WASTEWATER SYSTEM MASTER PLAN UPDATE

for

Big Sky County Water and Sewer District 363

July 2015

DOWL

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LIST OF ABBREVIATIONS AND ACRONYMS

µg/L	Microgram per Liter
ABNR	Algae Based Nutrient Removal
AOB	Ammonia Oxidizing Bacteria
ARM	Administrative Rules of Montana
BOD_5	Biochemical Oxygen Demand
EPDM	Ethylene Propylene Diene Monomer
EQ	Equation
gpcpd	Gallons per Capita per Day
GPD	Gallons Per Day
GPY	Gallons per Year
HP	Horsepower
I/I	Infiltration and Inflow
IPR	Indirect Potable Reuse
LMM	Lone Moose Meadows Development
MBMG	Montana Bureau of Mines and Geology
MCA	Montana Code Annotated
MCRT	Mean Cell Residence Time
MDEQ	Montana Department of Environmental Quality
MG	Million Gallons
MGD	Million Gallons per Day
MGY	Million Gallons per Year
NOB	Nitrite Oxidizing Bacteria
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
SBR	Sequencing Batch Reactor
SFE	Single Family Equivalent
SP	Spanish Peaks Development
TBD	To Be Determined
TDS	Total Dissolved Solids
TN	Total Nitrogen
TSS	Total Suspended Solids
USDW	Underground Source of Drinking Water
WWTP	Wastewater Treatment Plant

1.0 EXECUTIVE SUMMARY

1.1 INTRODUCTION

The Big Sky Water and Sewer District (District) retained DOWL to prepare a Wastewater Master Plan that would update the Long Term Compliance Work Plan prepared in 1999 and the Amendment to the Plan prepared in 2001. This plan is focused on improvements to the wastewater plant and an evaluation of storage and disposal alternatives for the projected full-buildout of the District. This plan does not consider potential expansions to the District.

1.2 FUTURE CAPACITY REQUIREMENTS

As of August 2014, the District has issued permits for 4,686 Single Family Equivalents (SFE's) and has a commitment to provide wastewater service for a total of 7,926.3 SFEs in the District plus an additional 80.86 million gallons per year (MGY) of wastewater flow to Spanish Peaks and Lone Moose Meadows. The current flow per SFE is approximately 26,500 gallons per year per SFE but with increasing occupancy rates the flow per SFE is expected to increase to 29,387 gallons per SFE per year. Since 2009, the annual flow to the District's wastewater plant has average 116.7 MGY with a low of 105.9 MGY in 2010 and a high of 132.6 MGY in 2014. However, discrepancies between the plant meters cast doubt on the true plant flow. The treatment plant has several flow meters at various key locations in the plant and there are large discrepancies in the flow balance between the meters. Based on total flows from January 1, 2012 through December 31, 2014 the flow recorded at the treatment plant totaled 376.6 million gallons, the flow to the filters totaled 298.3 million gallons, while the flow pumped to the Meadow Village and YC golf courses totaled 484.5 million gallons. While some of the error could be attributed to differences in the storage pond levels on January 1, 2013 and December 31, 2014, the error is thought to be small in relationship to the 107.9 MG difference between the treatment plant flow and the pumped irrigation volume. Additional investigation will be needed to determine if the cause of the inaccurate flow balance is caused by improper calibration, improper installation, stray electrical currents, partial pipe flow, or some other cause. The projected annual flow at full buildout is 313.8 MGY which includes 14.1 million gallons of infiltration flow during spring runoff conditions.

Since 1992, the annual growth rate in issued SFE's has averaged 4.09 percent and the median rate has been 3.01 percent. Three annual growth rates have been projected in this study based on the expected variation in the growth rate over the next 20-30 years; a low rate of 3.36%, an average rate of 4.09% and a high rate of 4.82%.

1.3 WASTEWATER PLANT

The existing wastewater treatment plant is a two basin Sequencing Batch Reactor (SBR) designed to reduce Total Nitrogen (TN) to 10 mg/L-N, in addition to secondary treatment levels for Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) of 20 mg/L. The plant operates on batch cycles and was originally designed based on a 4.8 hour cycle time per basin for a total of 10 cycles per day. The plant cycle time has been increased to a 6 hour cycle for a total of 8 cycles per day which helps in reducing the nitrogen in the effluent but also reduces the peak day capacity of the plant. With the current cycle time, the peak day plant capacity is 1.04 MGD. At full buildout the peak day is expected to be approximately 2.2 MGD.

The treatment plant produces a high quality effluent and meets the design goals with the exception of the effluent total nitrogen in 2013 and 2014 and periodic exceedances of the design BOD_5 and TSS values in the SBR effluent. Total nitrogen levels increased in 2013 and 2014, but it is suspected the increase was due to the failure of the fine bubble diffusers in the SBR basins. The diffusers were replaced in the fall of 2014. The diffusers have a life expectancy of five to eight years and had been in service for approximately ten years. The occasional high BOD_5 and TSS values are most likely due to a bulking sludge condition when the sludge does not settle properly. Bulking sludge is a common occurrence in activated sludge plants but can be controlled through the addition of chlorine or polymers to the SBR basins to improve the settling characteristics.

Aeration in the SBR is provided by four 100 HP constant-speed blowers. The blowers can be controlled either based on the dissolved oxygen level in the basin or on a timed basis. The District has recently purchased new dissolved oxygen probes to improve the accuracy of the dissolved oxygen reading but because the blowers are constant-speed the dissolved oxygen levels will often exceed the set point. It is recommended that two of the blowers be converted to variable frequency drives (VFD) which will require purchasing new VFD drives and motors rated for VFD operation. In addition to the new DO probes, the District purchased Oxidation-Reduction Probes (ORP) that will provide information on when nitrification and denitrification process is complete. The purchase of ammonia probes is also recommended to help the operators control the nitrification process. Unless the DO, ORP, and ammonia probes are tied into the blower operation any changes will have to be made manually. Therefore, it is recommended that the District work with the SBR vendor, the controls integrator, and engineer to develop the control logic to maximize the effectiveness of the on-line probes.

Effluent from the SBR is pumped to the 8.2 million gallon SBR effluent storage pond. The existing piping configuration only allows the pond to be drawn down two to three feet which minimizes the effective storage and the pond remains nearly full all the time. Reconfiguring the piping at the outlet structure and installing new piping at a lower elevation will allow the pond to be emptied giving the District a true 8.2 MG of storage.

While not an immediate need, the District should begin planning, by 2018, for the addition of a third SBR basin in 2021. In 2014 the peak day flow at the plant was 0.923 MGD, but in most of the preceding years the peak day flow has been approximately 0.6 to 0.7 MGD. In the projected high growth rate scenario, the peak day flow would routinely be in the 1 to 1.2 MGD range starting in 2021. The plant is capable of treating flows over the rated peak day design capacity but the system will go into a "storm mode" where the cycles are shortened and treatment levels decrease.

