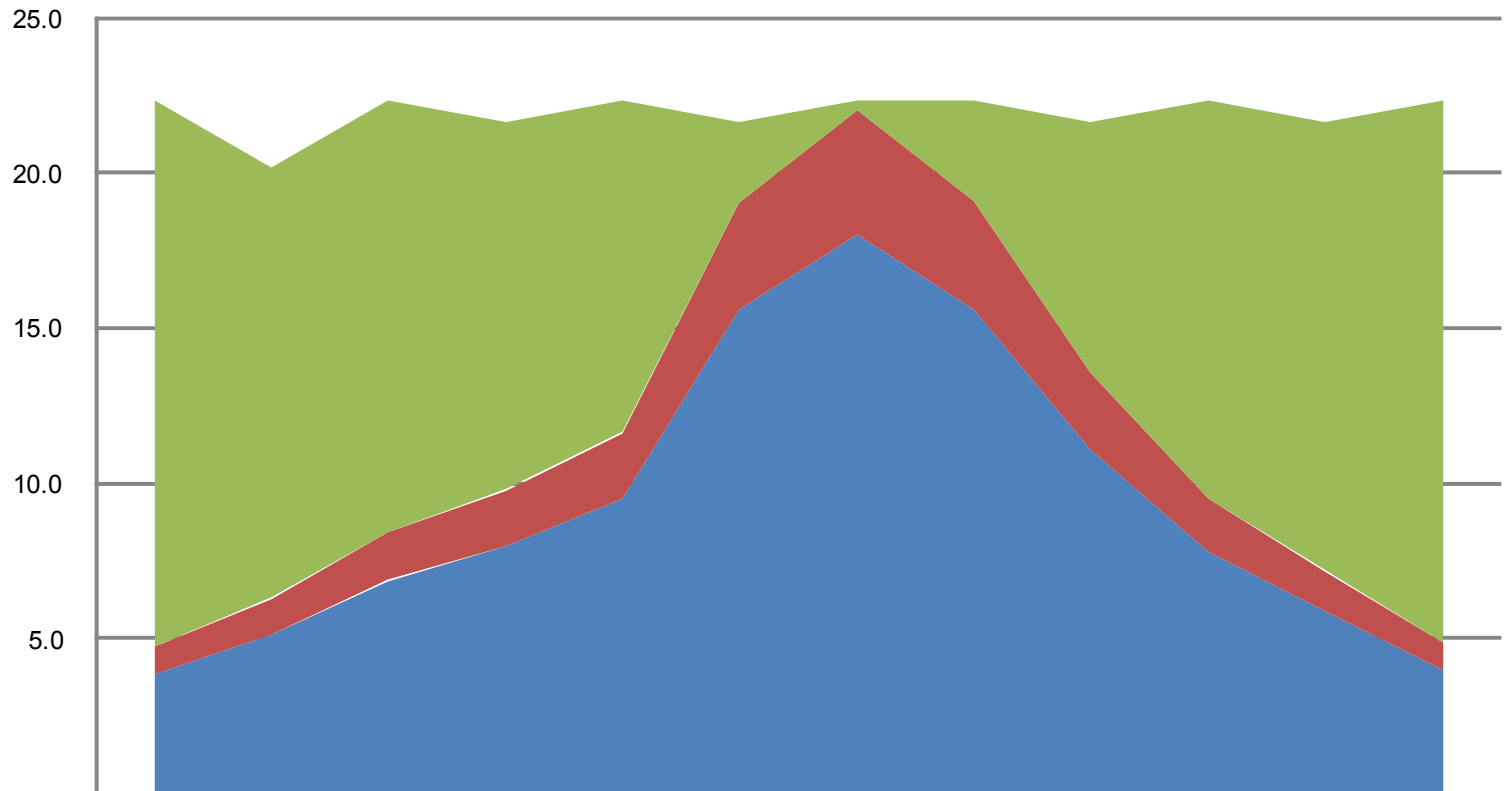
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Reclaimed Effluent to be Managed	17.6	13.9	14.0	11.8	10.7	2.6	0.3	3.3	8.1	12.9	14.4	17.5
Northern Parks Irrigation	0.9	1.1	1.5	1.8	2.1	3.5	4.0	3.5	2.5	1.7	1.3	0.9
Golf Course Irrigation	3.9	5.1	6.8	8.0	9.5	15.5	18.0	15.6	11.1	7.7	5.9	3.9

**BUCKSKIN SANITARY DISTRICT RECLAIMED WATER FLOW
2011 POTENTIAL FLOWS, FULL OCCUPANCY**

FIGURE 4.4

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE

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	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Reclaimed Effluent to be Managed	9.2	6.4	5.6	3.7	2.4	-	-	-	-	4.5	6.3	9.1
Northern Parks Irrigation	0.9	1.1	1.5	1.8	2.1	3.5	4.0	3.5	2.5	1.7	1.3	0.9
Golf Course Irrigation	3.9	5.1	6.8	8.0	9.5	15.5	18.0	15.6	11.1	7.7	5.9	3.9

Reclaimed Water FLow, MG/Month
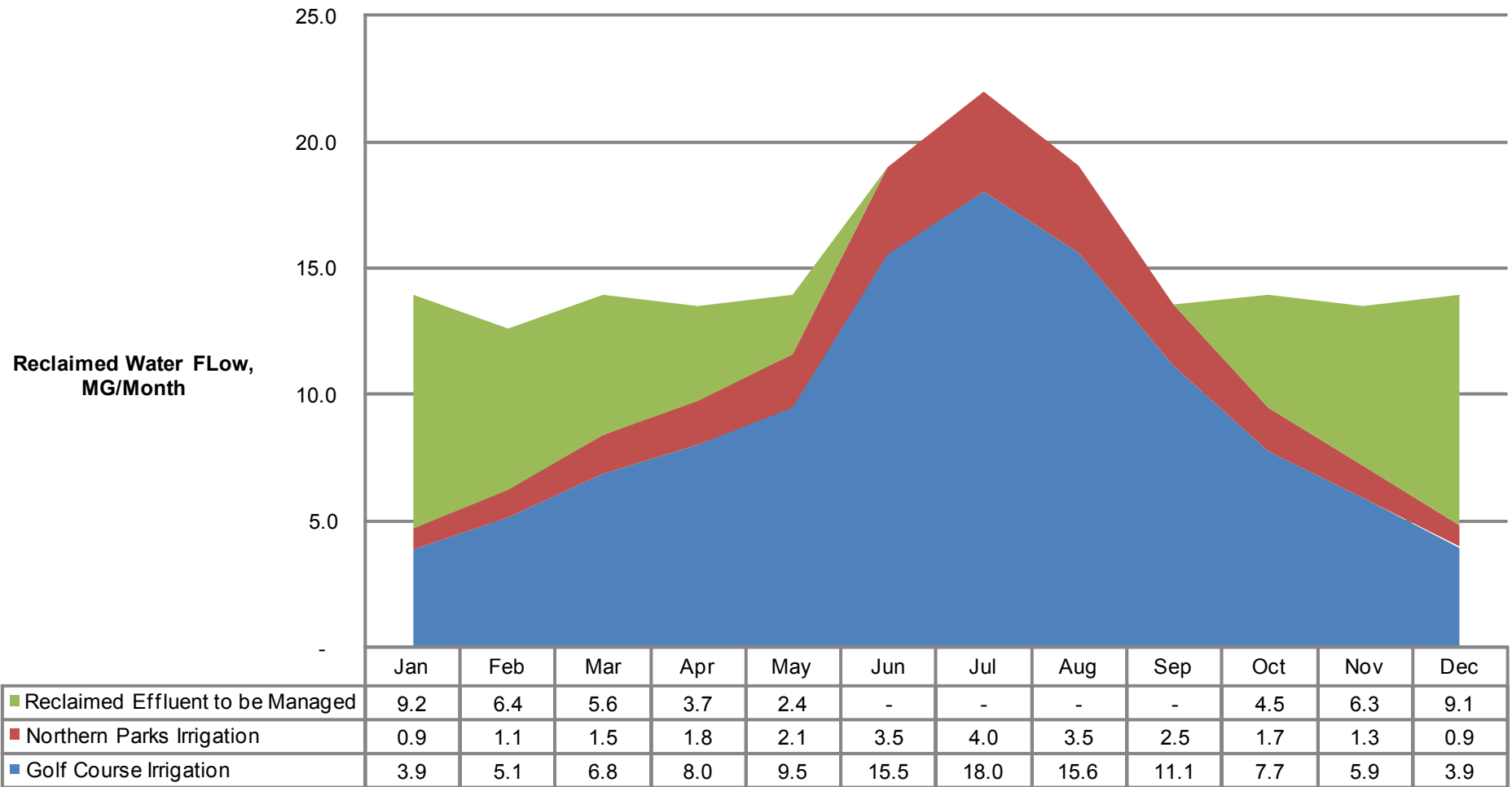
**BUCKSKIN SANITARY DISTRICT RECLAIMED WATER
2011 POTENTIAL FLOWS, 60% OCCUPANCY**

FIGURE 4.5

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE
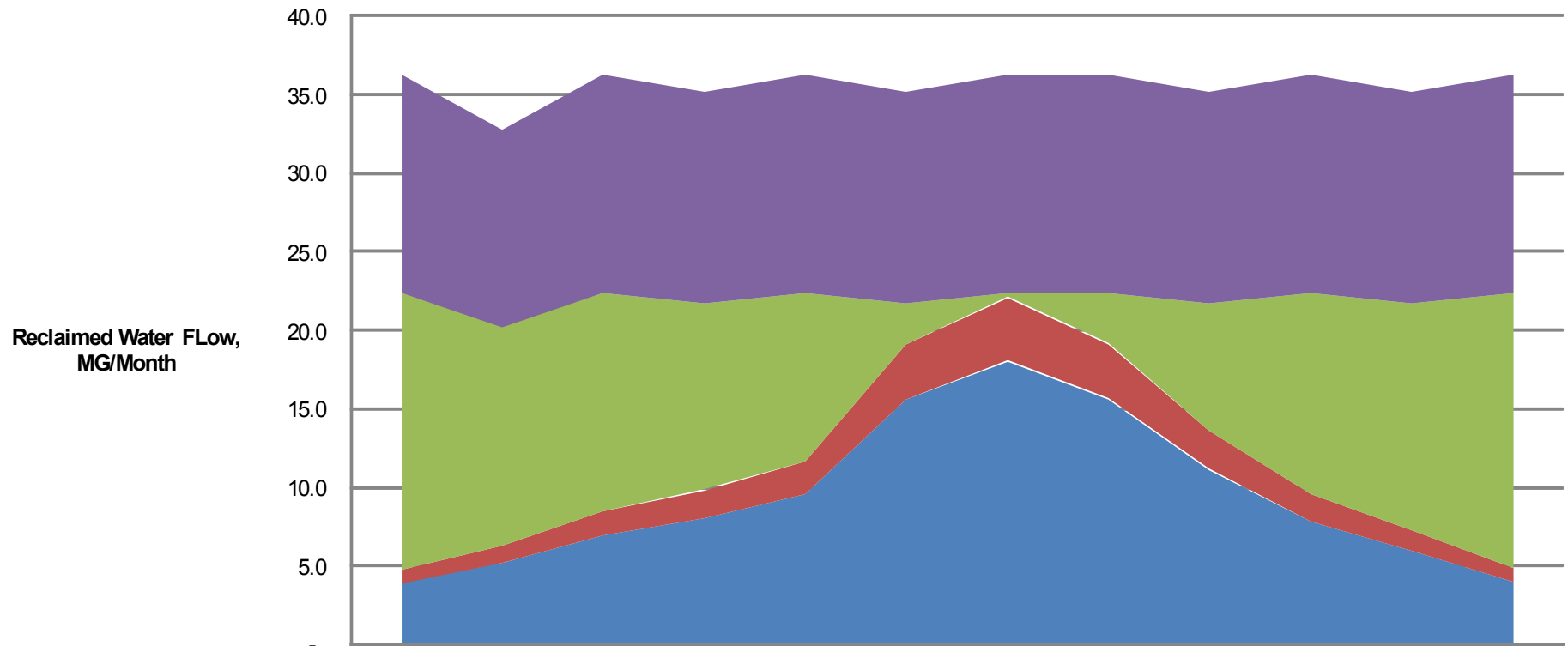
Reclaimed Water FLow, MG/Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
■ Potential Additional Flow from Growth	14.0	12.6	14.0	13.5	14.0	13.5	14.0	14.0	13.5	14.0	13.5	14.0
■ Reclaimed Effluent to be Managed	17.6	13.9	14.0	11.8	10.7	2.6	0.3	3.3	8.1	12.9	14.4	17.5
■ Northern Parks Irrigation	0.9	1.1	1.5	1.8	2.1	3.5	4.0	3.5	2.5	1.7	1.3	0.9
■ Golf Course Irrigation	3.9	5.1	6.8	8.0	9.5	15.5	18.0	15.6	11.1	7.7	5.9	3.9

BUCKSKIN SANITARY DISTRICT RECLAIMED WATER FLOW ALLOCATION AT BUILDOUT, 1.3 MGD AVERAGE DAILY FLOW

FIGURE 4.6

BUCKSKIN SANITARY DISTRICT, ARIZONA
WASTEWATER MASTER PLAN UPDATE

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pw://Carollo/Documents/Client/AZ/Buckskin/8631A00/Deliverables/Fig 4-06

4.6   WATER RECLAMATION PLANT

Water reclamation plant(s) for the BSD will need to produce A+ quality effluent. As mentioned above, the peak daily flows may be up to 2.5 times the lower daily flows. Peak hourly flows have not been quantified at this time. Due to the uncertainties associated with estimating flows, a flow study is recommended to better understand the average and peak flows coming from each village area. The total WRP capacity for the BSD should be capable of handling the peak wastewater flows even though there are many times that flows will be considerably lower.

Using a typical per capita flow value of 65 gpcd, and a current population of 9,800, the average daily wastewater flow is estimated to be 0.81 mgd. To allow for uncertainties in flow projections and growth, an initial average day treatment capacity of 0.9 mgd is recommended. Package WRPs are constructed in modular sections; so, even the 0.9 mgd can be phased to coincide with the number of customers that are converted from septic systems to the BSD collection system. Over time as villages are added to the collection system, the BSD will be able to measure flows more accurately and add modular treatment capacity as needed. If two WRPs are constructed, the WRP at the golf course would have a capacity of 0.6 mgd and the Castle Rock/State Parks WRP would have a capacity of 0.3 mgd.

4.7   VAULT AND HAUL STRATEGIES

The BSD may want to begin adding customers to increase the rate base to help pay for the WRP improvements. Therefore, the WRP(s) could be constructed before all of the primary collection system is in place. The secondary collection system constructed in each village would end at a vault that would become the lift station wet well for the village. Wastewater flows into these vaults could be hauled to the WRP for treatment.

4.8   WRP ALTERNATIVES

Four alternatives have been identified to illustrate different ways to construct a wastewater conveyance, treatment, and disposal system. These alternatives are explained below.

1.   **Alternative 1.** All wastewater flows (0.9 mgd) would be treated at a single WRP to be located south of the golf course (site of existing reclaimed water ponds). This WRP would receive wastewater flows from a lift station at the existing Buckskin WWTP location as well as an influent pump station in Village Area 4. Reclaimed water would flow via gravity to the golf course for irrigation, and excess effluent would be managed as previously noted.

2. **Alternative 2.** This alternative is the same as Alternative 1 except that the WRP would be located on the east side of the highway on land that was designated by the County for a BSD WRP.

3. **Alternative 3.** The golf course WRP would be located near the current reclaimed water ponds and have a capacity of 0.6 mgd. The Castle Rock/State Parks WRP would be located at a specific site yet to be determined, and have a capacity of 0.3 mgd.

4. **Alternative 4.** This alternative is identical to Alternative 3 except that the golf course WRP would be located east of the highway on the County identified site.

CAPITAL IMPROVEMENT PLAN

5.1   UNIT COSTS

Unit costs have been developed for the capital improvements that are being recommended for this master plan. This cost estimate was prepared in accordance with the guidelines of the AACE International (Association for the Advancement of Cost Engineers) for a Class 5 estimate. According to the definitions of AACE International, the Class 5 estimate is defined as:

"CLASS 5 Estimate. Generally prepared based on very limited information, where little more than proposed plant type, its location, and the capacity are known. Strategic planning purposes, such as but not limited to, market studies, assessments of viability, evaluation of alternative schemes, project screening, location and evaluation of resource needs and budgeting, long range capital planning, etc. Some examples of estimating methods used would be estimating methods such as cost/capacity curves and factors, scale up factors, parametric and modeling techniques. Typically, very little time is expended in the development of this estimate. The typical expected accuracy range for this class estimate is -20 percent to -50 percent on the low side and +30 percent to +100 percent of the high side."

Table 5.1 lists the unit costs that have been used to calculate construction costs. To obtain project costs, a factor of 30 percent is applied to the construction cost to account for engineering design, construction management, and a contingency. Costs are based on the ENR 20 Cities Average. The May 2011 ENR CCI index is 9035.

Table 5.1	Unit Costs used to Calculate Construction Costs
	2011 Wastewater Master Plan Update
	Buckskin Sanitary District, Arizona

Force Mains	
Diameter (in)	Construction Cost per LF
4	$84.21
6	$88.37
8	$93.64
10	$99.83
12	$106.36
16	$115.54

Table 5.1	Unit Costs used to Calculate Construction Costs 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona	
Gravity Mains		
Diameter (in.)	Construction Cost per LF	
8	$84.11	
10	$90.93	
12	$93.97	
15	$96.47	
Lift Station		
Flow Range, mgd	Construction Cost per MG	
0.1 - 0.9	$508,200	
1 - 2	$443,520	
Hard Rock Excavation		
2.5 - 5	$415,800	
Machine	Unit Cost per CY	
D8 Dozer, Class C (Hard Dig), Grade, Cut, Fill & Compact, 600' Haul	$10	
D4 Dozer, Class C (Hard Digging), Grade, Cut, Fill & Compact, 300' Haul	$25	

5.2   INFRASTRUCTURE RECOMMENDATIONS

Table 5.2 shows the estimated capital cost of each of the alternatives that were identified in Chapter 4. Based on the capital cost comparison, Alternative 1, the single WRP option, is the least total cost. However, implementation of Alternative 1 would require a "comprehensive" project approach to providing overall wastewater service to the District, including funding for the entire program. This approach would also delay actual sewer service to the north area of the District. The District may want to consider a version of the two-WRP alternative, and begin partial development of the north WRP. This approach and phasing would require less initial funding, would provide some "immediate" service in the north area, and could expand the rate base in the near term.

Table 5.2	Infrastructure Cost Comparison 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona			
Major Cost Item	**Alternative 1**	**Alternative 2**	**Alternative 3**	**Alternative 4**
WRP at Golf Course	$17,000,000	$19,250,000	$14,000,000	$16,250,000
WRP east of highway additional construction costs		$8,500,000		$8,500,000
WRP at Castle Rock/State Park	$-	$-	$11,000,000	$11,000,000
Collection System Improvements				
Moon Ridge/Polynesian Shores	$2,905,000	$2,905,000	$2,905,000	$2,905,000
Buckskin Valley/Holiday Harbor	$3,588,000	$3,588,000	$3,588,000	$3,588,000
Castle Rock	$3,450,000	$3,450,000	$1,804,000	$1,804,000
Rio Lindo/Sundance/Red Rock	$2,502,000	$2,502,000	$1,651,000	$1,651,000
Marina Village	$2,973,000	$2,973,000	$2,074,000	$2,074,000
County Park/Branson Resort	$3,111,000	$3,111,000	$2,427,000	$2,427,000
Phases I, II, and III	$1,314,000	$1,314,000	$1,314,000	$1,314,000
Reclaimed Water Irrigation	$-	$-	$1,008,000	$1,008,000
Effluent Management	$754,000	$2,002,000	$1,204,000	$2,452,000
Retire Buckskin WWTP	$100,000	$100,000	$100,000	$100,000
Land Purchase	$250,000	$250,000	$500,000	$500,000
Alternative Total	$37,900,000	$49,900,000	$43,600,000	$55,600,000

Excluded Costs:
1. Building Sewers and connections
2. Buckskin Valley/Holiday Harbor Primary collection system
3. Advanced treatment for discharge to the Colorado River
4. Land cost and improvements for 100% effluent management

Note:
1. Conveyance costs for each village listed above also includes the conveyance cost for villages to the north.

5.3   PHASING PLAN

An example phasing plan for implementation of Alternative 1 is shown in Table 5.3.

Table 5.3   **Project Phasing Plan for Alternative 1**
2011 Wastewater Master Plan Update
Buckskin Sanitary District, Arizona

Item	Trigger	Estimated Completion Year
Construct a phase 1 of the WRP to take flows from the existing WWTP	Obtain funding	2013
Construct a lift station and force main to take wastewater from the existing WWTP site to the new lift station	Obtain funding	2013
Abandon the existing WWTP	Complete new WRP and force main	2014
Complete the Holiday Harbor collection system and begin vault and haul operations to the WRP	Complete new WRP, construct wet well	2014
Construct the primary and secondary collection system facilities for Marina Village and Branson Resort, including the influent pump station and begin delivering wastewater flows to the WRP	Complete new WRP	2015
Construct the primary and secondary collection system for Sundance and Rio Lindo and begin transporting wastewater flows from these areas	Complete infrastructure improvements for Marina Village and Branson Resort	2017
Construct the primary and secondary collection system for Castle Rock and Buckskin Mountain State Park	Complete infrastructure improvements for Sundance and Rio Lindo	2019
Construct the lift station and transmission system from the Holiday Harbor collection system and cease vault and haul operations for this village	Complete improvements for Castle Rock and Buckskin Mountain State Park	2021
Construct the primary and secondary collection system for Village #1 (Polynesian Shores) and begin serving this area	Complete improvements for Holiday Harbor	2023

SEWAGE COLLECTION SYSTEM REQUIREMENTS
(A.A.C. R18-9-E301)

Other effluent disposal methods may be proposed and will be handled on a case-by-case basis. The minimum reclaimed water quality requirements for different reuse options are provided in Table A.1.

Table A.1 Minimum Reclaimed Water Requirements for Direct Reuse[1] 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona	
Type of Direct Reuse	**Minimum Class of Reclaimed Water Required**
Irrigation of food crops	A
Recreational impoundments	A
Residential landscape irrigation	A
School ground landscape irrigation	A
Open access landscape irrigation	A
Toilet and urinal flushing	A
Fire protection systems	A
Spray irrigation of an orchard or vineyard	A
Commercial closed loop air conditioning systems	A
Vehicle and equipment washing (does not include self-service vehicle washes)	A
Snowmaking	A
Surface irrigation of an orchard or vineyard	B
Golf course irrigation	B
Restricted access landscape irrigation	B
Landscape impoundment	B
Dust control	B
Soil compaction and similar construction activities	B
Pasture for milking animals	B
Livestock watering (dairy animals)	B
Concrete and cement mixing	B
Materials washing and sieving	B
Street cleaning	B
Pasture for non-dairy animals	C
Livestock watering (non-dairy animals)	C
Irrigation of sod farms	C
Irrigation of fiber, seed, forage, and similar crops	C
Silviculture	C
Notes: (1) Reference: A.A.C. R18-11-309 Table A (2) Denitrification is designated by adding a "+" to the Class, for example A+.	

A.1   Effluent Quality

As stated previously, the treated effluent must (at a minimum) meet or exceed the current standards set forth in the Arizona Administrative Code (A.A.C.), specifically as defined in R18-9 and R18-11. The specific WRP effluent limits, sampling parameters, and sampling frequency shall be determined by the various permits required for the Regional WRP.

Typically, ADEQ requires Class B+ for effluent disposal via recharge basins, while open access methods, also promoted by the City, require Class A+. Since the land requirement for disposal via recharge basins is substantial, it is anticipated that each Regional WRP will use multiple effluent disposal methods. Therefore, each Regional WRP may include the flexibility to produce Class A+ or Class B+ depending upon the disposal method. While effluent disposal methods requiring a Class C level of treatment may be available, it should be noted that the effluent would still be required to meet the BADCT standards, which requires denitrification and more stringent disinfection requirements.

The BADCT treatment performance requirements, reclaimed water quality standards, and recommended effluent design criteria are provided in Tables A.2, A.3, and A.4, respectively.

Table A.2	BADCT Effluent Requirements 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona	
Parameter	**Average Daily Flow < 250,000 gpd Effluent Limits**[1]	**Average Daily Flow > 250,000 gpd Effluent Limits**[1]
pH	6.0 - 9.0	6.0 - 9.0
BOD (30 day average)	< 30 mg/L	< 30 mg/L
BOD (7 day average)	< 45 mg/L	< 45 mg/L
TSS (30 day average)	< 30 mg/L	< 30 mg/L
TSS (7 day average)	< 45 mg/L	< 45 mg/L
Removal Efficiency for BOD, CBOD, TSS	85%	85%
Total Nitrogen (as N) [2]	< 10 mg/L	< 10 mg/L
Fecal Coliform		
Single sample maximum	800 cfu/100 mL	23 cfu/100 mL
Seven sample median	200 cfu/100 mL	2.2 cfu/100 mL
Notes: (1) Reference: A.A.C. R18-9-B204 (2) Five month rolling geometric mean		

Table A.3	Reclaimed Water Quality Standards 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona			
Parameter	**Class A+** [1] **Effluent Limits**	**Class B+** [2] **Effluent Limits**	**Class C** [3] **Effluent Limits**
Total Nitrogen (as N) [4]	< 10 mg/L	< 10 mg/L	N/A
Turbidity			
Daily (24-hour) average	2 NTU	N/A	N/A
Single sample maximum	5 NTU	N/A	N/A
Fecal Coliform			
Single sample maximum	23 cfu/100 mL	800 cfu/100 mL	4,000 cfu/100 mL
Four out of last seven daily samples	Non Detect	200 cfu/100 mL	1,000 cfu/100 mL

Notes:
(1) Reference: A.A.C. R18-11-303
(2) Reference: A.A.C. R18-11-305
(3) Reference: A.A.C. R18-11-307
(4) Five sample geometric mean
(5) Class A, B, C, etc uses are listed in R18-11 Table A.

Table A.4	Recommended Effluent Design Criteria 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona	
Parameter		**Effluent Limits**
pH		6.0 - 9.0
BOD		< 10 mg/L
TSS		< 10 mg/L
Total Nitrogen (as N)		< 8 mg/L
Turbidity [1]		
Daily (24-hour) average		2 NTU
Single sample maximum		5 NTU
Fecal Coliform [2]		
Single sample maximum		23 cfu/100 mL
Seven sample median		2.2 cfu/100 mL

Notes:
(1) Turbidity monitoring only required if Class A+ reclaimed water is being produced.
(2) Fecal Coliform for four out of last seven daily samples must be non-detect if Class A+
    reclaimed water is being produced.

In order to meet these effluent water quality requirements, the initial phase of the Regional WRP will be required to provide (at a minimum) preliminary and secondary treatment, followed by tertiary filtration and disinfection. Specific treatment processes and design criteria for the initial phase of the Regional WRP in the City will be described in further detail in Section 3.5.

A.2   Design Peaking Factors

The most common method of determining applicable peaking factors is from the analysis of existing flow rate data. However, if existing flow measurement records are unavailable or inadequate (as is expected for the initial phases of the WRPs), the peaking factors in Table A.5 (at a minimum) shall be applied.

Table A.5	Acceptable Design Peaking Factors 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona	
Parameter	**Peak Factor**
Peak Hour	4.0
Peak Day	3.0
Average Day Peak Month	2.0

A.3   WRP Site Requirements

Site selection criteria for the WRPs, including site access and security, aesthetics, location of facilities relative to future connection (Buckskin Sanitary District Wastewater Master Plan), facility redundancy and contingency options, flood control, and storm water management, are discussed herein.

Potential sites shall be of sufficient area to accommodate the ultimate WRP footprint. If the ultimate WRP will include adjacent effluent disposal features (such as riparian preserve, recharge basins, etc.), the land requirement will be contingent upon the site-specific soil infiltration capabilities WRP proposed without adjacent effluent disposal features must identify, in the Engineering Report, the location(s) for the effluent disposal features. The land requirements shown may be reduced based upon site-specific soil information, and increases of effluent disposal by methods other than recharge basins (such as irrigation and aquifer storage and recovery wells).

Potential sites for the WRPs must be of sufficient size to meet the setback requirements set forth in A.A.C. R18-9-B201, as summarized in Table A.6. Setback is defined as the distance from the WRP to the nearest contiguous property line. Selected sites <u>must</u> take into account the future expansion of the Regional WRP and its ultimate size. Additional space is to be set aside on the site for treating of flows from future developments within the wastewater service area, for future sludge processing on-site, or other operation and maintenance-related activities. Therefore the ultimate site footprint shall provide the setback requirements for facilities over 1 mgd, as defined in Table A.6. The setbacks may be required between the WRP and any contiguous effluent disposal features since these features will likely be frequented by the community. This will be addressed on a case-by-case basis.

Table A.6 Facility Setback Requirements 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona		
Sewage Treatment Facility Design Flow (gpd)	No Noise, Odor, or Aesthetic Controls (feet)	Full Noise, Odor, and Aesthetic Controls (feet)
24,000 to less than 100,000	350	50
100,000 to less than 500,000	500	100
500,000 to less than 1,000,000	750	250
1,000,000 or greater	1,000	350
<u>Source</u>: Arizona Administrative Code (A.A.C.) R18-9-B201		

All wastewater facilities within BSD are intended to be "good neighbors," meaning that all WRPs shall be provided with odor and noise control, as defined further in Sections 3.6 and 3.7, respectively. All property owners whose property is adjacent to the proposed WRP site shall be fully informed about the proposed WRP. The WRP site plan shall be submitted to the Buckskin Sanitary District for review and approval as part of the Engineering Report submittal.

A.3.1 Site Access and Security

The requirement for access for the WRP site includes adequate access for operations and maintenance vehicles to the site from public roads and within the site to the process tanks and buildings.

Access roads to a site from public roads and access roads within the site shall be a minimum 24-foot wide road, designed in accordance with the latest edition of the Maricopa Association of Governments Standards for Public Works Construction. Where possible, access shall be from a main road and not a side street in a subdivision. Two entrances from separate roads to the site shall be provided, where practical.

The WRP roadway system shall be laid out to provide access for large trucks to all treatment facilities for WRP operation and maintenance. The minimum turning radius for WRP roads shall be 40 feet. Where equipment maintenance requires the use of truck-mounted or portable hoists, space must be sufficiently allocated for this equipment.

Secured access to the WRP site shall be provided, in the form of either full perimeter fencing or block walls, with lockable gates. Signs shall be posted identifying the site as a water reclamation facility and forbidding trespassing. The potential for vandalism at a given site shall be discussed with the City, and appropriate security measures designed and constructed. All exterior doors shall be of metal construction, and all glass shall be tempered. Exterior locks shall be of the mortise type. A single key shall operate all locks.

The WRP access shall be reviewed with the Buckskin Sanitary District Fire Department to verify that the requirements for fire protection have been met.

Access within the WRP shall be provided for general operation and maintenance of all equipment as follows:

- A minimum of 3 feet clearance space around all process equipment.
- A minimum of 3 feet clearance in front of electrical panels.
- A minimum of 3 feet clearance in back of control panels.
- Walkways 4 feet wide shall be provided around all process tanks.

A.4   RECLAIMED WATER SYSTEM

This section presents a description of the criteria to be used for future reclaimed water distribution systems. The ultimate goal of the reclaimed water distribution system is to provide reliable delivery at adequate system pressures. Reliability in a distribution system is typically accomplished by providing system redundancy in the form of looping, extra pumps, and additional storage. In general, the level of system reliability is a function of the reliability of the individual system components.

A.4.1   Supply, Storage and Pumping

The future reclaimed water system supply will be related to the influent flow to the future Regional WRPs. Reclaimed water from the WRPs, and potentially groundwater pumped from recovery wells, will comprise the source water for distribution to the reclaimed water system.

System storage reservoirs may be provided at the Regional WRPs to serve mainly as "day tanks" to assist in maximizing reclaimed water utilization by capturing excess flow during diurnal peaking. The reservoirs are not intended to function as storage to meet abnormally high or extended demand periods. Additional storage located in the system, such as

individual large user storage facilities and possibly water from recovery wells, may be utilized to meet peak demands.

Pump stations associated with the reclaimed water system should have the capability to consistently meet maximum day system demands with the largest pump out of service (firm capacity).

A.4.2 Transmission and Distribution

The reclaimed water distribution system piping will serve to deliver water from the Regional WRPs and storage reservoirs to users throughout the system. A series of performance criteria have been developed for the proposed reclaimed water distribution systems, which has been based upon typical pressurized system design standards. These standards are provided as follows:

- System pressures must be maintained between 20 and 85 pounds per square inch (psi) throughout the reclaimed water distribution system. The maximum allowable pressure in the reclaimed water distribution is set at 85 psi, in order to protect any future PVC pipelines that are installed. At system pressures less than 20 psi, users may not have adequate pressure to operate sprinkler irrigation systems (for example), which will lead to complaints. Consequently, the minimum allowable pressure at any point in the reclaimed water distribution system is to be set at 20 psi. It is the end user's responsibility to provide additional boosting capability to meet pressure requirements exceeding 20 psi.