1.4 EFFLUENT DISPOSAL AND STORAGE

Irrigation during the summer is the only disposal currently available which effectively sets the limit on future growth unless additional disposal means are developed. Prior studies appear to have over-estimated the irrigation capacity of the Yellowstone and Spanish Peak courses and to a lesser extent the Meadow Village course. Table 1-1 list the calculate capacities that were based on soil conductivities published by the Soil Conservation Service and the revised rates that are based on current irrigation practices. **The estimated current disposal capacity is 95.8 MGY to 131.8 MGY short of the projected full buildout flow of 313.8 MGY. At the average projected growth rate, the flow to the**

treatment plant will exceed the wet-weather disposal capacity of the golf courses around the year 2022. It is recommended that ring permeameter tests be conducted on the Yellowstone Club and Spanish Peaks courses to document the actual hydraulic conductivities of the soils. Three ring permeameter were conducted on the Meadow Village course in 1995. It is recommended that 3-4 additional tests be conducted to better quantify the irrigation capacity of the Meadow Village also.

Table 1-1
Initial and Revised Golf Course Capacities

	Prior Calculated Capacities	Revised Capacity
Meadow Village Course	206 MGY	140-160 MGY
Spanish Peaks Course	76.0 MGY	22-28
Yellowstone Club Course	100.8 MGY	20-30
Total	382.8 MGY	182-218

At the present time, 159.7 MG of storage capacity is constructed and Spanish Peaks is in the process of designing additional storage that is expected to add 11 MG for a total of 170.7 MG. Even assuming that summer irrigation capacity would be available, at full buildout a total of 227.4 MG of storage will be required unless effluent can be discharged during the winter either in a subsurface discharge or to a surface stream. Table 1-2 shows the location and volume of existing effluent storage.

Table 1-2
Existing Storage

Storage Location	Volume Million Gallons
Meadow Village	
Storage Pond 1	60.1
Storage Pond 3	19.6
Subtotal	**79.7**
Yellowstone Mountain Club	80.0
Spanish Peaks (in design)	11.0
Total Storage	**170.7**

1.4.1 Meadow Village Golf Course

Irrigation on the Meadow Village golf course is the primary means of effluent disposal for the District. The District and Boyne are in the process of negotiating improvements to the course irrigation system to improve the tracking of irrigation flows and automation of the system. In the spring of 2015, the District replaced the booster pumps in the station on the Meadow Village golf course and is making improvements to the controls in the main irrigation pump control panel located in the filtration building. Figure 1-1 shows the new booster pumps which are now in operation. The current golf course has 176.5

Figure 1-1 New Golf Course Booster

acres under irrigation but Boyne has mentioned the possibility of expanding the current golf course into Tract B which, if completely irrigated would add approximately 86 acres of irrigated lands to the system.

While the land application to the Meadow Village golf course is applied at less than the agronomic uptake rate, monitoring completed by the Blue Water Task Force has shown evidence of anthropogenic sources of nitrogen in groundwater wells on the golf course and in stream samples as the Middle Fork passes through the golf course area. A draft report prepared by the Blue Water Task Force, in 2014 identified several nitrogen reduction strategies one of which was to improve the control of runoff water from the golf course and impervious areas, during storm events and snowmelt, by developing wetlands/swales for nutrient removal (1). Seven proposed site were identified in the golf course for the construction of runoff control with construction methods varying from simple grading and the addition of outlet control to installing a liner, outlet control and planting vegetation. The 2014 construction cost estimate was $390,000 (1). It is recommended that the District continue to explore alternatives in conjunction with the Blue Water Task Force and the golf course owners. Constructed wetlands have been shown to be effective for total nitrogen removal in cold climates but proper construction is important to eliminate freezing during cold weather.

1.4.2 Subsurface Disposal

A screening process based on land slopes, mapped landslide area, wetlands, surface streams, and distance from the effluent discharge line was used to identify potential sites where subsurface irrigation could be utilized to discharge effluent during the winter. A total of 54 sites were evaluated of which 18 were judged as potentially suitable. It is estimated that an area of 20 to 25 acres (including buffers) would be required to dispose of 100,000 gpd. The 18 sites identified have a total land area of 721.3 acres. A more detailed site investigation involving percolation tests and nondegradation evaluations will have to be completed to further refine the site(s) suitability and cost to construct multiple drainfield sites. However, an "order of magnitude" cost to construct a drainfield is $10.85 per gallon per day excluding conveyance costs and any land purchase costs.

1.4.3 Surface Discharge

The District has previously obtained a discharge permit to discharge treated effluent to the Gallatin River but the permit was never used and the pipeline required to discharge was never constructed. The permit has expired and was not renewed by the District. Montana has recently enacted numeric nutrient limits but those limits only apply from July 1 to September 30th of each year and consequently would not apply to a winter discharge. The discharge would still have to comply with the nondegradation requirements of ARM 7.30.705 but only the narrative standards apply in regards to nutrients which require that the discharge will not create a nuisance or have a demonstrable impact on the water quality. A prior study (2) estimated 0.891 MGD could be discharge to the Gallatin River under the nondegradation rules. That estimate still appears to be valid although further discussion with the Montana Department of Environmental Quality (MDEQ) would be required to confirm the estimate. There is a lack of data on many of the constituents regulated under ARM 7.30.705 and additional testing would be required to ensure they would not substantially reduce the allowable discharge. The application for a discharge permit requires the applicants with a discharge greater than 1 MGD submit effluent testing data on 126 Priority Pollutants that are regulated under the Clean Water Act and whole effluent toxicity testing. While a discharge from the District would not exceed 1 MGD, it is recommended the District begin the testing to support a discharge permit application if the District selects the surface discharge alternative. It is also recommended that the District initiate discussion with MDEQ to determine what if any additional testing would be required to demonstrate to MDEQ that a discharge would meet the nondegradation criteria.

While expanding the District to include the lower canyon area was not part of this study, the inclusion of the area into the District would provide the opportunity to apply for nutrient credits for removing the existing drainfields in the lower canyon area. Under the State's nutrient trading policy, when drainfields are removed from service a portion of the nutrient load they were discharging could be transferred to the District.

1.4.4 Snowmaking

Snowmaking has been considered in the past as a promising alternative and it continues to be a viable alternative for disposal of a portion of the winter flows. With the numeric nutrient limits only applying from July 1 to September 30th of each year, runoff during snowmelt conditions is less of a concern. It is unlikely that snowmaking can be used to make up the 95.8 MG to 131.8 MG difference between the projected full buildout flows and the current irrigation capacity but it could be used to supplement the existing snowmaking facilities with potential expansions into non-skiing terrain. A previous snowmaking pilot test was approved by MDEQ and conducted by the District and Yellowstone Club. If snowmaking is to be incorporated into the disposal options it is recommended that potential snowmaking sites be evaluated for a larger "proof of concept" test. In concept, the site would be located near existing snowmaking equipment but several hundred feet from a surface stream and would allow large piles of snow (20-30 feet) in height. In this concept the natural snow will melt first with the large piles melting slowly over the summer with enough distance between the pile and the surface stream to allow infiltration of the meltwater. At an average pile height of 20 feet, a 4 acre site would be needed to dispose of approximately 11 MG.