- Velocity criteria under maximum day demand conditions:

  Velocity ≤ 5 feet per second (fps) for pipes < 36 inches in diameter (H_L = 2 – 7 ft / 1,000 ft)

- The City will allow the installation of PVC for future reclaimed water mains. The Hazen-Williams design coefficient of roughness for PVC will be 140. (The design coefficient of roughness for ductile iron will be 120.)

A.4.3 Criteria Summary

Table A.7 summarizes the criteria that will be used for the reclaimed water system.

Table A.7	Reclaimed Water System Criteria Summary 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona	
Description	**Criteria**
Materials	PVC or Ductile Iron
Transmission/Distribution – Velocity/ Headloss Pipe < 36 " in diameter	$\leq$ 5 fps (HL = 2 to 7 ft / 1,000 ft)
System Pressure Criteria	$\geq$ 20 psi
	$\leq$ 85 psi
Supply Delivery	In general, customers with on-site storage will be supplied during the day. Direct use customers will be supplied in the evening.
	It is recommended that flow control valves be used to control contractual deliveries to individual customers.

A.5 WASTEWATER COLLECTION SYSTEM

This section describes the capacity requirements of future wastewater collection system improvements. The capacities of gravity sewers, force mains, and lift stations shall be based on the performance and design criteria presented herein.

A.5.1 Pipe Capacities

Sewer capacities are dependent on many factors. These include roughness of pipe, maximum allowable depth of flow, and limiting velocity and slope. The Continuity Equation and Manning's Equation are typically used for steady-flow hydraulic calculations. The Manning's coefficient 'n' is a friction coefficient that varies with respect to pipe material, size of pipe, slightly with depth of flow, smoothness of joints, root intrusion, and other factors. For gravity sewers, the Manning's coefficient shall be set at an 'n' value of 0.013 as this is a typical observed gravity sewer field value. No deviations from this value will be accepted by the City.

A.5.2 Flow Depth (d/D)

When designing sewers, it is common practice to adopt variable flow depth criteria for various pipe sizes. This criterion is expressed as a ratio of maximum depth of flow to pipe diameter (d/D). Design d/D ratios typically range from 0.5 to 1.0, with the lower values typically used for smaller pipes that may experience flow peaks greater than planned or may experience blockages from debris.

The flow depth criterion for new sewers is 0.5 for diameters less than 12 inches, and 0.75 for diameters 12 inches and greater. However, existing sewers will be evaluated based on a flow depth criteria of 0.9 at peak flows because there are fewer unknowns, especially in established, built-out areas, and because there is no need to replace or provide relief for an

existing sewer until flows are at the design capacity of the pipe. The hydraulic criteria used for sizing the proposed gravity sewers will have a greater factor of safety than the criteria used to evaluate the capacity of the existing system due to the uncertainties in making projections of future flows. The proposed difference between the design criteria and the existing system criteria allows full use of the existing sewer capacities and prevents unnecessary pipe replacement. This approach avoids the problem of replacing or upgrading existing sewers prematurely.

In order to minimize the settlement of solids in the flow and promote scour, it is standard design practice to specify that a minimum velocity of 2 feet per second (fps) be maintained when the pipe is flowing half full. At this velocity, the sewer flow will typically provide self-cleaning for the pipe. Due to the hydraulics of a circular pipe, the velocity for half pipe flow approaches the velocity of nearly full pipe flow. Table A.8 lists the minimum slopes for maintaining self-cleaning velocities with d/D = 0.5. The minimum slope listed in the table is 0.0008 ft/ft, which is the minimum practical slope for gravity sewer construction. Greater slopes are desirable if they are compatible with existing topography, as long as the velocity does not exceed 8 fps.

Table A.8	Recommended Minimum Slopes for Circular Pipes 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona		
Pipe Size (inches)	Minimum Slope[1] [2] (ft/ft)	Pipe Capacity[3]	
		(mgd)	(cfs)
8	0.0034	0.45	0.70
10	0.0025	0.70	1.09
12	0.0020	1.02	1.57
14	0.0016	1.38	2.14
15	0.0015	1.59	2.45
16	0.0014	1.80	2.79
18	0.0012	2.28	3.53
20	0.0010	2.82	4.36
21	0.0010	3.11	4.81
24	0.0008	4.06	6.28

Notes:
(1) Slopes are calculated using Manning's Equation for full pipe flow with a minimum velocity of 2 fps.
(2) Sewers larger than 24 inches should have a slope $\geq$ 0.0008.
(3) Pipe Capacity based on full pipe flow.

A.5.3 Changes in Pipe Size

When a smaller sewer joins a larger sewer, the invert of the larger sewer will be lowered sufficiently to maintain the same energy gradient. An approximate method for securing these results is to place the d/D 0.8-depth point of both sewers at the same elevation. Since

Geographic Information System data is available for the City's wastewater system, this information will be used for the sewer inverts. For master planning purposes, proposed sewer crowns will be matched at manholes when a smaller sewer joins a larger one.

A.5.4   Lift Stations

All lift stations (permanent, temporary or "package" type) will require City approval prior to construction. Lift stations to be constructed within the City must conform to the standards set forth herein, as follows:

- Arizona Administration Code R18-9-E301 D.

- Engineering Bulletin No. 11, Chapter V - Minimum Requirements for Design, Submission of Plans and Specifications of Sewage Works, ADEQ, July 1978.

- Maricopa Association of Governments (MAG) Uniform Standard Specifications and Details, latest version.

A.5.5   Force Mains

Lift stations shall conform to the following requirements and standards:

- Arizona Administration Code R18-9-E301 D.

- Chapter V of the Engineering Bulletin No. 11 - Minimum Requirements for Design, Submission of Plans and Specifications of Sewage Works, ADEQ, July 1978.

- Maricopa Association of Governments (MAG) Uniform Standard Specifications and Details, latest version.

In addition, force mains should have a minimum diameter of 6 inches. The velocity should be between 3 and 7 fps to provide scour velocity so that the solids deposited while the pumps are off will be transported when the pumps are operating.

A.5.6   Gravity Sewer Planning Guidelines

Gravity sewers should be designed and constructed to have a minimum 5 feet of cover or sufficient depth to serve the ultimate drainage area.

Gravity sewers should be designed and constructed with a minimum 4 feet of separation between the flowline of irrigation ditches and the crown of the sewer.

Gravity sewers and force mains should have a minimum separation of 6 feet from potable water mains unless they are encased in concrete as per Arizona Department of Environmental Quality requirements.

Manholes with sewers intersecting at greater than or equal to 90-degree angles should provide 0.2 feet of invert drop across the manhole. Other manholes should provide a minimum 0.1 feet of invert drop.

A.5.7 Criteria Summary

Table A.9 summarizes the performance and design criteria used to evaluate existing wastewater collection features and for planning new wastewater collection system features.

Table A.9	Wastewater System Criteria Summary 2011 Wastewater Master Plan Update Buckskin Sanitary District, Arizona		
Description		**Criteria**	
Pipe Size (inches)	**Minimum Slope**[(1)(2)] **(ft/ft)**	**Pipe Capacity**[(3)] **(mgd)**	**(cfs)**
8	0.0034	0.45	0.70
10	0.0025	0.70	1.09
12	0.0020	1.02	1.57
14	0.0016	1.38	2.14
15	0.0015	1.59	2.45
16	0.0014	1.80	2.79
18	0.0012	2.28	3.53
20	0.0010	2.82	4.36
21	0.0010	3.11	4.81
24	0.0008	4.06	6.28
Maximum Velocity		$\leq$ 7 feet per second	
Flow Depth, d/D			
d/D for New Sewer Pipes with Diameters less than 12 inches		= 0.5	
d/D for Designing New Sewer Pipes 12 inches and Higher		= 0.75	
d/D for Evaluating Existing Mains in Developed Areas		= 0.90	
Headloss in Existing Pipes			
Gravity Pipes		Manning's n = 0.013	
Pressure Pipes		Hazen William's C = 120	
Changes in Pipe Size			
When a smaller sewer joins a larger one:		Sewer crowns will be matched.	
Headloss at Manholes			
Manholes with pipelines intersecting at 90 degrees or greater		Provide 0.2' Invert Drop	
Manholes with pipelines intersecting at less than 90 degrees		Provide 0.1' Invert Drop	
Notes: (1) Slopes are calculated using Manning's Equation for pipes flowing full with a minimum velocity of 2 fps. (2) Sewers larger than 24 inches should have a slope $\geq$ 0.0008. (3) Pipe Capacity based on full pipe flow.			

PART E. TYPE 4 GENERAL PERMITS

R18-9-E301. 4.01 General Permit: Sewage Collection Systems

A.   A 4.01 General Permit allows a new sewage collection system or an expansion of an existing sewage collection system involving new construction.
   1.   A sewer collection system includes all sewer lines and associated structures, devices, and appurtenances that:
       a   Are owned or controlled by a public or private sewer utility extending from the treatment works to the upstream points in the system where private owners assume ownership or control; or
       b   Serve multiple private users from the upstream points where the individual users assume ownership or control to the downstream point where the sewer delivers wastewater to a sewage collection system owned or controlled by a public or private sewer utility, or to a sewage treatment facility.
   2.   A sewer collection system repair is not an expansion of the system that requires a Notice of Intent to Discharge. Repairs include work performed in response to deterioration of existing structures, devices, and appurtenances with the intent to maintain or restore the system to its original operational characteristics.

B.   Performance. An applicant shall design, construct, and operate a sewage collection system so that it:
   1.   Provides adequate wastewater flow capacity for the planned service;
   2.   Minimizes sedimentation, blockage, and erosion through maintenance of proper flow velocities throughout the system;
   3.   Prevents sanitary sewer overflows through appropriate sizing, capacities, and inflow and infiltration prevention measures throughout the system;
   4.   Protects water quality through minimization of exfiltration losses from the system;
   5.   Provides for adequate inspection, maintenance, testing, visibility, and accessibility; and
   6.   Maintains system structural integrity.

C.   Notice of Intent to Discharge. In addition to the Notice of Intent to Discharge requirements specified in R18-9-A301(B), an applicant shall submit the following information:
   1.   A statement, signed by the owner or operator of the sewage treatment facility that treats or processes the sewage from the proposed sewer collection system.
       a   The owner or operator shall affirm that the additional volume of wastewater delivered to the facility by the proposed sewer collection system will not cause any flow or effluent quality limits of the individual permit for the facility to be exceeded.
       b   If the facility is classified as a groundwater protection permit facility under A.R.S. § 49-241.01(C), or if no flow or effluent limits are applicable, the owner or operator shall affirm that the design flow of the facility will not be exceeded.
   2.   If the proposed sewage collection system delivers wastewater to a downstream sewer collection system under different ownership or control, a statement, signed by the owner or operator of the downstream sewer collection system, affirming that the downstream system can maintain the performance required by subsection (B) if it receives the increased flows associated with the new system or the expansion;
   3.   A general site plan showing the boundaries and key aspects of the project;
   4.   Construction quality drawings that provide overall details of the site and the engineered works comprising the project including:
       a   Relevant plans and profiles of sewer lines, force mains, manholes, and lift stations with sufficient detail to allow Department verification of design and performance characteristics;
       b   Relevant cross sections showing construction details and elevations of key components of the sewer collection system to allow Department verification of design and performance characteristics, including the slope of each gravity sewer segment stated as a percentage; and
       c   Drainage features and controls, and erosion protection as applicable, for the components of the project.
   5.   Documentation of design flows for significant components of the sewage collection system and the basis for calculating the design flows;
   6.   An operation and maintenance plan if the project has a design flow of more than 10,000 gallons per day;
   7.   Drawings, reports, and other information that are clear, reproducible, and in a size and format specified by the Department. The applicant may submit the drawings in a Department-approved electronic format; and
   8.   Design documents, including plans, specifications, drawings, reports, and calculations that are signed and sealed by an Arizona-registered professional engineer unless prohibited by law. The designer shall use good engineering judgement following engineering standards of practice, and rely on appropriate engineering methods, calculations, and guidance.

D. Design Requirements.
 1. General Provisions. An applicant shall ensure that the design, installation, and testing of a new sewage collection system or an expansion to an existing sewage collection system involving new construction complies with the following rules. An applicant shall:
  a Base design flows for components of the system on unit flows specified in Table 1, Unit Daily Design Flows. If documented by the applicant, the Department may accept lower unit flow values in the served area due to significant use of low flow fixtures.
  b Use the "Uniform Standard Specifications for Public Works Construction," referenced in this Section and published by the Maricopa Association of Governments, revisions through 2000, or the "Pima County Wastewater Management," November 1994 Edition, as the applicable design and construction criteria, unless the Department approved alternative design standards or specifications authorized by a delegation agreement under A.R.S. § 49-107.
  c Use gravity sewer lines, if appropriate. The applicant shall design gravity sewer lines and all other sewer collection system components, including force mains, manholes, lift stations, and appurtenant devices and structures to accommodate maximum sewage flows as determined by the method specified in subsections (D)(1)(c)(i) or (D)(1)(c)(ii) that yields the greatest calculated flow:
   i. Any point in a sewer main when flowing full can accommodate an average flow of 100 gallons per capita per day for all populations upstream from that point, or
   ii. Any point in a sewer collection system can accommodate a peak flow for all populations upstream from that point as tabulated below:

Upstream Population	Peaking Factor
100	3.62
200	3.14
300	2.90
400	2.74
500	2.64
600	2.56
700	2.50
800	2.46
900	2.42
1000	2.38
1001 to 10,000	$PF = (6.330 \times p^{-0.231}) + 1.094$
10,001 to 100,000	$PF = (6.177 \times p^{-0.233}) + 1.128$
More than 100,000	$PF = (4.500 \times p^{-0.174}) + 0.945$
PF = Peaking Factor p = Upstream Population	

  d Ensure the separation of sewage collection system components from drinking water distribution system components under R18-4-502.
  e Request review and approval of an alternative to a design feature specified in this Section by following the requirements of R18-9-A312(G).
 2. Gravity sewer lines. An applicant shall:
  a Ensure that any sewer line that runs between manholes, if not straight, is of constant horizontal curvature with a radius of curvature not less than 200 feet;
  b Cover each sewer line with at least three feet of backfill meeting the requirements of subsection (D)(2)(h)(i). The applicant shall:
   i. Include at least one note specifying this requirement in construction plans;
   ii. If site-specific limitations prevent three feet of earth cover, provide the maximum cover attainable, and construct the sewer line of ductile iron pipe or other materials of equivalent or greater tensile and compressive strength;
   iii. If ductile iron pipe is not used, design and construct the sewer line pipe with restrained joints or an equivalent feature; and
   iv. Ensure that the design of the pipe and joints can withstand crushing or shearing from any expected load. Construction plans shall note locations requiring these measures.

c   If sewer lines cross floodways, place the lines at least two feet below the 100-year storm scour depth and construct the lines using ductile iron pipe or pipe with equivalent tensile strength, compressive strength, shear resistance, and scour protection. The applicant shall ensure that sewer lines constructed in this manner extend at least 10 feet beyond the boundary of the 100-year storm scouring. Construction plans shall note locations requiring these measures.

d   Ensure that each sewer line is eight inches in diameter or larger except:

    i.   The first 400 feet of a dead end sewer line with no potential for extension may be six inches in diameter if the design flow criteria specified in subsection (D)(1)(c) are met. If the line is ever extended, the applicant seeking the extension shall replace the entire length with larger pipe to accommodate the new design flow; or

    ii.   The sewer lines for a sewage collection system for a manufactured home, mobile home, or recreational vehicle park are not less than four-inches in diameter for up to 20 units, five-inches in diameter for 21 to 36 units, and six-inches in diameter for 37 to 60 units.

e   Design sewer lines with at least the minimum slope calculated from Manning's Formula using a coefficient of roughness of 0.013 and a sewage velocity of two feet per second when flowing full.

    i.   An applicant may request a smaller minimum slope under R18-9-A312(G) if the smaller slope is justified by a quarterly program of inspections, flushings, and cleanings.

    ii.   If a smaller minimum slope is requested, the slope shall not be less than 50% of that calculated from Manning's formula using a coefficient of roughness of 0.013 and a sewage velocity of two feet per second.

f   Design sewer lines to avoid a slope that creates a sewage velocity greater than 10 feet per second. The applicant shall construct any sewer line carrying a flow with a normal velocity of greater than 10 feet per second using ductile iron pipe or pipe with equivalent erosion resistance, and structurally reinforce the receiving manhole or sewer main.

g   Design and install sewer lines, connections, and fittings with materials that meet or exceed manufacturer's specifications not inconsistent with this Chapter to:

    i.   Limit inflows, infiltration, and exfiltration;

    ii.   Resist corrosion in the project electrochemical environment;

    iii.   Withstand anticipated live and dead loads; and

    iv.   Provide internal erosion protection.

h   Indicate trenching and bedding details applicable for each pipe material and size in the design plans. Sewer lines shall be placed in trenches and bedded following the specifications established in subsections (D)(2)(h)(i) and (D)(2)(h)(ii). This material is incorporated by reference and does not include any later amendments or editions of the incorporated matter. Copies of the incorporated material are available for inspection at the Department of Environmental Quality and the Office of the Secretary of State, or may be obtained from the Maricopa Association of Governments, 302 N. 1st Avenue, Suite 300, Phoenix, Arizona 85003, or from Pima County Wastewater Management, 201 N. Stone Avenue, Tucson, Arizona 85701-1207.

    i.   "Trench Excavation, Backfilling, and Compaction" (Section 601), published in the "Uniform Standard Specifications for Public Works Construction," published by the Maricopa Association of Governments, revisions through 2000; and

    ii.   "Rigid Pipe Bedding for Sanitary Sewers" (WWM 104), and "Flexible Pipe Bedding for Sanitary Sewers" (WWM 105), published by Pima County Wastewater Management, revised November 1994.

i   Perform a deflection test of the total length of all sewer lines made of flexible materials to ensure that the installation meets or exceeds the manufacturer's recommendations and record the results.

j   Test each segment of the sewer line for leakage using the applicable method below and record the results:

    i.   "Standard Test Method for Installation of Acceptance of Plastic Gravity Sewer Lines Using Low-Pressure Air" published by the American Society for Testing and Materials, (F 1417-92), reapproved 1998;

    ii.   "Standard Practice for Testing Concrete Pipe Sewer Lines by Low-Pressure Air Test Method" published by the American Society for Testing and Materials, (C 924-89), reapproved 1997;

    iii.   "Standard Test Method for Low-Pressure Air Test of Vitrified Clay Pipe Lines" published by the American Society for Testing and Materials, (C 828-98), approved March 10, 1998; or

    iv.   The material listed in subsections (D)(2)(j)(i), (D)(2)(j)(ii), and (D)(2)(j)(iii) is incorporated by reference and does not include any later amendments or editions of the incorporated matter. Copies of the incorporated material are available for inspection at the Department of Environmental Quality and the Office of the Secretary of State, or may be obtained from the American Society for Testing and Materials, 100 Barr Harbor Drive, Conshohocken, PA 19428-2959.

k   Test the total length of the sewer line for uniform slope by lamp lighting, remote camera or similar method approved by the Department, and record the results.

3. Manholes.
   a An applicant shall install manholes at all grade changes, all size changes, all alignment changes, all sewer intersections, and at any location necessary to comply with the following spacing requirements:

   Sewer Pipe Diameter (inches) Maximum Manhole Spacing (feet)

Sewer Pipe Diameter (inches)	Maximum Manhole Spacing (feet)
4 to less than 8	300
8 to less than 18	500
18 to less than 36	600
36 to less than 60	800
60 or greater	1300

   b The Department shall allow greater manhole spacing following the procedure provided in R18-9-A312(G) if documentation is provided showing the operator possesses or has available specialized sewer cleaning equipment suitable for the increased spacing.
   c The applicant shall ensure that manhole design is consistent with "Pre-cast Concrete Sewer Manhole" (#420), "Offset Manhole for 8" - 30" Pipe" (#421), and "Brick Sewer Manhole and Cover Frame Adjustment" (#422), 1998, including revisions through 2000, published by the Maricopa Association of Governments; and "Manholes and Appurtenant Items" (WWM 201 through WWM 211), Standard Details for Public Improvements, 1994 Edition, published by Pima County Wastewater Management.
   d The material specified in subsection (D)(3)(c) is incorporated by reference and does not include any later amendments or editions of the incorporated matter. Copies of the incorporated material are available for inspection at the Department of Environmental Quality and the Office of the Secretary of State, or may be obtained from the Maricopa Association of Governments, 302 N. 1st Avenue, Suite 300, Phoenix, Arizona 85003, or from Pima County Wastewater Management, 201 N. Stone Avenue, Tucson, Arizona 85701-1207.
   e The applicant shall not locate manholes in areas subject to more than incidental runoff from rain falling in the immediate vicinity unless the manhole cover assembly is designed to restrict or eliminate storm water inflow.
   f The applicant shall test manholes using one of the following test protocols:
      i. Watertightness testing by filling the manhole with water. The applicant shall ensure that the drop in water level does not exceed 0.001 of total manhole volume in one hour.
      ii. Air pressure testing using the "Standard Test Method for Concrete Sewer Manholes by Negative Air Pressure (Vacuum) Test," published by the American Society for Testing and Materials, (C 1244-93), approved August 15, 1993. This material is incorporated by reference, does not include any later amendments or editions of the incorporated matter, and is on file with the Office of the Secretary of State. The material may be viewed at the Department of Environmental Quality, Water Quality Division, or obtained from the American Society for Testing and Materials, 100 Barr Harbor Drive, Conshohocken, PA 19428-2959.
   g The applicant shall perform manhole testing under subsection (D)(3)(f) after installation of the manhole cone to verify watertightness of the manhole from the top of the cone down.
      i. Upon satisfactory test results, the applicant shall install the manhole ring and any spacers, complete the joints, and seal the manhole to a watertight condition.
      ii. If the manhole cone, spacers, and ring can be installed to final grade without disturbance or adjustment by later construction, the applicant may perform the testing from the top of the manhole ring on down.
   h The applicant shall locate a manhole to provide adequate visibility and vehicular maintenance accessibility after the manhole has been built.
4. Force mains. If it is impractical to install a gravity sewer line system, an applicant may install a force main if it meets the following design, installation, and testing requirements. The applicant shall:
   a Design force mains to maintain a minimum flow velocity of three feet per second and a maximum flow velocity of seven feet per second.
   b Ensure that force mains have the appropriate valves and controls required to prevent drainback to the lift station. If drainback is necessary during cold weather to prevent freezing, the control system may allow manual or automatic drainback.
   c Incorporate air release valves or other appropriate components in force mains at all high points along the line to eliminate air accumulation. If engineering calculations provided by the applicant demonstrate that air will not accumulate in a given high point under typical flow conditions, the Department shall waive the requirement for an air release valve.
   d Provide thrust blocks or restrained joints if needed to prevent excessive movement of the force main. Construction plans shall show thrust block or restrained joint locations and details. The documentation submitted to the Department for verification of the general permit shall include calculations and analysis

of water hammer potential and surge control measures and shall be signed and sealed by an Arizona-registered professional engineer.

    e   If a force main is proposed to discharge directly to a sewage treatment facility without entering a flow equalization basin, include in the Notice of Intent to Discharge a statement from the owner or operator of the sewage treatment facility that the design is acceptable.

    f   Design a force main to withstand, and upon completion test the force main for leakage, at a pressure of 50 pounds per square inch or more above the design working pressure.

    g   Supply flow to a force main using a lift station that meets the requirements of subsection (D)(5).

5.   Lift stations. An applicant shall:

    a   Secure a lift station to prevent tampering and affix on its exterior, or on the nearest vertical object if the lift station is entirely below grade, at least one warning sign that includes the 24-hour emergency phone number of the owner or operator of the collection system;

    b   Protect lift stations from physical damage from a 100-year flood event. Construction of a lift station is prohibited in a floodway;

    c   Lift station wet well design. The applicant shall:

        i.   Ensure that the minimum wet well volume in gallons shall be 1/4 of the product of the minimum pump cycle time, in minutes, and the total pump capacity, in gallons per minute;

        ii.   Protect the wet well against corrosion to provide at least a 20-year design life;

        iii.   Ensure that wet well volume does not allow the sewage retention time to exceed 30 minutes unless the sewage is aerated, chemicals are added to prevent or eliminate hydrogen sulfide formation, or adequate ventilation is provided. Notwithstanding these measures, the applicant shall not allow the septic condition of the sewage to adversely affect downstream collection systems or sewage treatment facility performance;

        iv.   Ensure that excessively high or low levels of sewage in the wet well trigger an audible or visual alarm at the wet well site and at the system control center; and

        v.   Ensure that a wet well designed to accommodate more than 5000 gallons per day has a horizontal open cross-sectional area of at least 20 square feet.

    d   Equip a lift station wet well with at least two pumps. The applicant shall ensure that:

        i.   The pumps are capable of passing a 2.5-inch sphere or are grinder pumps;

        ii.   The lift station is capable of operating at design flow with any one pump out of service; and

        iii.   Piping, valves, and controls are arranged to allow independent operation of each pump.

    e   Not use suction pumps if the sewage lift is more than 15 feet. The applicant shall ensure that other types of pumps are self-priming and that pump water brake horsepower is at least 0.00025 times the product of the required discharge, in gallons per minute, and the required total dynamic head, in feet;

    f   For safety during operation and maintenance, design lift stations to conform with all applicable state and federal confined space requirements; and

    g   For lift stations receiving an average flow of more than 10,000 gallons per day, include a standby power source in the lift station design that may be put into service immediately and remain available for 24 hours per day.

E.   Additional Verification of General Permit Conformance requirements. An applicant shall:

1.   Supply a signed and sealed Engineer's Certificate of Completion, unless prohibited by law, in a format approved by the Department that provides the following:

    a   Confirmation that the project was completed in compliance with the requirements of this Chapter, as described in the plans and specifications corresponding to the Provisional Verification of General Permit Conformance issued by the Director, or with changes that are reflected in as-built plans submitted with the Engineer's Certificate of Completion;

    b   As-built plans, if required, that are properly identified and numbered; and

    c   Confirmation of satisfactory test results from deflection, leakage, and uniform slope testing.

2.   Provide any other relevant information required by the Department to determine that the facility conforms to the terms of this general permit; and

3.   If the project has a design flow of more than 10,000 gallons per day, provide a final operation and maintenance plan that includes the 24-hour emergency number of the owner or operator of the system.

F.   Operation and maintenance requirements.

1.   The permittee of a sewage collection system that includes a force main and lift station or that has a design flow of more than 10,000 gallons per day shall maintain, and revise as needed, an operation and maintenance plan for the system at the system control center.

2.   The permittee shall ensure that the operation and maintenance plan is the basis for operation and continuing maintenance of the sewer collection system.

Historical Note

New Section adopted by final rulemaking at 7 A.A.R. 235, effective January 1, 2001 (Supp. 00-4).

BUCKSKIN WWTP PROCESS MODEL RESULTS

B.1   PROCESS EVALUATION

Biotran is a computer model developed by Carollo Engineers specifically for wastewater treatment plant process evaluations. This program utilizes mass balances and biological and physical models, to simulate the interactions between the processes in a wastewater treatment facility. The model is used in conjunction with the established wastewater flows and characteristics and design criteria to determine treatment capacities for each unit process. The model also generates projections for biosolids production, oxygen usage, etc., that can be used to size auxiliary facilities (i.e., blowers, pumps, etc.).

A brief description of each process is provided based on existing facility drawings and site visits. Likewise, capacity estimates are based upon available facility drawings and do not take into account physical condition of structures and equipment.

Ultimately, the capacity estimates presented as part of this evaluation do not account for mechanical and electrical limitations of the associated units attributed to age or maintenance issues not detected as part of this initial evaluation.

B.1.1   Surge Tank and Influent Screens

Raw influent passes through a comminutor with a screen bypass before discharging into the surge tank. The existing surge tank has a total capacity of 32,650 gallons, and has two submersible pumps that equalize influent flow and send the screened wastewater flow to the aeration basins. No information was available regarding the capacity or condition of the pumps.

B.1.2   Activated Sludge Basins

B.1.2.1   Overview

The existing activated sludge process is an extended aeration system consisting of two 93,250-gallon aeration basins, a fine bubble air diffuser system, and three 15-HP blowers. The aeration basins are designed as two completely-mixed tanks in series, and do not have anoxic zones for denitrification.

As outlined above, a process model of the Buckskin WWTP was created to simulate process operation based on various inputs for flow, loading, and other operating scenarios. The model was used to evaluate the treatment capacity of the existing secondary treatment system at the facility. Outputs from the model include process effluent characteristics, process safety factors, and the maximum allowable loadings for the associated unit operations.

Ultimately, the results of the process model assisted in determining the required Buckskin WWTP process modifications for treatment of wastewater flows at higher wastewater strength, and at higher flows compared to original design wastewater characteristics and permitted flows. The results of the model are outlined in the subsequent sections. The actual model results are included in Appendix B.

B.1.2.2 <u>Capacity Estimate</u>

Permitted Flow. The existing aeration basins are capable of meeting the permitted flow of 228,000 gpd and the associated loadings outlined in the previous sections, both for the original design scenario and for the high-strength scenario. The mixed liquor suspended solids (MLSS) used in the evaluations was 2,000 mg/L, and the resulting solids retention times (SRT) for the original design scenario and the high-strength scenario were 9.9 days and 5.5 days, respectively. Maintaining a minimum SRT of 5 days is recommended for this system. Therefore, the high-strength scenario loadings represent an upper loading limit for the aeration basins of the Buckskin WWTP. The MLSS of 2,000 mg/L was determined by estimating the allowable solids loading to the secondary clarifier (see section discussing secondary clarifier below).

Increased Flow. The aeration basins could accommodate an increased capacity of 250,000 gpd at similar SRT values as mentioned above for the 228,000 gpd flow and the two loading scenarios considered in this analysis. However, the operating MLSS would need to be increased to 2,200 mg/L at this increased flow condition. The practical feasibility of loading the secondary clarifiers at an MLSS of 2,200 mg/L and plant influent flows of 250,000 gpd ultimately depends on the settling properties of the sludge, but appears to be feasible (see section discussing secondary clarifier below).

Nitrogen Removal. If BADCT requirements dictate nitrogen removal in the future, the aeration basins could be modified to perform nitrification-denitrification. The existing basins could be retrofitted in a Modified Ludzak-Ettinger (MLE) process configuration. The MLE process is widely used in Arizona facilities and could be utilized at the Buckskin WWTP to reduce effluent total nitrogen concentrations to below 10 mg/L. Modifications to the existing plant would be required to provide the required anoxic zones and maintain the rated treatment capacity. The possible modification includes converting the surge tanks into dedicated anoxic zones for denitrification, by performing the following:

- Installation of baffles and mixers in the surge tank to create two anoxic zones. The first anoxic zone would receive the raw wastewater influent and an internal mixed liquor return (IMLR).

- Installation of IMLR pumps in the second aeration basin, and routing IMLR flow to the first anoxic zone created in the surge tank.

B.1.3 Aeration System

B.1.3.1 Overview

The existing aeration system consists of three rotary lobe blowers and a fine bubble diffuser system, including one standby and two duty blowers. The blowers have a capacity of 290 cubic feet per minute (cfm) each, and 15-HP motors. The drawings show a separate blower for aeration to the surge tank, but no information was found in the treatment plant specifications. Process air is distributed to the two aeration basins, the aerobic sludge digester, and the surge tank.

The aeration basins have diffuser grids providing full coverage with evenly spaced membrane disk circular fine bubble diffusers. The surge tank and the aerobic sludge digester have membrane tube diffuser arrangements along one or two sides of the rectangular basins, respectively, and provide mixing air to such basins. The aeration system upgrade from coarse bubble aeration (original plant) to fine bubble aeration was part of the modifications that led to the re-rating of the plant to 228,000 gpd.

The process model includes an estimation of the air requirements at the different scenarios evaluated, and uses established oxygen transfer efficiencies for the specific type of diffusers at the Buckskin WWTP.

B.1.3.2 Capacity Estimate

Permitted Flow. The air requirements associated with the process blowers include air for the aeration basins and for the aerobic sludge digester (assuming there is a separate blower dedicated to the surge tank). The analysis of blower capacity was based on operating only the two duty blowers and not the standby unit. At the original design loading scenario and rated flow of 228,000 gpd, the existing 290 cfm process blowers have sufficient capacity for average day loadings, but not for peak day loading conditions. For peak loading conditions, the required blower capacity is approximately 2.3 times the existing firm blower capacity. At the high-strength loading scenario, the required blower capacity is approximately 1.7 and 4.5 times the existing firm capacity for average and peak loading conditions, respectively.

The diffuser air loading in the aeration basins at the original design loading scenario is within an acceptable design range for membrane disk circular fine bubble diffusers. However, under the high-strength loading scenario, the resulting diffuser air loadings are approximately 2 times the acceptable operating values of 1.5 scfm/diffuser under average loading and 4.5 scfm/diffuser under peak loading.