1.4.5 Algae Based Nutrient Removal (ABNR)

This alternative would replace the existing filtration system with a system that uses algae for additional nutrient removal and incorporates a microfiltration system to separate the algae from the discharge stream. The process is an emerging technology with a few small plants located throughout the US. The test results provided by the vendor have shown promise as a cost-effective treatment alternative. The company headquarters is in Missoula where they have a pilot plant in operation. In order to test the process the vendor has recommended a nutrient recovery test be completed which would consist of collecting a volume of water from the existing SBR and running it through their pilot plant in Missoula.

1.5 RECOMMENDATIONS

The following recommendations include improvements at the treatment plant and additional evaluations that will be needed to determine the best disposal options both in terms of environmental impacts and costs. Table 1-3 summarizes the recommended improvements along with the estimated capital costs. The improvements are not prioritized in this study but will be reviewed and prioritized with the District staff and Board during the preparation of the Capital Improvement Plan.

As mentioned above, effluent disposal on the current golf courses does not meet the needs for future development. Of the disposal alternatives evaluated, a surface discharge to the Gallatin River during the winter months is the best suited to meet the needs of the District and arguably has the least environmental impacts to surface streams in the area. It is recommended that the District begin testing the effluent for the Priority Pollutants and Whole Effluent Toxicity Tests.

Table 1-3
Summary of Recommended Improvements and Additional Studies

Recommended Improvements	Estimated Cost
Wastewater Treatment Plant	
SBR, digester blower and control upgrades	$295,300
Headworks odor control	$42,000
Enclose grit removal	$50,000
Replace plant water pumps	$7,200
Reconfigure SBR Storage Pond Piping	$187,000
Evaluate alternatives to cover the SBR basins	TBD (not scoped at this time)
Effluent Disposal	
Ring permeameter tests on golf courses	$15,000
Priority Pollutant testing –once per year	$1,500/year
Whole Effluent Toxicity Tests – once per year	$3,250/year

The District should continue working with the Blue Water Task Force and Boyne to evaluate the best locations to construct runoff control measures on and around the golf course.

If the District is interested in evaluating the ABNR process in more detail, the Nutrient Recovery Test should be completed as well as refining a process layout with a detailed cost estimate.

2.0 BASIS OF PLANNING

2.1 PLANNING AREA

The planning area consists of the current Water and Sewer District Boundaries as shown in Figure 2-1. The District encompasses approximately 6,285 acres with properties in both Madison and Gallatin Counties. Development within the District is typical of a resort community and consists of single family residences, condominiums and townhouses, hotels, and commercial centers. There is no industrial development within the District. The population of the area changes throughout the year, increasing in the winter and on holidays, and decreasing in the "shoulder seasons" when the ski area is closed and the golf courses have not yet opened in the summer and in the late fall as the summer season winds down.

2.2 EXISTING AND PROJECTED FLOWS

2.2.1 Historical Wastewater Treatment Flows

Monthly wastewater flow records from January 1993 to July 2014 have been used to estimate future flows into the system. Table 2-1 presents the historical wastewater flow data and Figure 2-2 shows the total annual flows and single family equivalents (SFEs) per year.

Table 2-1
Wastewater Flows

Values are in units of Million Gallons (MG)														
Year	**Jan**	**Feb**	**Mar**	**April**	**May**	**Jun**	**Jul**	**Aug**	**Sept**	**Oct**	**Nov**	**Dec**	**Yearly Total (MG)**	**Average Month (MG)**
1993	7.4	7.3	8.8	6.9	16.0	14.6	14.3	12.2	12.4	5.9	4.2	7.1	117.1	9.8
1994	7.6	7.2	8.9	8.5	6.0	6.2	8.0	7.8	5.5	3.7	3.7	7.5	80.6	6.7
1995	8.1	7.8	9.0	9.1	15.9	12.3	9.0	5.0	3.2	2.4	2.8	5.7	90.4	7.5
1996	5.9	6.0	7.4	8.8	11.7	9.1	5.3	4.9	4.6	4.1	3.8	7.8	79.3	6.6
1997	8.5	7.9	10.2	9.3	19.4	14.5	9.7	8.0	5.6	4.1	4.5	8.6	110.1	9.2
1998	9.0	8.3	10.1	9.0	11.2	11.8	9.8	7.8	5.9	4.7	4.5	8.1	100.1	8.3
1999	8.2	8.2	10.1	7.2	7.2	7.6	7.7	7.6	5.8	4.2	3.9	7.2	85.0	7.1
2000	8.4	8.1	10.0	8.7	7.2	6.2	7.6	7.2	5.2	4.5	4.5	7.8	85.3	7.1
2001	8.9	9.0	10.6	7.6	5.5	6.3	7.8	7.5	5.6	4.4	4.4	7.8	85.5	7.1
2002	8.6	8.8	10.5	7.2	6.6	7.6	8.4	7.6	5.8	4.1	4.0	7.5	86.8	7.2
2003	8.5	8.6	10.5	8.3	8.4	6.6	7.6	7.6	5.3	4.3	3.9	7.7	87.4	7.3
2004	9.0	8.6	10.0	6.3	4.3	6.1	7.5	7.1	6.2	4.4	4.8	7.9	82.2	6.9
2005	8.6	8.4	10.1	6.1	5.5	8.3	7.7	7.7	7.4	5.8	7.5	13.0	96.0	8.0
2006	15.0	11.7	13.5	14.6	9.8	7.7	7.8	9.3	5.8	6.5	6.4	11.3	119.4	10.0
2007	12.5	11.2	14.2	9.8	6.6	7.5	8.8	9.0	7.7	7.0	6.4	10.4	111.2	9.3
2008	11.8	11.0	12.0	6.9	15.6	14.2	11.5	8.4	6.5	5.2	5.4	10.6	119.1	9.9
2009	11.8	11.4	12.4	10.4	13.2	8.9	8.8	7.4	5.2	4.7	4.8	9.3	108.1	9.0
2010	10.1	10.2	11.0	9.5	8.6	9.4	9.7	8.9	7.1	4.8	5.9	10.7	105.9	8.8
2011	11.6	11.1	12.8	9.4	20.5	14.8	11.9	9.2	7.0	6.6	6.7	10.3	131.8	11.0
2012	10.8	10.1	12.9	11.0	6.7	7.3	10.3	9.8	7.8	5.4	5.6	11.0	108.6	9.1
2013	11.0	10.6	12.2	9.2	8.6	8.8	11.0	10.3	7.8	5.8	6.1	12.0	113.5	9.5
2014	14.8	13.8	15.4	13.4	14.8	10.0	10.2	9.7	6.67	5.71	6.77	11.35	132.6	11.05