Increased Flow. As mentioned above, the analysis of blower capacity was based on operating only the two duty blowers and not the standby unit. At the original design loading scenario and at an increased flow of 250,000 gpd, the existing process blowers have sufficient capacity for average day loadings, but not for peak day loading conditions. For peak loading conditions, the required blower capacity is approximately 2.6 times the existing firm blower capacity. At the high-strength loading scenario, the required blower

capacity is approximately 1.9 and 5 times the existing firm capacity for average and peak loading conditions, respectively.

The diffuser air loading in the aeration basins at the original design loading scenario is slightly (10 percent) above acceptable design range for membrane disk circular fine bubble diffusers. However, under the high-strength loading scenario, the resulting diffuser air loadings are approximately 2.4 times the acceptable operating values of 1.5 scfm/diffuser under average loading and 4.5 scfm/diffuser under peak loading.

B.1.4 Secondary Clarifier

B.1.4.1 Overview

The Buckskin WWTP is equipped with one 31,450-gallon secondary clarifier with a chain and flight scraper system, an air-lift eductor-type sludge withdrawal mechanism, skimmer trough, and adjustable weir plate at the effluent trough. The basin has two bottom hoppers where the settled sludge is scraped to. From the hoppers, return sludge is sent to Aeration Basin No. 1, and waste activated sludge (WAS) is sent to the aerobic sludge digester.

The capacity estimate presented herein only addresses the basin capacity of the secondary clarifier, and does not address the effectiveness of the mechanical components of the secondary clarifier. Also, the capacity of the RAS/WAS pumping mechanism (air-lift eductor-type sludge withdrawal mechanism) is not addressed in this analysis, due to lack of specific equipment data.

B.1.4.2 Capacity Estimate

The clarifier capacity analysis was based on evaluation of the clarifier safety factor (CSF), which is defined as the ratio between the initial settling velocity (ISV) of the mixed liquor and the basin surface overflow rate (SOR). The CSF was used as the limiting criterion to determine the available capacity of the existing secondary clarifier. The clarifier safety factor at average day flow is established to be, at a minimum, equal to the peaking factor to always maintain a CSF of 1.0 even at peak conditions (and avoid solids blanket to rise). A minimum clarifier safety factor of 3.0 at average day flow conditions was established for this evaluation, due to the peaking factor of 3.0 used in the evaluation.

The ISV of the activated sludge depends on the settling properties of the sludge, and on the MLSS. The settling properties can be directly measured with a settleometer test. When test data are not available, the settling properties can be estimated by using sludge volume index (SVI) data, which is a simpler test that can be routinely measured at wastewater treatment plants. There are several published mathematical correlations that allow estimating settling properties from given SVI values. With the settling properties and an MLSS value, the ISV can be estimated from the correlation chosen for the analysis.

There was no settleometer data or SVI data available for the Buckskin WWTP. Therefore, an SVI value of 150 mL/g and the Pitman correlation were used to estimate ISV based on given MLSS values. These values provide a conservative estimate of ISV and therefore, CSF. The increased flow scenarios were evaluated using the Daigger correlation (and

same SVI of 150 mL/g), which is less conservative, but still provides a reasonable estimate of the settleability properties of the mixed liquor. An SVI of 150 mL/g represents relatively moderate to poor settling characteristics.

Permitted Flow. The existing secondary clarifier has sufficient capacity for the rated plant capacity of 228,000 gpd, at an MLSS of 2,000 mg/L. The resulting CSF for average day flow conditions is 2.9, slightly under the target of 3.0 (which covers the peaking factor of 3.0). This estimate is based on the conservative assumptions of an SVI value of 150 mL/g and using the Pitman correlation for the settling properties of the mixed liquor. An MLSS of 2,000 mg/L is required to maintain sufficient SRT in the aeration basins under the original design and high-strength loading scenarios.

Increased Flow. Using the same conservative assumptions for the settling properties of the mixed liquor as mentioned above for the permitted flow scenarios, the existing secondary clarifier does not have sufficient capacity for the increased flow of 250,000 gpd at an MLSS of 2,200 mg/L. The resulting CSF for average day flow conditions is 2.4, which is below the target of 3.0 and does not cover the peaking factor of 3.0. This estimate is based on the conservative assumptions of an SVI value of 150 mL/g and the Pitman correlation for the settling properties of the mixed liquor. An MLSS of 2,200 mg/L is required to maintain sufficient SRT in the aeration basins under the original design and high-strength loading scenarios.

Given the uncertainty of the settling characteristics of the mixed liquor, a second set of assumptions were used to estimate the impact on the capacity of the secondary clarifier. Instead of using the Pitman correlation, the Daigger correlation was used, but maintaining the same SVI of 150 mL/g. The resulting CSF for a flow of 250,000 gpd and an MLSS of 2,200 mg/L is 2.8, which is slightly below the target value of 3.0 used for the analysis. It is possible that the settling properties of the mixed liquor allow successful operation of the secondary clarifier at the increased flow of 250,000 gpd and an MLSS of 2,200 mg/L. Confirmation of the assumptions used for this analysis of the secondary clarifier require, at a minimum, a long-term set of SVI values, but preferably settleometer test data to directly measure the ISV of the mixed liquor. Given that the secondary clarifier is the bottleneck of the secondary treatment system of the Buckskin WWTP, it is recommended that the District develop a testing plan to routinely monitor settling properties of the mixed liquor.

Redundancy. While the analyses suggest that the existing secondary clarifier is adequate to treat the permitted and increased flows, the lack of redundancy in the secondary clarifier system may limit the overall capacity of the Buckskin WWTP. With only one secondary clarifier available, the facility does not meet the requirements set by ADEQ Engineering Bulletin No. 11 regarding redundancy. According to the design criteria established by ADEQ, facilities must provide the ability to take clarifiers out of service without interrupting plant flow, or have the ability to bypass influent flow when a secondary clarifier is out of service. Currently, the Buckskin WWTP cannot meet either of these requirements.

B.1.5   Denitrification Filters

B.1.5.1   Overview

The Buckskin WWTP has three denitrification filters that receive flow from an intermediate pump station after the secondary clarifier. The purpose of these filters is to reduce nitrate generated in the aeration basins, as there are no anoxic zones for denitrification in the aeration basins. The denitrification filters were originally designed as upflow gravel filters with methanol feed. These filters require a continuous feed of a carbon source (e.g., methanol) to achieve reduced nitrates through denitrification. The methanol feed system was not operational at the time of this study, and the original filter media (gravel) had been replaced with plastic carriers.

B.1.5.2   Capacity Estimate

Permitted Flow. The existing filters were evaluated on the basis of hydraulic loading criteria. Based on hydraulic loading alone, the filters would be able to handle the permitted flow of 228,000 gpd. The average hydraulic loading rate is 1.5 gpm/sf with one unit out of service, which is within an acceptable range for this type of filters. Since there was no performance data available for these filters, it was not possible to evaluate the actual performance of the filters. Collection of regular water quality data before and after the filters is recommended in order to validate the field performance of the denitrification filters and determine whether these filters can provide appropriate nitrogen removal (the carbon feed system would need to be operational). It is worth mentioning, however, that since the plant permit currently does not include nitrogen limits, there is no permit requirement to operate the denitrification filters for nitrogen removal.

Increased Flow. Based on hydraulic loading alone, the filters would be able to handle the increased flow of 250,000 gpd. The average hydraulic loading rate is 1.6 gpm/sf with one unit out of service, which is within an acceptable range for this type of filters. As mentioned above, collection of regular water quality data before and after the filters is recommended in order to validate the field performance of the denitrification filters.

B.1.6   Tertiary Filters

B.1.6.1   Overview

The Buckskin WWTP has three rapid sand filters that receive flow from the denitrification filters. The purpose of these filters is to reduce TSS and turbidity before the final disinfection step. A mudwell next to the filters receives the filter backwash, which is pumped back to the surge tank.

B.1.6.2   Capacity Estimate

Permitted Flow. The existing filters were evaluated on the basis of hydraulic loading criteria. Based on hydraulic loading alone, the filters would be able to handle the permitted flow of 228,000 gpd. The average hydraulic loading rate is 1.5 gpm/sf with one unit out of service, which is within an acceptable range for this type of filters. Under peak flow

conditions, however, all three filters need to be run to stay within acceptable hydraulic loading rates. Since there was no performance data available for these filters, it was not possible to evaluate the actual performance of the filters. Collection of regular water quality data before and after the filters is recommended in order to validate the field performance of the rapid sand filters.

Increased Flow. The existing filters were evaluated on the basis of hydraulic loading criteria. Based on hydraulic loading alone, the filters would be able to handle the increased flow of 250,000 gpd. The average hydraulic loading rate is 1.6 gpm/sf with one unit out of service, which is within an acceptable range for this type of filters. Under peak flow conditions, however, all three filters need to be run to stay within acceptable hydraulic loading rates. Since there was no performance data available for these filters, it was not possible to evaluate the actual performance of the filters. Collection of regular water quality data before and after the filters is recommended in order to validate the field performance of the rapid sand filters.

B.1.7   Disinfection

B.1.7.1   Overview

Effluent from the Buckskin WWTP is equipped with a single, 12,860-gallon, baffled chlorine contact basin/clear well. Chlorine is provided through the addition of liquid chorine solution to the contact basin. The chlorine contact basin receives flow from the rapid sand filters and discharges into the effluent pump station via a "V" notch weir plate at the end of the basin.

B.1.7.2   Capacity Estimate

Permitted Flow and Increased Flow. The overall size of the existing chlorine contact basin is adequate to satisfy the minimum hydraulic retention time specified in ADEQ Engineering Bulletin No. 11 of 15 minutes at peak flow. However, the hydraulics of the basin are not optimal according to specific criteria such as the total length to width ratio, and the width to depth ratio of the basin. Therefore, the evaluation of the minimum hydraulic retention time was performed using a 75 percent modal to actual contact time ratio. Using this assumption, the hydraulic retention times under peak flow conditions for the permitted flow and increased flow scenarios were 21 and 19 minutes, respectively.

There was no detailed equipment information available regarding the chlorine feed system, and therefore an evaluation of the existing chlorine feed capabilities was not performed as part of this evaluation. The chlorine feed system needs to have sufficient capacity to dose sufficient chlorine to achieve disinfection to Class A requirements.

Redundancy. Currently the facility is not equipped with a redundant chlorine contact basin. However, ADEQ does not require fully redundant chlorine contact basins. In addition, because there are no mechanical/moving parts inside the chlorine contact basins that require routine maintenance or replacement, redundancy is not as critical as for other more mechanically intensive processes.

B.1.8 Sludge Digestion

B.1.8.1 Overview

The Buckskin WWTP is equipped with an existing aerobic sludge digester consisting of two basins equipped with fine bubble air bubble diffusion. One basin is the original aerobic sludge digester (24,865 gallons), and the second basin is the effluent lift station that was converted into an aerobic sludge digester. There are airlift supernatant return lines to the aeration basin, and at least one sludge draw-off location. The aeration basin and the digesters utilize the same three 15-hp blowers (two duty and one standby).

B.1.8.2 Capacity Estimate

The existing aerobic sludge digesters were evaluated on the basis of the solids retention time. The digesters receive WAS produced from the aeration basins. Higher wastewater strengths result in increased WAS production and decreased SRT in the digesters.

Aerobic digestion has the potential to stabilize sludge to produce material suitable for beneficial reuse, such as land application (Class B quality per EPA and ADEQ biosolids regulations). However, Class B biosolids are achieved with aerobic digestion when the SRT is at least 40 days at 20 degrees Celsius. Partial stabilization that helps significantly with the production of odors can be achieved at SRT values between 20 and 40 days, without necessarily producing Class B quality biosolids. Biosolids not stabilized to Class B quality standards must be disposed in a landfill per federal and state regulations.

Permitted Flows and Increased Flows. For the permitted flows, the existing aerobic sludge digesters provide a solids retention time of approximately 11 and 6 days at the original design and high strength loading scenarios, respectively. For the increased flows, the estimated SRTs decrease to approximately 10 and 5 days for the two loading scenarios considered. These SRTs were estimated assuming the sludge is decanted to a concentration of 10,000 mg/L. The SRTs can be increased if the sludge is decanted to higher concentrations.

Because the actual solids retention times are much shorter than what is required for complete sludge stabilization, the basins will function primarily as sludge storage tanks. These storage tanks will allow the temporary storage of the WAS solids prior to disposal, as well as achieving partial stabilization. The addition of air in the digester tank will assist in reducing potential production of odors. Thickening the solids to 20,000 mg/L (instead of 10,000 mg/L) would approximately double the estimated SRTs to values that would provide additional partial stabilization for the reduction of odors. Biosolids not stabilized to Class B quality standards must be disposed in a landfill per federal and state regulations.

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Biotran-1602

	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250

| NOTES regarding this application: | Permitted flow Use Pitman for SVI-ISV (default) | | Increased flow (9.6%) Use Pitman for SVI-ISV (default) | | Increased flow (9.6%) Use Daigger for SVI-ISV | |

SUMMARY:

FLOW RATES, mgd:

- Raw WW Flow	0.228	0.228	0.250	0.250	0.250	0.250
- Influent Flow to Activated Sludge	0.243	0.245	0.266	0.268	0.266	0.267

INFLUENT WASTEWATER QUALITY, mg/L

- BOD, mg/L	200	320	200	320	200	320
- TSS, mg/L	220	352	220	352	220	352
- TKN, mg/L	32	52	32	52	32	52

SECONDARY EFFLUENT QUALITY, mg/L:

- BOD (est.), mg/L	3	3	3	3	3	3
- TSS (nominal), mg/L	10	10	10	10	10	10
- NH3-N, mg/L [Note]	0.12	0.15	0.11	0.15	0.11	0.15
- NO3-N, mg/L	18.9	28.8	18.9	28.8	19.0	28.9
- NO2-N, mg/L	0.03	0.05	0.03	0.05	0.03	0.04
- T.I.N., mg/L	19.1	29.0	19.1	29.0	19.1	29.1

TERTIARY EFFLUENT QUALITY, mg/L:

- BOD (est.), mg/L	2	2	2	2	2	2
- TSS (nominal), mg/L	4	4	4	4	4	4
- NH3-N, mg/L	0.12	0.15	0.11	0.15	0.11	0.15
- NO3-N + NO2-N, mg/L	4.0	4.0	4.0	4.0	4.0	4.0
- T.I.N., mg/L	4.1	4.2	4.1	4.1	4.1	4.1
- T.N., mg/L	6.2	6.7	6.2	6.7	6.2	6.7

AERATION BASINS

- # of Basins	2	2	2	2	2	2
- # in Service	2	2	2	2	2	2
- Hydraulic Deten. Time, hr	18.5	18.4	16.9	16.8	17.0	16.8
- Operating Last-Pass MLSS, mg/L	2,000	2,000	2,200	2,200	2,200	2,200
- Design Temperature, deg C	20.0	20.0	20.0	20.0	20.0	20.0
- Unaerated Volume Fraction	0.00	0.00	0.00	0.00	0.00	0.00
- Aerobic SRT, days	9.9	5.5	9.9	5.6	9.9	5.6
-- Min. Aerobic SRT for Nitrification	3.9	3.9	3.9	3.9	3.9	3.9
- Total SRT, days	9.9	5.5	9.9	5.6	9.9	5.6
-- Recommended Min. Total SRT for Nitrification	3.9	3.9	3.9	3.9	3.9	3.9
- F/M, lb BOD Appl./lb MLSS-day	0.12	0.19	0.12	0.19	0.12	0.19
- Aer. BOD Loading, lb BOD/1000 cf-day	15	24	17	27	17	27

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Biotran-1602	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
- Number of Diffusers	252	252	252	252	252	252
- Diffuser air loading (average), scfm/diffuser	1.5	3.3	1.7	3.6	1.7	3.6
- Diffuser air loading (peak), scfm/diffuser	4.5	9.8	5.0	10.8	5.0	10.8
- Process Air Demand (average), scfm	410	850	450	940	450	940
- Process Air Demand (peak), scfm	1,170	2,490	1,290	2,760	1,290	2,760
SECONDARY CLARIFIERS						
- # of Basins	1	1	1	1	1	1
- # in Service	1	1	1	1	1	1
- Sec. Clarifier SOR, gpd/sf	515	510	565	559	565	559
- Sec. Clar. Solids Loading, lb/day-sf	12	12	15	15	14	15
- Clarification Safety Factor (CSF)	2.8	2.9	2.4	2.4	2.8	2.8
-- CSF Target	3.0	3.0	3.0	3.0	3.0	3.0
DENITRIFICATION FILTERS						
- # of Filters	3	3	3	3	3	3
- # in Service	2	2	2	2	2	2
- Nitrate Loading at ADF, lb/d-kcf	41.7	62.7	45.7	68.8	45.7	68.9
- Hydraulic Loading at ADF, gpm/sf	1.5	1.5	1.6	1.6	1.6	1.6
- Contact Time at ADF, minutes	40.8	41.2	37.2	37.6	37.2	37.6
- Methanol required, gal/day	13.5	22.1	14.8	24.3	14.8	24.4
TERTIARY FILTERS						
- # of Filters	3	3	3	3	3	3
- # in Service	2	2	2	2	2	2
- Average Hydraulic Loading, gpm/sf	1.5	1.5	1.6	1.6	1.6	1.6
- Peak Hydraulic Loading, gpm/sf	4.4	4.4	4.8	4.8	4.8	4.8
CHLORINE CONTACT TANK						
- # of Basins	1	1	1	1	1	1
- # in Service	1	1	1	1	1	1
- Estimated Modal Contact time (Peak Flow), min	21	21	19	19	19	19
- Chlorine Dose Required, mg/L	12	12	13	13	13	13
AEROBIC DIGESTER						
- # of Basins	1	1	1	1	1	1
- # in Service	1	1	1	1	1	1
- Decanted Solids Concentration, mg/L	10,000	10,000	10,000	10,000	10,000	10,000
- SRT After Decant, days	11.2	6.3	10.1	5.7	10.1	5.7
- Total Aerobic SRT (A. Basins + Digesters), days	21.1	11.8	20.0	11.2	20.1	11.2
- Process Air Demand (average), scfm	140	290	150	310	150	300
- Process Air Demand (peak), scfm	180	380	200	400	200	390

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Biotran-1602	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
AERATION BLOWERS						
- Number of Duty Blowers	2	2	2	2	2	2
- Number of Standby Blowers	1	1	1	1	1	1
- Total Blower Air Capacity Required (avg), scfm	550	1,140	600	1,250	600	1,240
- Total Blower Air Capacity Required (peak), scfm	1,350	2,870	1,490	3,160	1,490	3,150
- Required Capacity per Blower for avg load, scfm	280	570	300	630	300	620
- Recommended Capacity per Blower (for peak), scfm	680	1,440	750	1,580	750	1,580
- Current Installed Blower Capacity (each), scfm	290	290	290	290	290	290

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Biotran-1602	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250

DETAILED CALCULATIONS:

RAW WASTEWATER (excluding Recycles)

	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
o Plant Flow Rate, mgd	0.2	0.2	0.2	0.2	0.2	0.2
o Flow Characteristic Ratios						
- Max Month/Annual Avg flow ratio	1	1	1	1	1	1
- Peak-hour /Annual Avg	3.0	3.0	3.0	3.0	3.0	3.0
o Wastewater Characteristics						
- BOD, mg/L, Annual Average	**200**	**320**	**200**	**320**	**200**	**320**
-- Mass Load (lb/d) Peaking Factor	1	1	1	1	1	1
-- Effective BOD, mg/L	200	320	200	320	200	320
- TSS, mg/L, Annual Average	**220**	**352**	**220**	**352**	**220**	**352**
-- Mass Load (lb/d) Peaking Factor	1	1	1	1	1	1
-- Effective TSS, mg/L	220	352	220	352	220	352
- Fpv, VSS fraction	0.83	0.83	0.83	0.83	0.83	0.83
-- Effective VSS, mg/L	183	292	183	292	183	292
- NH3-N, mg/L, Annual Average	21.0	34.0	21.0	34.0	21.0	34.0
-- Mass Load (lb/d) Peaking Factor	1	1	1	1	1	1
-- Effective NH3-N, mg/L	21.0	34.0	21.0	34.0	21.0	34.0
Organic-N, mg/L, Annual Average	11.0	18.0	11.0	18.0	11.0	18.0
-- Mass Load (lb/d) Peaking Factor	1	1	1	1	1	1
-- Effective Org-N, mg/L	11.0	18.0	11.0	18.0	11.0	18.0
- NO3-N + NO2-N, mg/L, Annual Average	0	0	0	0	0	0
- Alkalinity, mg/L, Annual Average	250	250	250	250	250	250
- Filterable BOD						
-- fraction, **Fbf**	0.43	0.43	0.43	0.43	0.43	0.43
-- mg/L	86	138	86	138	86	138
- **Fvu**, Fraction VSS that is Unbiodeg [Comment]	0.280	0.280	0.280	0.280	0.280	0.280
o Design Temperature, deg. C						
- Minimum (Winter)	20	20	20	20	20	20
- Maximum (Summer)	30	30	30	30	30	30
- Design	20	20	20	20	20	20

RECYCLE TO HEADWORKS/PRIM CLAR.S

	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
o Flow Rate, mgd						
- Backwash Flow	0.012	0.012	0.013	0.013	0.013	0.013

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Biotran-1602	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o Wastewater Characteristics, mg/L						
- Total Recycle						
-- BOD	28	27	28	27	28	27
-- TSS	120	120	120	120	120	120
-- VSS	89	91	89	91	89	91
-- NH3-N	0	0	0	0	0	0
-- Organic-N	8	9	8	9	8	9
-- NO3-N + NO2-N	4	4	4	4	4	4
-- Alkalinity	168	125	168	125	168	125
-- Filterable ("soluble") BOD	0.7	0.8	0.7	0.8	0.7	0.8
-- Total soluble Organic N	1.9	2.3	1.9	2.3	1.9	2.3
-- Fpv, VSS fraction	0.74	0.76	0.74	0.76	0.74	0.76
- Fvu, Fraction Total VSS that is Unbiodeg	0.700	0.700	0.700	0.700	0.700	0.700
RECYCLE TO ACTIVATED SLUDGE						
o Flow Rate, mgd						
- Dewatering/Decant directly from Digester	0.002	0.005	0.002	0.004	0.002	0.004
- Spray Water to Basins	0.001	0.001	0.001	0.001	0.001	0.001
- Total	0.003	0.005	0.003	0.005	0.003	0.005
o Wastewater Characteristics, mg/L						
- Total Recycle						
-- BOD	7	15	8	16	7	16
-- TSS	116	131	112	128	108	126
-- VSS	79	92	77	90	74	89
-- NH3-N	1	1	1	1	1	1
-- Organic-N	8	9	8	9	7	9
-- NO3-N + NO2-N	8	9	7	9	7	8
-- Alkalinity	110	83	107	81	103	80
-- Filterable ("soluble") BOD	0.0	0.0	0.0	0.0	0.0	0.0
-- Total soluble Organic N	2.3	2.6	2.2	2.6	2.2	2.5
-- Fpv, VSS fraction	0.68	0.70	0.68	0.70	0.68	0.71
- Fvu, Fraction VSS that is Unbiodeg	0.700	0.700	0.700	0.700	0.700	0.700
ACTIVATED SLUDGE PROCESS						
o Flow Rate, mgd						
- Main-Stream Influent	0.240	0.240	0.263	0.263	0.263	0.263
- Recycle directly to AS	0.003	0.005	0.003	0.005	0.003	0.005
- Total Main Influent to Activated Sludge	0.243	0.245	0.266	0.268	0.266	0.267

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Biotran-1602	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o Influent Characteristics, mg/L						
- Total BOD	189	299	189	300	190	301
- TSS	214	336	214	337	214	337
- VSS	177	278	177	279	177	279
- NH3-N	20	32	20	32	20	32
- Organic-N	11	17	11	17	11	17
- NO3-N + NO2-N	0	0	0	0	0	0
- Alkalinity	244	240	244	241	244	241
- Filterable ("soluble") BOD	81	128	81	128	81	129
- Fpv, VSS fraction	0.83	0.83	0.83	0.83	0.83	0.83
- AB Influent D.O. Concentration, mg/L	0.0	0.0	0.0	0.0	0.0	0.0
o Basin dimensions						
- Main Basins						
-- No. of Basins	2	2	2	2	2	2
-- Number of Units in Service	2	2	2	2	2	2
-- Length, ft (inside)	44	44	44	44	44	44
-- Width, ft (inside)	30	30	30	30	30	30
-- Side Water Depth, ft	9.5	9.5	9.5	9.5	9.5	9.5
.. Recomm inside Wall height, incl. Freeboard, ft	12.5	12.5	12.5	12.5	12.5	12.5
-- Liquid Volume per Basin, mil gal	0.094	0.094	0.094	0.094	0.094	0.094
-- Liquid Volume per Basin, gal	93,799	93,799	93,799	93,799	93,799	93,799
o Total Volume of Basins, mil gal						
- Total Basin volume in service	0.188	0.188	0.188	0.188	0.188	0.188
-- Reduction for MBR cassettes	0.000	0.000	0.000	0.000	0.000	0.000
- Biological Reaction Volume	0.188	0.188	0.188	0.188	0.188	0.188
o Aerated Zone BOD Loading, lb/1,000 cf-day	15.2	24.3	16.6	26.6	16.6	26.6
o Hydraulic Detention Time, hr	18.5	18.4	16.9	16.8	17.0	16.8
o Selected Operating L-P MLSS, mg/L	2,000	2,000	2,200	2,200	2,200	2,200
PROCESS LAYOUT						
o Zone Sizes (Fraction of Total Volume)						
- Zone 1	0.500	0.500	0.500	0.500	0.500	0.500
- Zone 2	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 3	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 4	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 5	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 6	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 7 (by difference)	0.500	0.500	0.500	0.500	0.500	0.500
-- Total	1.000	1.000	1.000	1.000	1.000	1.000

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	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o DO in each Zone (Unaerated, Set = 0), mg/L						
- Zone 1	2.0	2.0	2.0	2.0	2.0	2.0
- Zone 2	2.0	2.0	2.0	2.0	2.0	2.0
- Zone 3	2.0	2.0	2.0	2.0	2.0	2.0
- Zone 4	2.0	2.0	2.0	2.0	2.0	2.0
- Zone 5	2.0	2.0	2.0	2.0	2.0	2.0
- Zone 6	2.0	2.0	2.0	2.0	2.0	2.0
- Zone 7	2.0	2.0	2.0	2.0	2.0	2.0
o Enhanced Simultaneous Nitrif-Denitrification						
- Apply enhanced SND? Y=1, N=0	0	0	0	0	0	0
o Aerated/Unaerated Fractions						
- Total Unaerated Volume Fraction	0.00	0.00	0.00	0.00	0.00	0.00
-- Total Unaerated Volume, mil gal	0.00	0.00	0.00	0.00	0.00	0.00
- Total Aerated Volume Fraction	1.00	1.00	1.00	1.00	1.00	1.00
-- Total Aerated Volume, mil gal	0.19	0.19	0.19	0.19	0.19	0.19
- Total Aerated Mass Fraction	1.00	1.00	1.00	1.00	1.00	1.00
o Plant Influent Flow Routing						
- Fraction to Zone 1	1.00	1.00	1.00	1.00	1.00	1.00
o Return Sludge Routing						
- Fraction to Zone 1	1.00	1.00	1.00	1.00	1.00	1.00
o Sludge Wasting Method						
- Wasting from RAS (1) or ML (0)	1	1	1	1	1	1
LOADING CRITERIA						
o BOD Applied, lb/d						
- BOD in Influent	383	612	420	670	420	670
- BOD in External Stream	0	0	0	0	0	0
- (-) WAS BOD Recycled	3	3	3	3	3	3
- Net BOD Load	380	609	417	668	417	668
o MLSS under aeration, lb	3,157	3,166	3,470	3,479	3,470	3,480
- F/M, lb BOD Appl./lb MLSS-day	0.12	0.19	0.12	0.19	0.12	0.19
o Organic Loading, Based on Aerated Zone						
- Aerated Volume in Service, 1,000 cf	25	25	25	25	25	25
- Aer. BOD Loading, lb BOD/1000 cf-day	15.2	24.3	16.6	26.6	16.6	26.6
WAS SOLIDS PRODUCTION						
o Solids Production, TSS, lb/d						
- TSS Entering in Feed, lb/d	504	800	552	876	552	876
- TSS Entering in External Input, lb/d	0	0	0	0	0	0
- VSS Change in A.B. Zones	-183	-239	-201	-263	-201	-263
- ISS Change in A.B. Zones	11	21	12	24	12	24
- ISS due to Bio-P (Est.), lb/d	0	0	0	0	0	0
- Unbiodeg VSS due to Bio-P (Est.), lb/d	0	0	0	0	0	0
- **Total Solids Production, lb/d**	**332**	**582**	**363**	**637**	**363**	**637**

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Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
MLSS CHARACTERISTICS						
o Mixed Liquor Components, mg TSS/L (from Final Zone)						
- Solids, mg TSS/L						
-- Slowly Biodegradable	17	25	19	28	19	28
-- Active Biomass	555	718	610	790	610	790
-- Endogenous Biomass	262	192	289	212	290	212
-- Ammonia Oxidizers	11	12	12	14	12	14
-- Nitrite Oxidizers	7	8	7	8	7	8
-- Unbiodegradable VSS (Influent + Bio-P)	632	566	695	622	694	621
-- Unbiodegradable VSS from External input	0	0	0	0	0	0
-- Inorganic SS (influent + Biogrowth)	519	480	571	528	570	528
-- Inorganic SS in External input	0	0	0	0	0	0
-- Inorganic SS due to Bio-P (est.)	0	0	0	0	0	0
-- Total Last-Pass MLSS	2,002	2,001	2,202	2,201	2,202	2,201
-- Total Organic-N	106.4	115.4	116.9	126.6	116.8	126.6
-- Alkalinity, mg/L as CaCO3	114	37	114	37	114	37
o Org N fraction of MLVSS (NinVSS)	0.071	0.074	0.070	0.074	0.070	0.074
o MLVSS Fraction	0.74	0.76	0.74	0.76	0.74	0.76
o BOD of AS Solids						
- BOD/TSS ratio	0.20	0.27	0.20	0.27	0.20	0.27
SOLIDS RETENTION TIME, SRT						
o Total Solids Wasted, lb/d	332	582	363	637	363	637
- Recycled WAS Solids, lb/d	12	12	13	13	13	13
- Net lb Solids Yield/day	320	571	350	624	350	624
o Total BOD Load, lb/d	383	612	420	670	420	670
- Recycled BOD, lb/d	3	3	3	3	3	3
- Net BOD Load, lb/d	380	609	417	668	417	668
o Solids Production						
- lb Dry SS/lb BOD Applied	0.841	0.937	0.840	0.935	0.839	0.934
o Total Mass TSS in System, lb	3,157	3,166	3,470	3,479	3,470	3,480
- **Total SRT (Rs), days**	**9.87**	**5.55**	**9.91**	**5.57**	**9.92**	**5.58**
o Total Mass TSS in Aerated Zones, lb	3,157	3,166	3,470	3,479	3,470	3,480
- Nominal Aerated Mass Fraction	1.000	1.000	1.000	1.000	1.000	1.000
- **Nominal Aerobic SRT, days**	**9.87**	**5.55**	**9.91**	**5.57**	**9.92**	**5.58**
o Mass Fraction in Each Zone						
- Zone 1	0.504	0.506	0.504	0.505	0.504	0.505
- Zone 2	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 3	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 4	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 5	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 6	0.000	0.000	0.000	0.000	0.000	0.000
- Zone 7	0.496	0.494	0.496	0.495	0.496	0.495
	1.000	1.000	1.000	1.000	1.000	1.000