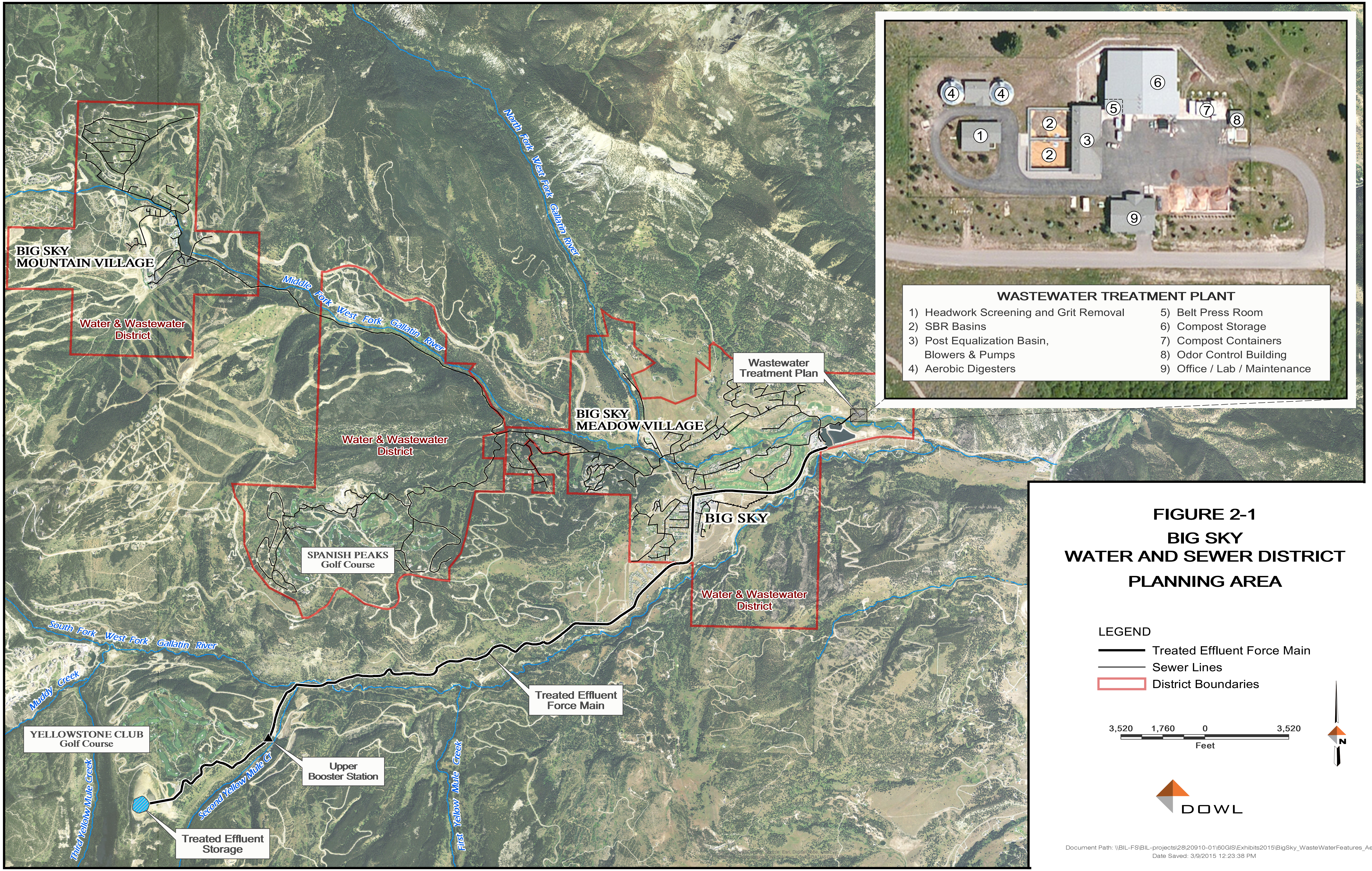

FIGURE 2-1
BIG SKY WATER AND SEWER DISTRICT PLANNING AREA

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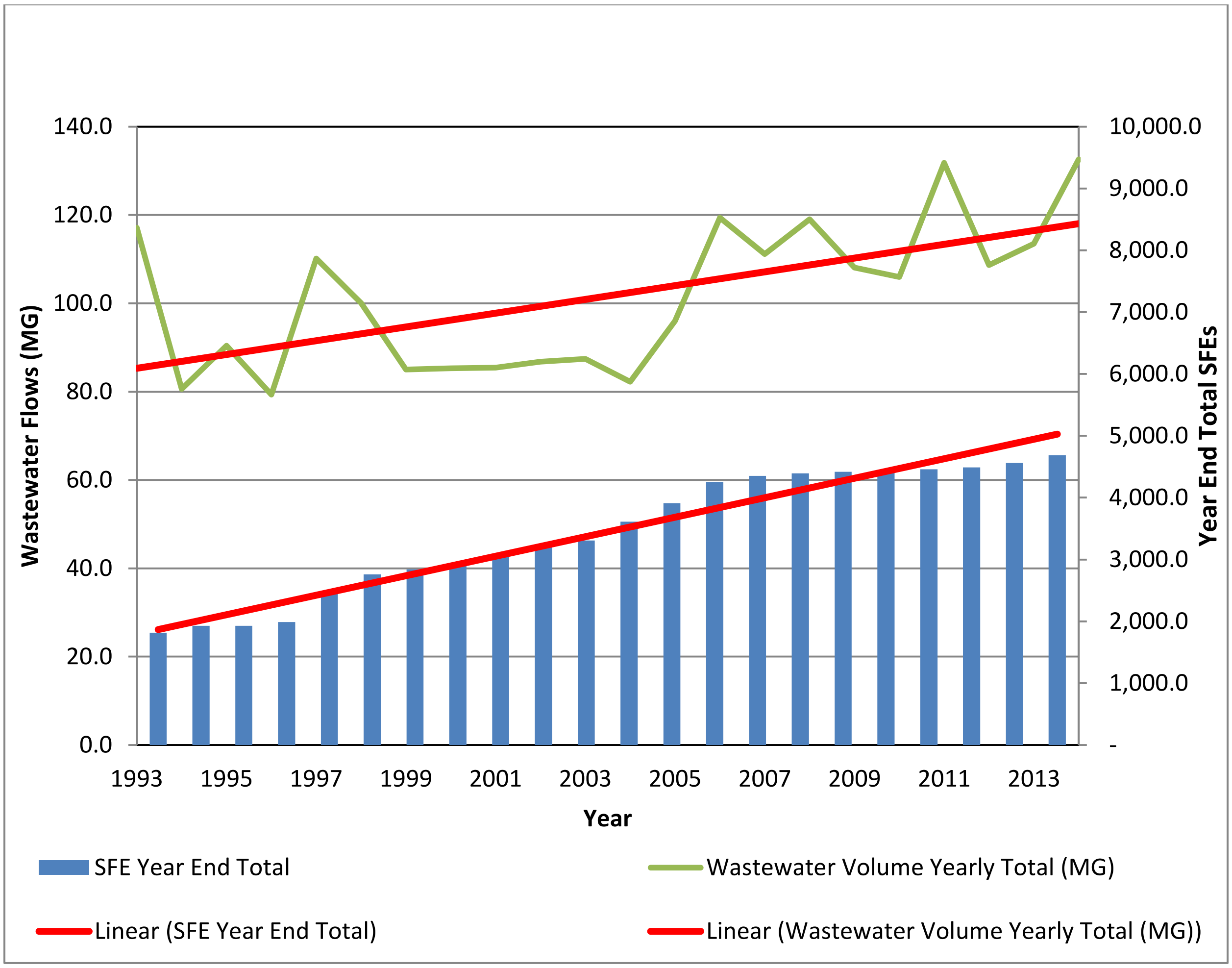
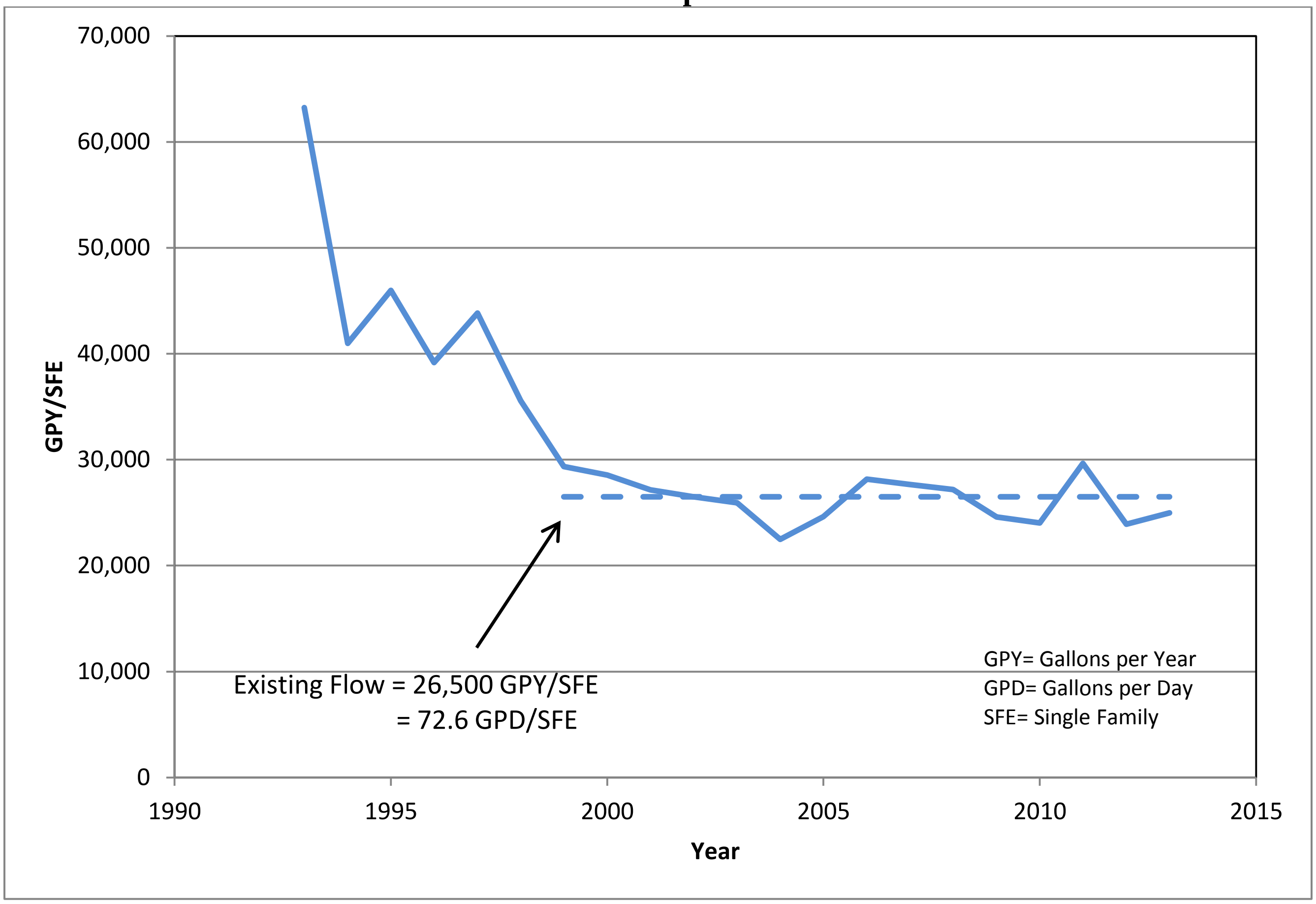
The total volume of wastewater has fluctuated each year dependent upon the amount of inflow and infiltration (I/I) entering the collection system, development growth, and the number of skier days. Following collection system improvements in 1998, the total wastewater volume decreased significantly when compared to wastewater volumes in 1997 and 1998. While the collection system improvements appear to have decreased the I/I entering the system from 1998 to 2004, heavy runoff in 2006 and 2011 caused a considerable increase in volume from previous years, indicating some deficiencies still remain in the collection system. As expected, the total volume of wastewater increases with increasing numbers of Single Family Equivalents (SFEs); however, as presented in Figure 2-2, the various peaks resulting from I/I obscure the year-to-year increase.

Figure 2-2
Wastewater Volume and Total SFEs