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Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o Min. Aer. SRT recommended for nitrification, days	3.9	3.9	3.9	3.9	3.9	3.9
- Washout SRT(total)						
-- Rwashout = 1/(Ua*DOsw - ba)	1.56	1.56	1.56	1.56	1.56	1.56
- Recommended Operating SRT						
-- Max recomm. change in NH3, mg/L, for *10% change in SRT*	0.10	0.10	0.10	0.10	0.10	0.10
-- Max slope criterion, as dNH3/dSRT, mg/L-d	0.26	0.26	0.26	0.26	0.26	0.26
-- Recomm. Min. Operating SRT(total)	3.9	3.9	3.9	3.9	3.9	3.9
-- Recomm. Min. Op. SRT(Nominal aerobic)	3.9	3.9	3.9	3.9	3.9	3.9
-- Nitrification Safety Factor	2.47	2.47	2.47	2.47	2.47	2.47
AERATION REQUIREMENTS						
o Oxygen Required, lb/d						
- Net Oxygen Demand in Zone 1	394	578	430	630	430	631
- Net Oxygen Demand in Zone 2	0	0	0	0	0	0
- Net Oxygen Demand in Zone 3	0	0	0	0	0	0
- Net Oxygen Demand in Zone 4	0	0	0	0	0	0
- Net Oxygen Demand in Zone 5	0	0	0	0	0	0
- Net Oxygen Demand in Zone 6	0	0	0	0	0	0
- Net Oxygen Demand in Zone 7	183	259	203	287	202	286
- (-) Oxygen provided by MBR Scouring	0	0	0	0	0	0
- Total Oxygen required lb/d	**577**	**836**	**633**	**917**	**633**	**917**
o Diffuser Analysis						
Note:						
All values of air and blower requirements given below are preliminary estimates, to be refined during detailed design						
o Oxygen Transfer Efficiency	[Sanitaire]	[Sanitaire]	[Sanitaire]	[Sanitaire]	[Sanitaire]	[Sanitaire]
- Diffuser Type	Membrn Disk 9"	Membrn Disk 9"	Membrn Disk 9"	Membrn Disk 9"	Membrn Disk 9"	Membrn Disk 9"
- Aeration Basin D.O. (Avg), mg/L	2.0	2.0	2.0	2.0	2.0	2.0
- Design Water Temperature, C	30	30	30	30	30	30
- Diffuser submergence, ft	8.6	8.6	8.6	8.6	8.6	8.6
- Air loading, scfm/unit [Note]	1.51 scfm/dfr	3.25 scfm/dfr	1.67 scfm/dfr	3.61 scfm/dfr	1.67 scfm/dfr	3.61 scfm/dfr
- Floor Coverage	25.6 At/Ad	25.6 At/Ad	25.6 At/Ad	25.6 At/Ad	25.6 At/Ad	25.6 At/Ad

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Biotran-1602

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Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
- Clean Water SOTE	14.8	13.1	14.6	12.8	14.6	12.8
- Site Conditions Adjustment Factor						
F = Actual / Standard OTE						
-- Alpha factor, including fouling	0.53	0.40	0.53	0.40	0.53	0.40
-- Theta factor	1.024	1.024	1.024	1.024	1.024	1.024
-- Temp. correction, Tau	0.83	0.83	0.83	0.83	0.83	0.83
-- Elevation above MSL, ft	423	423	423	423	423	423
-- ..Pressure correction, Omega	0.98	0.98	0.98	0.98	0.98	0.98
-- Beta factor	0.99	0.99	0.99	0.99	0.99	0.99
-- Equilibrium C*20	9.92	9.92	9.92	9.92	9.92	9.92
..Depth Adjustment Factor	0.37	0.37	0.37	0.37	0.37	0.37
- F = Alpha x [Theta ^(T-20)]	0.41	0.31	0.41	0.31	0.41	0.31
x (Tau Beta Omega C*20 - C)/C*20						
- Oxygen Transfer Efficiency	6.06	4.05	5.98	3.98	5.99	3.99
OTE = F x SOTE	Percent	Percent	Percent	Percent	Percent	Percent
Preliminary Estimate						
o SOTR Required						
- Average Day @ Design flow						
-- Actual Ox Tr Requd, AOTR, lb/d	577	836	633	917	633	917
-- Site Conditions Adjustment, F	0.41	0.31	0.41	0.31	0.41	0.31
-- Standard Ox Tr Rate, SOTR, lb/d	1,409	2,701	1,541	2,956	1,540	2,955
SOTR = AOTR / F						
o Air Supply Required						
- Average Day @ Design flow						
-- Ox Transfer Rate, AOTR, lb/d	577	836	633	917	633	917
-- Oxygen Supplied, lb/min	6.6	14.4	7.3	16.0	7.3	16.0
-- cf Air/lb Oxygen	57.0	57.0	57.0	57.0	57.0	57.0
[23.3 lb O2/100 lb Air]						
[0.0753 lb Air/scf]						
-- Process Air, scfm	380	820	420	910	420	910
..scfm per lb/d Oxygen	0.658	0.981	0.664	0.993	0.664	0.993
..scf/lb BOD Applied	1,438	1,938	1,450	1,963	1,450	1,963
-- Other Uses, e.g. Channel Air	30	30	30	30	30	30
-- Total Blower Air, scfm	410	850	450	940	450	940
- Peak Day @ Design Flow						
-- Peaking factor	3	3	3	3	3	3
-- Process Air, scfm	1,140	2,460	1,260	2,730	1,260	2,730
-- Total Blower Air, scfm	1,170	2,490	1,290	2,760	1,290	2,760

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Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o Diffusers						
- Expressed as active sq ft or # diffusers	dfr	dfr	dfr	dfr	dfr	dfr
- Recommended						
-- Air Loading, scfm/(sf or dfr)	0.95	0.95	0.95	0.95	0.95	0.95
-- Number recommended per Basin	198	431	220	479	220	479
- Actual Installed, per basin						
-- Main Basin	126	126	126	126	126	126
-- Additional Basin	0	0	0	0	0	0
- Total Installed, sf or dfr	252	252	252	252	252	252
- Air Loading, scfm/sf or dfr						
-- Daily Average	1.51	3.25	1.67	3.61	1.67	3.61
-- Peak	4.52	9.76	5.00	10.83	5.00	10.83
- Floor Coverage						
-- Total Aerobic Floor Area in Service, sf	2,640	2,640	2,640	2,640	2,640	2,640
-- Total Floor Area with diffusers	2,640	2,640	2,640	2,640	2,640	2,640
-- Coverage	25.6	25.6	25.6	25.6	25.6	25.6
.. Expressed as	At/Ad	At/Ad	At/Ad	At/Ad	At/Ad	At/Ad
- Active sf/diffuser, or 1	1	1	1	1	1	1
- Number of diffuser units	252	252	252	252	252	252
o Blower Discharge pressure						
- Head, ft water						
-- Submergence	8.6	8.6	8.6	8.6	8.6	8.6
-- Freeboard above normal op level	0.0	0.0	0.0	0.0	0.0	0.0
-- Diffuser head loss	1.0	1.0	1.0	1.0	1.0	1.0
-- Pipe & Valve friction	2.5	2.5	2.5	2.5	2.5	2.5
-- Total Head, ft	12.1	12.1	12.1	12.1	12.1	12.1
- Discharge pressure, psig	5.2	5.2	5.2	5.2	5.2	5.2
o Delivered Horsepower						
- Max Operating Air Temp, C	35	35	35	35	35	35
- Barometric Pressure, psia	14.5	14.5	14.5	14.5	14.5	14.5
- Blower Suction Pressure, psia	14.2	14.2	14.2	14.2	14.2	14.2
- Daily Average Total Air, scfm	410	850	450	940	450	940
- Avg Delivered Horsepower, hp	10	20	10	22	10	22
- Peak Day Delivered hp	27	58	30	64	30	64
o Wire power required						
- Energy Efficiency, %	61.0	61.0	61.0	61.0	61.0	61.0
- Wire power required, hp						
-- Daily Average	20	30	20	40	20	40
-- Firm Installed	40	100	50	110	50	110

CAROLLO ENGINEERS, PC						
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Biotran-1602	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
SECONDARY SEDIMENTATION BASINS						
o Flow Rates, mgd						
- AS Influent, Q	0.243	0.245	0.266	0.268	0.266	0.267
- Net Sed. Basin Inflow (excl. RAS), Qci	0.243	0.245	0.266	0.268	0.266	0.267
- Return Sludge Flow, Qr	0.088	0.089	0.104	0.103	0.097	0.097
(not including waste sludge flow)						
- Total Sed Basin Inflow	0.331	0.334	0.370	0.371	0.362	0.365
- Total Sed. Basin Underflow	0.094	0.099	0.110	0.113	0.102	0.107
- Net Sec. Effluent, Qe	0.238	0.235	0.260	0.258	0.260	0.258
o Basin dimensions						
-- No. of Basins	1	1	1	1	1	1
-- Number of Units in Service	1	1	1	1	1	1
-- Length, ft (inside)	29.75	29.75	29.75	29.75	29.75	29.75
-- Width, ft (inside)	15.50	15.50	15.50	15.50	15.50	15.50
.. L/W Ratio	1.92	1.92	1.92	1.92	1.92	1.92
-- Side Water Depth, ft	9.5	9.5	9.5	9.5	9.5	9.5
-- Surface Area per Basin, sf	461	461	461	461	461	461
-- Volume per Basin, cf	4,381	4,381	4,381	4,381	4,381	4,381
o Flow Split						
- Fraction of ML Flow to Group 1:						
o Surface Overflow Rate						
-- Surface Area in service, sf	461	461	461	461	461	461
-- Surface Overflow Rate, gpd/sf	515	510	565	559	565	559
o Solids Loading Rate, lb/day-sf						
- Group 1	12	12	15	15	14	15
o Volume in service, mil gal						
- Group 1	0.03	0.03	0.03	0.03	0.03	0.03
o Hydraulic Detention Time, hr (based on Q)						
- Group 1	3.2	3.2	3.0	2.9	3.0	2.9
o Weir Loading						
- Group 1						
-- Actual weir length per unit, ft	62	62	62	62	62	62
-- Weir loading, gpd/ft	3,832	3,792	4,199	4,155	4,199	4,155
o Sludge Settling Characteristics						
ISV = V_0 exp(- MLSS/X_M), ft/h						
- Design Settling Constants						
-- V_0, ft/hr	**19.5**	**19.5**	**19.5**	**19.5**	**21.3**	**21.3**
-- X_M, mg/L	**2,300**	**2,300**	**2,300**	**2,300**	**2,480**	**2,480**
o Target Settling Values						
- Effluent rise rate (SOR), ft/hr						
-- Group 1	2.87	2.84	3.14	3.11	3.14	3.11
- Clarification Safety Factor, CSF	3.00	3.00	3.00	3.00	3.00	3.00
- Required Initial Settling Velocity, ISV, ft/hr	8.6	8.5	9.4	9.3	9.4	9.3
- Preferred Max. Last-Pass MLSS, mg/L	1,880	1,905	1,670	1,694	2,020	2,046

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Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o Selected Settling Values						
- Operating L-P MLSS conc, mg/L	2,000	2,000	2,200	2,200	2,200	2,200
- Operating ISV, ft/h	8.17	8.17	7.49	7.49	8.77	8.77
- Operating CSF						
-- Group 1	2.85	2.88	2.38	2.41	2.79	2.82
SLUDGE RETURN AND WASTAGE						
o Wasting Method (see Process Layout)						
- Waste Flow from RAS, Qw	0.005	0.010	0.006	0.010	0.005	0.010
o Return Sludge						
- Qr/Q, fraction (based on Qr to Aer Basin)	0.36	0.36	0.39	0.38	0.36	0.36
-- [Ratio based on Clarifier Underflow/Q]	0.39	0.41	0.41	0.42	0.38	0.40
- RAS flow to Aer Basin, Qr, mgd Average	0.088	0.089	0.104	0.103	0.097	0.097
- RAS flow to Aer Basin, Qr, gpm Average	61	62	72	71	67	68
- RAS concentration, mg/L	7,042	6,709	7,393	7,193	7,791	7,457
o Sludge Wastage						
- Total Solids Wasted, lb/d	332	582	363	637	363	637
- Adjustment for ESS:						
-- Solids in Effluent, lb/d	20	20	22	21	22	21
-- Solids in WAS, lb/d	312	563	341	616	341	615
- Wasting from -	RAS	RAS	RAS	RAS	RAS	RAS
- WAS Concentration, mg/L	7,042	6,709	7,393	7,193	7,791	7,457
- Organic N, lb/d	16	32	18	35	18	35
- Flow Rate, mgd Average	0.005	0.010	0.006	0.010	0.005	0.010
- Flow Rate, gpm Average	3.7	7.0	3.8	7.1	3.6	6.9
o WAS Characteristics, mg/L						
- BOD	1,443	1,785	1,515	1,914	1,596	1,984
- TSS	7,042	6,709	7,393	7,193	7,791	7,457
- VSS	5,217	5,099	5,478	5,467	5,773	5,669
- NH3-N	0.1	0.2	0.1	0.1	0.1	0.1
- Organic-N	369.7	381.3	388.0	408.5	408.7	423.4
- NO3-N + NO2-N	18.9	28.8	18.9	28.8	19.0	28.9
- Alkalinity	114	37	114	37	114	37
- Filterable ("soluble") BOD	0.7	0.8	0.7	0.8	0.7	0.8
- Total soluble Organic N	1.9	2.3	1.9	2.3	1.9	2.3
SECONDARY EFFLUENT						
o Flow Rate						
- Net Secondary Effluent, mgd	0.238	0.235	0.260	0.258	0.260	0.258

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Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o Secondary Effluent Quality						
- BOD, mg/L	3	3	3	3	3	3
- TSS (nominal), mg/L	10	10	10	10	10	10
- VSS, mg/L	7.4	7.6	7.4	7.6	7.4	7.6
- NH3-N, mg/L	0.1	0.2	0.1	0.1	0.1	0.1
- Total Organic N, mg/L	2.4	2.9	2.4	2.9	2.4	2.9
- NO3-N, mg/L	18.9	28.8	18.9	28.8	19.0	28.9
- NO2-N, mg/L	0.0	0.0	0.0	0.0	0.0	0.0
- Alkalinity, mg/L	114	37	114	37	114	37
- Soluble Organic N, mg/L	1.9	2.3	1.9	2.3	1.9	2.3
- T.I.N., mg/L	19.1	29.0	19.1	29.0	19.1	29.1
- Total N, mg/L	21.5	31.8	21.5	31.9	21.5	31.9
DENITRIFICATION FILTERS						
o Influent						
- Flow, mgd	0.238	0.235	0.260	0.258	0.260	0.258
- Nitrate, mg/L	18.9	28.8	18.9	28.8	19.0	28.9
o Effluent Nitrate Required, mg/L	4.0	4.0	4.0	4.0	4.0	4.0
o Nitrate Removal Rate, %	79	86	79	86	79	86
o Effluent Alkalinity, mg/L	168	125	168	125	168	125
o Filters						
- Number of Filters	3	3	3	3	3	3
- Number of Units in Service	2	2	2	2	2	2
- Filter Size (per Filter)						
-- Length	7.5	7.5	7.5	7.5	7.5	7.5
-- Width	7.5	7.5	7.5	7.5	7.5	7.5
-- Expanded Bed Height	8	8	8	8	8	8
-- Filter Area, sf	56.25	56.25	56.25	56.25	56.25	56.25
-- Filter Volume, kcf	0.45	0.45	0.45	0.45	0.45	0.45
- Loading Rate						
-- Hydraulic, gpm/sf	1.47	1.45	1.61	1.59	1.61	1.59
-- Nitrate, lb/d-kcf	41.7	62.7	45.7	68.8	45.7	68.9
-- Contact Time, minutes	40.8	41.2	37.2	37.6	37.2	37.6
o Methanol required						
- Nitrate removed, lb/d	30	49	32	53	32	53
- lb Methanol/lb NO3-N	3	3	3	3	3	3
- Methanol used, lb/d	89	146	97	160	97	160
-- Gal/day	13	22	15	24	15	24

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Biotran-1602	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
TERTIARY FILTRATION	In Service	In Service	In Service	In Service	In Service	In Service
o Tertiary Filtration in Service? (Y=1, N=0)	1	1	1	1	1	1
o Influent						
- Flow, mgd						
-- Total	0.238	0.235	0.260	0.258	0.260	0.258
- BOD, total, mg/L	3.0	3.0	3.0	3.0	3.0	3.0
- SS, total, mg/L	10.0	10.0	10.0	10.0	10.0	10.0
o Filter Area						
- Surface Area per Filter, sf	56.25	56.25	56.25	56.25	56.25	56.25
- Backwash - Continuous (0) or Intermittent (1)?	0	0	0	0	0	0
- Standby Units Provided	1	1	1	1	1	1
- Number of Filters						
-- Existing	3	3	3	3	3	3
-- New	0	0	0	0	0	0
-- Total	3	3	3	3	3	3
- Number of Units in Service	2	2	2	2	2	2
o Filter Loading						
- Equalization provided? (Y=1, N=0)	0	0	0	0	0	0
- Peaking factor	3.00	3.00	3.00	3.00	3.00	3.00
- Surface Area in Service, sf	113	113	113	113	113	113
- Liquid Loading Rate, gpm/sf						
-- At Daily Average Flow, gpm/sf	1.47	1.45	1.61	1.59	1.61	1.59
-- At Peak Flow, gpm/sf	4.40	4.35	4.82	4.77	4.82	4.77
o Removal						
- SS Removal, %	60	60	60	60	60	60
- SS removed, lb/d	12	12	13	13	13	13
- BOD removed, lb/d	3	3	3	3	3	3
o Backwash Flow						
- Percent of Flow, %	5	5	5	5	5	5
- Backwash Flow, mgd	0.012	0.012	0.013	0.013	0.013	0.013
- Backwash Flow, gpm	8.2	8.2	9.0	8.9	9.0	8.9
o Backwash Characteristics, mg/L						
- BOD	28	27	28	27	28	27
- TSS	120	120	120	120	120	120
- VSS	89	91	89	91	89	91
- NH3-N	0.1	0.2	0.1	0.1	0.1	0.1
- Organic-N	8	9	8	9	8	9
- NO3-N + NO2-N	4.0	4.0	4.0	4.0	4.0	4.0
- Alkalinity	168	125	168	125	168	125
o Net Flow to Disinfection, mgd						
- Undisinfected Plant Water Used	0.00	0.00	0.00	0.00	0.00	0.00
- To Disinfection	0.23	0.22	0.25	0.24	0.25	0.24