The flow rate per SFE from 1993 to 2013 shows a decreasing trend presumably due to collection system improvements and water saving technologies, such as low-flow toilets and showerheads, in new construction. Figure 2-3 presents the graph of the flow per SFE from 1993 to 2013.

Figure 2-3
Flow Rate per SFE



2.2.2 Equivalent Population, Gallons per Capita per Day, and I/I Estimates

The equivalent population served by the wastewater system was estimated based on the organic loading measured at the treatment plant and applying typical per person criteria. Organic loading, measured in terms of the five day Biochemical Oxygen Demand (BOD_5), is typically estimate at 0.2 pounds BOD_5 per capita per day (3). The value of 0.2 pounds BOD_5 per capita per day was used to convert the BOD_5 loading into an equivalent population (EQ 1). The monthly flow rate was then divided by the equivalent population to calculate gallons per capita per day (gpcpd) values (EQ 2). Table 2-2 presents the equivalent population estimates for 2004-2013.

$$Capita\ (population) = \frac{0.2\frac{lbs}{Capita*day}}{\left[BOD_5\left(\frac{mg}{l}\right)\times Flow\ (MGD)\times 8.34\ (conversion)\right]\ \left(\frac{lbs}{day}\right)} \qquad \textbf{(EQ 1)}$$

$$\text{Gallons per Capita per Day} = gpcpd = \frac{Flow\ (GPD)}{Capita} \qquad \textbf{(EQ 2)}$$

Table 2-2
Equivalent Population Estimates From BOD_5 (2004-2013)

	2004 (capita)	2005 (capita)	2006 (capita)	2007 (capita)	2008 (capita)	2009 (capita)	2010 (capita)	2011 (capita)	2012 (capita)	2013 (capita)	Monthly Median (capita)
January	2,587	3,176	6,072	3,960	5,712	6,486	4,020	7,666	2,758	3,844	3,990
February	3,434	4,500	6,426	5,436	5,095	6,606	7,527	11,238	4,677	5,446	5,441
March	3,677	4,619	4,877	4,629	4,333	5,350	4,297	4,623	4,555	5,444	4,621
April	1,047	1,364	935	2,181	4,297	2,034	1,314	2,223	3,120	2,499	2,107
May	5,540	1,113	784	1,204	1,481	1,119	1,329	1,860	1,803	2,073	1,405
June	1,519	1,964	1,708	2,703	3,858	1,846	1,433	1,457	3,101	2,582	1,905
July	4,554	2,325	2,631	2,676	3,466	2,136	2,730	6,564	3,895	3,483	3,098
August	1,394	2,122	3,631	3,264	2,700	1,745	2,520	1,724	7,387	2,912	2,610
September	2,537	2,023	1,932	3,264	4,657	1,579	1,835	6,399	6,140	6,420	2,901
October	1,240	1,436	2,092	1,406	1,957	1,922	1,223	7,304	6,950	4,514	1,940
November	1,714	3,090	5,269	1,326	5,479	1,628	1,750	1,961	2,498	6,330	2,230
December	3,973	5,543	6,903	6,686	5,702	11,690	13,002	5,216	5,469	11,444	6,686

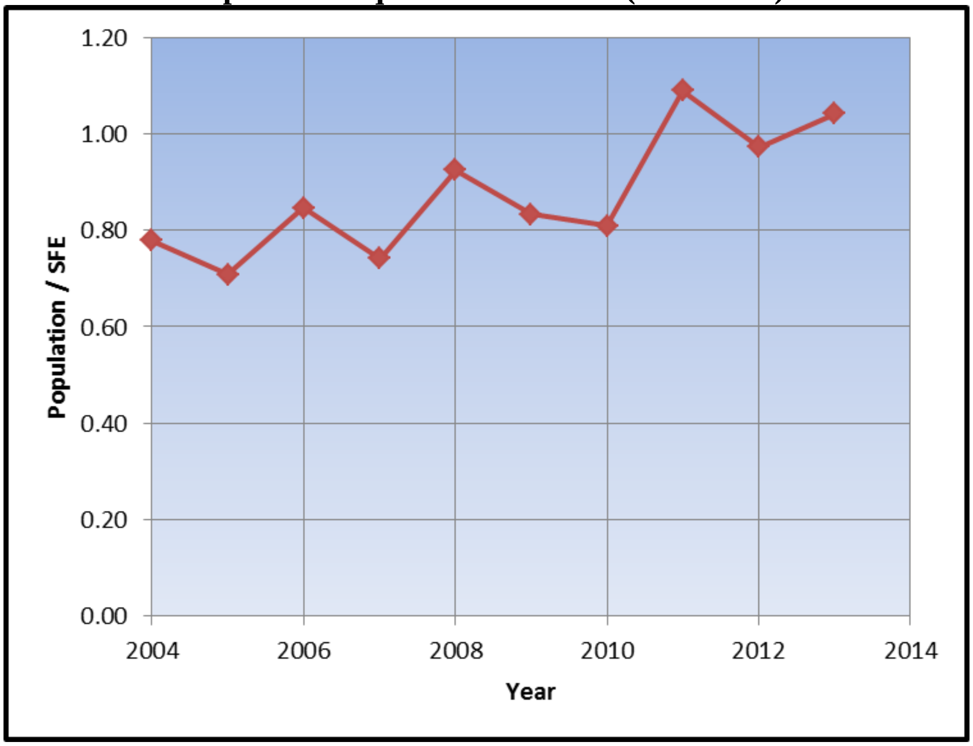
Table 2-3 presents the flow per person for 2004-2013. On an annual basis, the per capita flow averages 104 gallons per capita per day. During the "dry months" of January through March and July through December, the median per capita flow is 86 gallons per day.

Table 2-3
Gallons per Capita per Day for 2004-2013

	2004 (gpcpd)	2005 (gpcpd)	2006 (gpcpd)	2007 (gpcpd)	2008 (gpcpd)	2009 (gpcpd)	2010 (gpcpd)	2011 (gpcpd)	2012 (gpcpd)	2013 (gpcpd)	Monthly Median 2004-2013 (gpcpd)	Notes
January	110	88	80	108	67	71	82	49	128	92	82	
February	83	67	80	74	77	61	57	35	77	70	70	
March	92	73	92	102	92	77	91	92	94	75	92	
April	184	150	521	150	60	224	244	143	118	124	150	High I&I Months
May	62	160	311	184	345	177	213	370	120	133	180	
June	138	141	155	92	127	160	218	339	79	115	140	
July	53	111	96	107	107	150	114	58	86	102	107	
August	167	89	83	89	100	138	124	171	43	124	100	
September	82	124	102	96	48	109	134	36	49	41	96	
October	121	138	100	35	86	84	127	31	25	41	84	
November	109	81	41	166	33	99	117	160	75	33	81	
December	75	78	55	70	71	64	32	58	67	36	64	

As illustrated in Figure 2-4, the estimated population equivalent per SFE is trending to higher occupancy rates per SFE. The estimated population density per SFE was 0.78 in 2004 and increased to 1.04 persons per SFE in 2013.

Figure 2-4
Population Equivalent Per SFE (2004-2013)



2.2.3 Inflow and Infiltration (I/I)

Infiltration and Inflow (I/I) makes up a portion of the current flow and fluctuates year-by-year. Infiltration is water that enters a sewer system from the ground through defective pipes, pipe joints, damaged lateral connections, or at manhole walls. Infiltration is most often related to high groundwater conditions but can also be influenced by storm events. Inflow is stormwater that enters the sanitary sewer system through roof drains, foundation drains, sump pumps, and cross connections with storm sewers. Inflow is usually short in duration, lasting primarily during the storm events or for a few hours after the storm event ends. The exception is inflow from sump pumps located in high groundwater areas that pump nearly continuously into the sanitary collection system.

In 1993, I/I constituted over half of the flow measured at the lagoons. In order to reduce the I/I volume, the Water and Sewer District instituted an aggressive repair program. During 1993 and 1994, approximately 28,300 feet of sewer line was inspected with a television camera and repairs were made at many locations where infiltration was occurring. While the springtime flow decreased substantially in 1994, it returned to the historic high levels again during 1995. The low I/I flows observed in the spring of 1994 may have been the result of the low snowpack and short runoff period rather than improvements in the collection system. Flows in 1996 were generally lower than those in 1995 but high springtime flows again occurred during April through June. The sewer outfall line from the Mountain Village to the Meadow Village was replaced in 1998 and flows in the following years were down.