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	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o Tertiary Effluent Quality, mg/L						
- BOD	1.6	1.7	1.6	1.7	1.6	1.7
- SS	4.0	4.0	4.0	4.0	4.0	4.0
- VSS, mg/L	3.0	3.0	3.0	3.0	3.0	3.0
- NH3-N, mg/L	0.1	0.2	0.1	0.1	0.1	0.1
- Total Organic N, mg/L	2.1	2.5	2.1	2.5	2.1	2.5
- NO3-N + NO2-N, mg/L	4.0	4.0	4.0	4.0	4.0	4.0
- Alkalinity, mg/L	168	125	168	125	168	125
- Filterable ("soluble") BOD	0.7	0.8	0.7	0.8	0.7	0.8
- Soluble Organic N, mg/L	1.9	2.3	1.9	2.3	1.9	2.3
- T.I.N., mg/L	4.1	4.2	4.1	4.1	4.1	4.1
- Total N, mg/L	6.2	6.7	6.2	6.7	6.2	6.7
CHLORINE CONTACT TANKS	In Service	In Service	In Service	In Service	In Service	In Service
o Flow Rate, mgd	0.226	0.223	0.247	0.245	0.247	0.245
- Peaking factor	3.0	3.0	3.0	3.0	3.0	3.0
o Number of Tanks	1	1	1	1	1	1
o Volume per Tank, mil gal	0.013	0.013	0.013	0.013	0.013	0.013
- Number of Passes per tank	2	2	2	2	2	2
- Length per pass, ft	23.5	23.5	23.5	23.5	23.5	23.5
- Width per pass, ft	7	7	7	7	7	7
- Side Water Depth, ft	5.25	5.25	5.25	5.25	5.25	5.25
- Flow Length: Width Ratio	6.7	6.7	6.7	6.7	6.7	6.7
o Detention Time @ peak, all UIS						
- Hydraulic residence time, HRT, min.	27	28	25	25	25	25
- Assumed Modal/Actual contact time ratio	75%	75%	75%	75%	75%	75%
- Estimated Modal Contact Time, min	21	21	19	19	19	19
o Capacity with one Tank OOS, mgd	0.00	0.00	0.00	0.00	0.00	0.00
o Chlorine Dose						
- Required Cr T, (mg/L)(min.)	200	200	200	200	200	200
- Requ'd (calc) Chlorine Residual, mg/L	9.7	9.6	10.6	10.5	10.6	10.5
- Allowance for Influent chlorine demand, mg/L	2	2	2	2	2	2
- Average Chlorine Dose, mg/L	11.7	11.6	12.6	12.5	12.6	12.5
- Average Chlorine Consumption, lb/d	22	22	26	26	26	26
FINAL EFFLUENT						
o Flow Rate, mgd						
- Plant Water used	0.002	0.002	0.002	0.002	0.002	0.002
- Final Effluent Flow	0.224	0.222	0.246	0.243	0.246	0.243

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	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250

RESIDUALS MANAGEMENT

SOLIDS GENERATED

o Total Waste Activated Sludge						
- Flow, mgd	0.005	0.010	0.006	0.010	0.005	0.010
- Solids, lb/d	312	563	341	616	341	615
- Concentration, %	0.70	0.67	0.74	0.72	0.78	0.75
- VSS, %	74	76	74	76	74	76
- Volatile solids, lb/d	231	428	253	468	253	468
- Organic N, lb/d	16	32	18	35	18	35

SLUDGE ROUTING

o Waste Activated Sludge						
- (a) Thickening	None	None	None	None	None	None
- (b) Then routed to - -	Aero Diges	Aero Diges	Aero Diges	Aero Diges	Aero Diges	Aero Diges

AEROBIC DIGESTER

	In Service	In Service	In Service	In Service	In Service	In Service
o Feed	WAS	WAS	WAS	WAS	WAS	WAS
- Flow, mgd	0.005	0.010	0.006	0.010	0.005	0.010
- Solids, lb/d	312	563	341	616	341	615
- Volatile Solids, lb/d	231	428	253	468	253	468
- Organic N, total, lb/d	16	32	18	35	18	35
- Filterable Components, mg/L						
-- NH3-N, mg/L	0.1	0.2	0.1	0.1	0.1	0.1
-- NO3-N + NO2-N, mg/L	18.9	28.8	18.9	28.8	19.0	28.9
-- Alkalinity, mg/L	114	37	114	37	114	37
-- Filterable ("soluble") BOD, mg/L	0.7	0.8	0.7	0.8	0.7	0.8
-- Total soluble Organic N, mg/L	1.9	2.3	1.9	2.3	1.9	2.3
o VSS destruction						
- VSS Destruction Reaction Rate						
-- First Order Rate Constant, K, day-1	0.2	0.2	0.2	0.2	0.2	0.2
-- Solids Detention Time, days	11.2	6.3	10.1	5.7	10.1	5.7
-- Kinetic rate factor	0.691	0.557	0.670	0.531	0.670	0.531
- VSS destroyed, %	25.3	25.8	24.5	24.6	24.5	24.6
- VSS destroyed, lb/d	59	110	62	115	62	115
- Solids remaining, lb/d	253	453	279	501	279	500
- Undecanted concentration, mg/L	5,721	5,395	6,050	5,848	6,376	6,063
- VSS fraction remaining	0.68	0.70	0.68	0.70	0.68	0.71
o Dewatering/Decant flow						
- Sludge Conc. Target, mg/L	10,000	10,000	10,000	10,000	10,000	10,000
- Dewatering, Percent of Influent	43.4	46.8	40.1	42.2	36.8	40.0
- Return flow, mgd	0.002	0.005	0.002	0.004	0.002	0.004
- Remaining flow, mgd	0.003	0.005	0.003	0.006	0.003	0.006
- Sludge Conc., mg/L	10,000	10,000	10,000	10,000	10,000	10,000

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	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o Dewatering/Decant Operation						
- Dewater from Influent (1) or from Digester (0)?	0	0	0	0	0	0
- Actual Feed Flow to (First) Aeration Tank, mgd	0.005	0.010	0.006	0.010	0.005	0.010
o Basin Sizes						
- Number of Units [Note]	1	1	1	1	1	1
-- Area, each, sf	692	692	692	692	692	692
-- Peak Side Water Depth, ft	9.8	9.8	9.8	9.8	9.8	9.8
-- Average Operating Depth, ft	6.5	6.5	6.5	6.5	6.5	6.5
-- Total volume, mil gal	0.034	0.034	0.034	0.034	0.034	0.034
o Detention Time						
- Solids reten time (after decant), d	11.2	6.3	10.1	5.7	10.1	5.7
-- Temperature, C	20.0	20.0	20.0	20.0	20.0	20.0
-- T x SRT	224	126	203	113	203	113
- Total SRT, including A/S Aer Basin, days	21.1	11.8	20.0	11.2	20.1	11.2
o Intermittent Operation for N Removal						
- Total hours aerating per day	16	16	16	16	16	16
o Nitrogen Balance over Digester(s)						
- Influent to (First) Aeration Tank						
-- NH3-N, lb/d	0.0	0.0	0.0	0.0	0.0	0.0
-- NO3-N + NO2-N, lb/d	0.8	2.4	0.9	2.5	0.8	2.4
- Organic N released as NH3-N, lb/d	4	8	4	9	4	9
-- Effluent NH3-N, mg/L (est)	1.0	1.0	1.0	1.0	1.0	1.0
-- NH3-N converted to NO3-N, lb/d	4	8	4	8	4	8
Effluent NO3-N as Operated, mg/L (est)	10	10	10	10	10	10
-- (Without denitrification, mg/L)	92	97	94	99	99	103
- NO3-N Denitrified, lb/d	4	10	5	10	5	10
o Oxygen required						
- VSS destruction						
-- lb O2/lb VSS destroyed	1.42	1.42	1.42	1.42	1.42	1.42
-- lb O2 used, lb/d	83	157	88	163	88	163
- Nitrification						
-- lb O2 used for nitrif, lb/d	19	37	20	39	20	39
- Denitrification						
-- lb O2 recovered, lb/d	-13	-28	-14	-29	-13	-29
- Total Oxygen used, lb/d	89	166	94	173	94	173
o Oxygen required during Air-On cycle						
-- lb/hr	6	10	6	11	6	11
-- lb/day (effective, while aerating)	133	249	141	260	141	260
o Aerator/Diffuser Requirements						
- See subsequent Section, below						
o Alkalinity Addition						
- Effluent Alk without Alk Addition	143	95	143	95	143	95

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Biotran-1602

	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
o Dewatering/Decant directly from Digester						
- Flow Rate, mgd	0.0023	0.0047	0.0022	0.0043	0.0019	0.0040
- Characteristics, mg/L						
-- BOD	9	18	10	19	10	19
-- TSS	150	150	150	150	150	150
-- VSS	102	105	103	106	103	106
-- NH3-N	1.0	1.0	1.0	1.0	1.0	1.0
-- Organic-N	10.2	10.8	10.2	10.9	10.2	10.9
-- NO3-N + NO2-N	10.0	10.0	10.0	10.0	10.0	10.0
-- Alkalinity	143	95	143	95	143	95
o Digested Sludge						
- Flow Rate, mgd	0.003	0.005	0.003	0.006	0.003	0.006
- Characteristics, mg/L						
-- BOD	633	1180	677	1248	677	1247
-- TSS	10,000	10,000	10,000	10,000	10,000	10,000
-- VSS	6,810	7,016	6,834	7,050	6,834	7,050
-- NH3-N	1	1	1	1	1	1
-- Organic-N	481	523	483	526	483	526
-- NO3-N + NO2-N	10	10	10	10	10	10
-- Alkalinity	143	95	143	95	143	95
o Digested Sludge Routing	Hauling	Hauling	Hauling	Hauling	Hauling	Hauling
AERATION REQUIREMENTS FOR AEROBIC DIGESTER						
o Oxygen required during Aeration period						
- lb/day (effective, while aerating)	133	249	141	260	141	260
o Aeration Method						
- Surface Aeration (1) or Diffused Air (2)?	2	2	2	2	2	2
o Diffused Aeration - **Summary**	Membrn Tube	Membrn Tube	Membrn Tube	Membrn Tube	Membrn Tube	Membrn Tube
- Air Supply while operating, scfm	140	290	150	300	150	310
- Mixing Air, scfm/sf	0.18	0.18	0.18	0.18	0.18	0.18
- Mixing Air, scfm	124	124	124	124	124	124
- Controlling Air Rate, scfm	140	290	150	300	150	310
- Aeration capacity, installed, scfm	180	380	200	390	200	400
- Power consumption while operating, hp	10	10	10	10	10	10
- Average hp/day	7	7	7	7	7	7
- Installed hp, total	10	20	10	20	10	20
o Aeration Analysis						
- Basic design information						
-- Design Water Temperature, C	30	30	30	30	30	30
-- *Theta* factor	1.024	1.024	1.024	1.024	1.024	1.024
-- Temp. correction, *Tau*	0.83	0.83	0.83	0.83	0.83	0.83
-- Site Elevation above MSL, ft	423	423	423	423	423	423
-- ..Pressure correction, *Omega*	0.98	0.98	0.98	0.98	0.98	0.98
-- *Beta* factor	0.99	0.99	0.99	0.99	0.99	0.99

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Biotran-1602		Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd		0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd		0.228	0.228	0.250	0.250	0.250	0.250
o Diffused Aeration							
o Oxygen Transfer Efficiency							
- Diffuser Type		Membrn Tube	Membrn Tube	Membrn Tube	Membrn Tube	Membrn Tube	Membrn Tube
- Operating Average DO, mg/L		2.0	2.0	2.0	2.0	2.0	2.0
- Operating depth for this calculation, ft		6.5	6.5	6.5	6.5	6.5	6.5
- Diffuser submergence, ft		6.0	6.0	6.0	6.0	6.0	6.0
- Air loading, scfm/unit	[Note]	4.83	10.00	5.17	10.34	5.17	10.69
		scfm/sf	scfm/sf	scfm/sf	scfm/sf	scfm/sf	scfm/sf
- Floor Coverage							
- Clean Water SOTE		12.3	12.5	12.1	12.7	12.1	12.8
- Site Conditions Adjustment Factor							
F = Actual / Standard OTE							
-- Alpha factor, including fouling		0.42	0.35	0.42	0.34	0.42	0.34
-- Equilibrium C*20		9.66	9.66	9.66	9.66	9.66	9.66
..Depth Adjustment Factor		0.37	0.37	0.37	0.37	0.37	0.37
- F = Alpha x [*Theta* ^(T-20)]		0.32	0.27	0.32	0.26	0.32	0.26
x (*Tau Beta Omega* C*20 - C)/C*20							
- Oxygen Transfer Efficiency		3.89	3.36	3.83	3.34	3.83	3.38
OTE = F x SOTE		Percent	Percent	Percent	Percent	Percent	Percent
Preliminary Estimate							
o SOTR Required							
- Average Day @ Design flow							
-- Actual Ox Tr Requd, AOTR, lb/d		133	249	141	260	141	260
-- Site Conditions Adjustment, F		0.32	0.27	0.32	0.26	0.32	0.26
-- Standard Ox Tr Rate, SOTR, lb/d		420	926	446	987	445	987
SOTR = AOTR / F							
o Air Supply Required							
- Average Day @ Design flow							
-- Ox Transfer Rate, AOTR, lb/d		133	249	141	260	141	260
-- Oxygen Supplied, lb/min		2.4	5.1	2.6	5.4	2.6	5.3
-- cf Air/lb Oxygen		57.0	57.0	57.0	57.0	57.0	57.0
[23.3 lb O2/100 lb Air]							
[0.0753 lb Air/scf]							
-- Total Blower Air while Operating, scfm		140	290	150	310	150	300
- Peak Day @ Design Flow							
-- Peaking factor		1.3	1.3	1.3	1.3	1.3	1.3
-- Total Blower Air provided, scfm		180	380	200	400	200	390
o Diffusers							
- Expressed as active sq ft or # diffusers		sq ft	sq ft	sq ft	sq ft	sq ft	sq ft
- Actual Installed, total for all basins		29	29	29	29	29	29
- Air Loading, scfm/sf or dfr							
-- Daily Average while operating		4.83	10.00	5.17	10.69	5.17	10.34

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Biotran-1602	Original Design Strength	High Strength	Original Design Strength	High Strength	Original Design Strength	High Strength
Annual Average Plant Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
Design (Max-Month) Flow, mgd	0.228	0.228	0.250	0.250	0.250	0.250
- Floor Coverage						
-- Total Floor Area in Service, sf	692	692	692	692	692	692
-- Coverage	4.2	4.2	4.2	4.2	4.2	4.2
.. Expressed as	%Actv A	%Actv A	%Actv A	%Actv A	%Actv A	%Actv A
- Active sf/diffuser, or 1	2.54	2.54	2.54	2.54	2.54	2.54
- Number of diffuser units	11	11	11	11	11	11
o Blower Discharge pressure						
- Head, ft water						
-- Peak Submergence (tank full)	9.3	9.3	9.3	9.3	9.3	9.3
-- Diffuser head loss	2.0	2.0	2.0	2.0	2.0	2.0
-- Pipe & Valve friction	2.5	2.5	2.5	2.5	2.5	2.5
-- Total Head, ft	13.75	13.75	13.75	13.75	13.75	13.75
- Discharge pressure, psig	6.0	6.0	6.0	6.0	6.0	6.0
o Delivered Horsepower while Operating						
- Max Operating Air Temp, C	35	35	35	35	35	35
- Barometric Pressure, psia	14.5	14.5	14.5	14.5	14.5	14.5
- Blower Suction Pressure, psia	14.2	14.2	14.2	14.2	14.2	14.2
- Daily Average Total Air, scfm	140	290	150	310	150	300
- Avg Delivered Horsepower, hp	4	8	4	8	4	8
- Peak Day Delivered hp	5	10	5	10	5	10
o Wire power required						
- Energy Efficiency, %	61.0	61.0	61.0	61.0	61.0	61.0
- Wire power required, hp						
-- Daily Average	10	10	10	10	10	10
-- Firm Installed	10	20	10	20	10	20