The Environmental Protection Agency (EPA), through the Code of Federal Regulations (CFR 35.2120), considers infiltration excessive if the flow to the wastewater treatment plant is greater than 120 gallons per capita per day during high groundwater conditions. As shown in Table 2-3, during the spring runoff period the flows are considered excessive. During the "dry months" the median per capita flows is 86 gpcpd and is assumed to represent the baseline condition with minimal infiltration. The 86 gpcpd value corresponds well with the typical value of 82 gpcpd reported (4) for above average homes but is above the 69.3 gpcpd reported in the EPA in a summary of twelve studies (5).

In order to estimate the infiltration volumes during the "dry months" the flow at the wastewater treatment plant was compared to the volume of potable water sold through metered sales during the October through March time period. During the October through March period, it is assumed that all of the water used in a residence or business is discharged to the sewer system and is measured at the wastewater plant. Table 2-4 shows the differences in water sales and wastewater plant flows.

Table 2-4
Water Usage for 2009-2014

Date	Water Sales (Gallons)	WWTP Flows (Gallons)	Oct - March Difference
4th Quarter 2009	19,483,415	18,736,000	-747,415
1st Quarter 2010	27,432,400	31,380,000	3,947,600
4th Quarter 2010	16,711,414	21,341,000	4,629,586
1st Quarter 2011	29,113,576	35,520,000	6,406,424
4th Quarter 2011	16,112,463	23,607,000	7,494,537
1st Quarter 2012	26,271,319	33,770,000	7,498,681
4th Quarter 2012	17,572,521	22,014,000	4,441,479
1st Quarter 2013	28,086,011	33,830,000	5,743,989
4th Quarter 2013	18,196,292	23,938,000	5,741,708
1st Quarter 2014	30,921,048	44,020,000	13,098,952
		Median (Gal)	5,742,849
		Median Difference(GPD)	63,500

When the difference in flows is divided by the equivalent population numbers for each time period the result is an average volume of 11.9 gpcpd of baseline flow above the metered water sales. While a portion of the 63,500 gallons per day of additional flow may be infiltration, some of the excess flow may also be the result of condensate drains from high efficiency furnaces in newer homes, foundation drains, or sump pump connections. In this report, it is assumed that the 11.9 gpcpd is part of the base flow and is included in the design flows for future developments. It is noted that if the 11.9 gpcpd is subtracted from the 86 gpcpd calculated above, the "domestic flows" would closely match the EPA study.

Table 2-5 presents estimated I/I volumes for 2004-2013, which were calculated using the following equation:

I/I Volume = (monthly average gpcpd from Table 2-3–86.0 gpcpd)*(days/month)*(population from month of interest) = MG; <u>if (monthly average gpcpd from Table 2-3- 86 gpcpd)<0, then I/I = 0.0 MG</u>

Table 2-5
Estimates of Infiltration and Inflow

Values in Million Gallons											
	2004	2005	2006	2007	2008	2009	2010	2011	2012	2013	Average
January	1.9	0.2	0.0	2.7	0.0	0.0	0.0	0.0	3.6	0.7	0.9
February	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
March	0.7	0.0	0.9	2.3	0.8	0.0	0.7	0.9	1.1	0.0	0.8
April	3.2	2.7	12.6	4.3	0.0	8.7	6.4	3.9	3.1	2.9	4.8
May	0.0	2.5	5.5	3.7	11.9	3.2	5.2	16.4	1.9	3.0	5.3
June	2.5	3.4	3.7	0.5	4.9	4.2	5.9	11.4	0.0	2.3	3.9
July	0.0	1.8	0.8	1.7	2.3	4.2	2.4	0.0	0.0	1.8	1.5
August	3.5	0.2	0.0	0.3	1.2	2.8	3.0	4.6	0.0	3.5	1.9
September	0.0	2.4	0.9	1.0	0.0	1.1	2.8	0.0	0.0	0.0	0.8
October	1.3	2.3	0.9	0.0	0.0	0.0	1.5	0.0	0.0	0.0	0.6
November	1.2	0.0	0.0	3.3	0.0	0.7	1.7	4.5	0.0	0.0	1.1
December	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Totals	14.4	15.5	25.3	19.9	21.1	24.9	29.6	41.7	9.7	14.3	21.6

The estimated yearly I/I flows range from 9.7 MG in 2012 to 41.7 MG in 2011. We recommend the District continue to try and locate sources of infiltration; however, due to the uncertainty of the actual I/I reduction that can be achieved and the additional future growth (i.e. more locations for I/I), it is recommended that the historical average monthly I/I flow from April through June be included in the design flows. The average monthly I/I flow from April through June is 4.71 MG per month.

2.3 DESIGN FLOWS

2.3.1 Full Buildout Flows

Yearly flow projections are based on historical flows and the assumption that domestic flows will increase proportional to the growth rate and occupancy. This study is concerned with the full development in addition to the year-by-year development projections. The Big Sky Water and Sewer District is legally obligated to provide water and wastewater services for 7,926.3 SFEs as determined through the District's subdivision review process. As of August 2014, the District has issued permits for 4,686.0 SFEs, leaving 3,240.3 SFEs for future developments. In addition to the 7,926.3 SFE's allotted to subdivisions, the District has an obligation to provide wastewater service to Lone Moose Meadows (LMM) and Spanish Peaks (SP) through an agreement signed March 29, 2001 (6). The agreement specifies the "Developer's future right to the District's wastewater treatment facilities shall not exceed 1,900 SFE's or an equivalent annual flow in the amount of 80.86 million gallons per year". At the current flow of 26,500 GPY per SFE, the 1900 SFE's allotted to LMM and Spanish Peaks amounts to an annual flow of 50.3 MGY. However, in projecting full buildout flows it is have assumed that the Developer will be allowed additional SFE's until the flow from LMM and Spanish Peaks equals the 80.86 MG stipulated in the Agreement.

The following assumptions were used to project the annual flow at full buildout:

- All available SFEs will be claimed by future developments within the Big Sky Water and Sewer District
- Projected new District SFEs = 7,926.3 – 4,686.0 = 3,240.3

- The occupancy rate, defined as the number of people per SFE, will increase by 15 percent during the ski season (November through March) and by 20 percent during the remaining months as the resort markets itself as a year-round resort.
- The I/I flow of 4.71 MG per month in April, May, and June will remain
- The SFE's allotted to Spanish Peaks and LMM are increased until the annual flow from the developments equals 80.86 MGY. (80.86 MGY corresponds to 2,751.5 SFE at the projected flow per SFE of 29,387 gallons per year, which includes the flows attributed to the projected increased occupancy rate)
- Full build out SFE = 7,926.3 (District) + 2,751.5 (SP and LMM) = 10,677.8

Table 2-6 shows the estimated wastewater flows at full buildout. The population per SFE is calculated for each month based on the equivalent populations from Table 2-2 and the number of SFE's allocated at the end of each year. The median population per SFE is then calculated for each month of the year. The full buildout population per SFE is based on the assumed increase in occupancy rates noted above. A per capita flow of 86 gallon per day is used with an I/I flow of 4.71 MG added for the spring run-off months of April, May, and June. The resulting full buildout flow of 313.8 million gallons per year corresponds to a flow of 29,387 gallons per year per SFE.

Table 2-6
Projected Flow at Full Buildout of District

	Population per SFE											Full Buildout Flows			
	2005	2006	2007	2008	2009	2010	2011	2012	2013	Median Population per SFE	Full Buildout Population/ SFE	District Full Buildout Flow gallons/month	Lone Moose Meadows Commitment [1]	Cross Harbor Commitment [2]	TOTAL FULL BUILDOUT FLOW gallons
January	0.812	1.426	0.909	1.300	1.468	0.907	1.720	0.614	0.843	0.909	1.046	22,101,888	4,063,615	4,308,768	**30,474,270**
February	1.150	1.509	1.249	1.160	1.496	1.699	2.521	1.042	1.194	1.249	1.436	27,404,142	3,111,632	5,342,443	**35,858,218**
March	1.180	1.146	1.063	0.986	1.211	0.970	1.037	1.015	1.193	1.063	1.223	25,834,055	4,749,805	5,036,355	**35,620,214**
April	0.349	0.220	0.501	0.978	0.460	0.297	0.499	0.695	0.548	0.499	0.574	16,438,981	2,156,470	2,286,567	**20,882,019**
May	0.284	0.184	0.277	0.337	0.253	0.300	0.417	0.402	0.454	0.300	0.360	12,317,614	1,398,723	1,483,106	**15,199,443**
June	0.502	0.401	0.621	0.878	0.418	0.324	0.327	0.691	0.566	0.502	0.602	17,026,253	2,264,445	2,401,056	**21,691,755**
July	0.594	0.618	0.614	0.789	0.484	0.616	1.473	0.868	0.763	0.618	0.742	14,154,427	2,602,408	2,759,409	**19,516,244**
August	0.542	0.853	0.750	0.615	0.395	0.569	0.387	1.646	0.638	0.615	0.738	15,586,562	2,865,718	2,940,584	**21,392,864**
September	0.517	0.454		1.060	0.358	0.414	1.436	1.368	1.407	0.789	0.946	19,350,002	3,557,658	3,772,287	**26,679,947**
October	0.367	0.492		0.446	0.435	0.276	1.639	1.548	0.990	0.469	0.562	11,881,136	2,184,445	2,316,230	**16,381,811**
November	0.790	1.238	0.368	1.247	0.369	0.395	0.440	0.556	1.388	0.556	0.640	13,522,929	2,486,302	2,636,298	**18,645,529**
December	1.417	1.621	1.536	1.298	2.647	2.935	1.170	1.218	2.509	1.536	1.766	37,316,148	6,860,883	7,274,791	**51,451,822**
District SFE's at years end	3913.0	4257	4354	4393	4417	4430	4457	4489	4562			**232,934,137**	38,302,105	42,557,894	**313,794,136**

1. Lone Moose Meadows flows based ratio of 900/1900 * 80.86 MGY
2. Cross Harbor flows based on ratio of 1000/1900* 80.86 MGY

It is important to note that the District's full buildout SFE value (7923.6) is based on the number of sewer connections in subdivisions that have been approved by the Montana Department of Environmental Quality and where the District's sewage treatment system was relied upon for DEQ approval. If the District makes improvements that increase the District's treatment and disposal capacity above the level required to serve the existing obligations it is reasonable to expect additional SFE's could be added. The additional SFE's could result from higher density development within the District or expansion of the District to include outlying areas such as the lower canyon area.

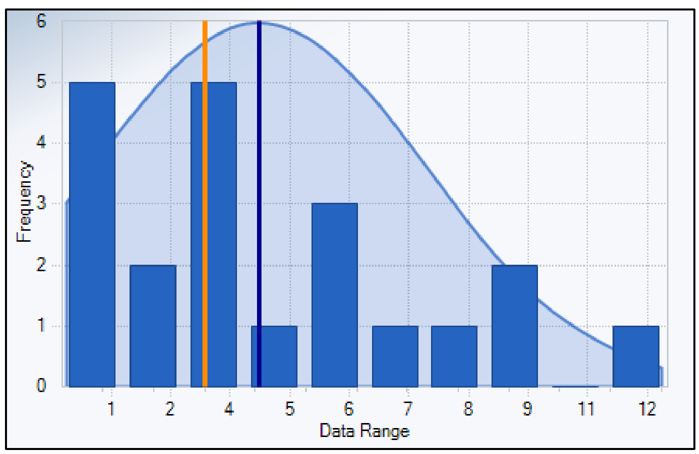
2.3.2 Projected Annual Growth Rate

To project the annual growth rate in SFE's, the historical rates were reviewed and projections were made for low, average, and high rates. Table 2-7 shows the historical growth rates from 1992 through 2014 and Figure 2-5 shows the frequency distribution. The zero percent in 1995 was the result of a Compliance Order issued by the Montana Department of Health and Environmental Sciences which restricted the issuing of permits to connect to the sewage system. The 24.05 percent increase in 1997 was the result of a change in the way SFE's were calculated. The increase in 1998 was largely due to the construction of the Summit Hotel. Excluding the years 1995 and 1997, the average annual growth rate has been 4.09 percent the median rate has been 3.01 percent.

Table 2-7
Historical Growth Rates

Year	Year End SFE Count	Growth Rate %	Year	Year End SFE Count	Growth Rate %
1992	1,730.0	4.98	2003	3,307.0	2.86
1993	1,817.0	5.03	2004	3,611.0	9.19
1994	1,929.0	6.16	2005	3,913.0	8.36
1995	1,929.0	0.00	2006	4,257.0	8.79
1996	1,987.1	3.01	2007	4,354.0	2.28
1997	2,465.0	24.05	2008	4,393.0	0.90
1998	2,761.0	12.01	2009	4,417.0	0.55
1999	2,841.0	2.90	2010	4,430.0	0.29
2000	2,932.0	3.20	2011	4,457.0	0.61
2001	3,089.0	5.35	2012	4,489.0	0.72
2002	3,215.0	4.08	2013	4,564.0	1.67
			2014	4,695.0	2.87

Figure 2-5
Annual Growth Rate Distribution



Growth rates are projected for three conditions based on the expected variation in the average SFE growth over the next 20-30 years 1) a low rate (3.36%) 2) the historical average growth rate of 4.09 percent and 3) a high growth rate (4.82%). In any one or two year period the growth rate can be expected to vary substantially from the average, but over the long term the growth rate is expected to be near the historical average.

Figure 2-6 illustrates the three growth scenarios and Figure 2-7 illustrates the projected annual flows corresponding to the growth rate scenarios. At the high growth rate, the full buildout flow will be reach around 2032 while at the low growth rate the full buildout flow will be reached around 2039. At the average growth rate the full build out flows will be reached in approximately 20 years.

Figure 2-6
Projected SFE Growth

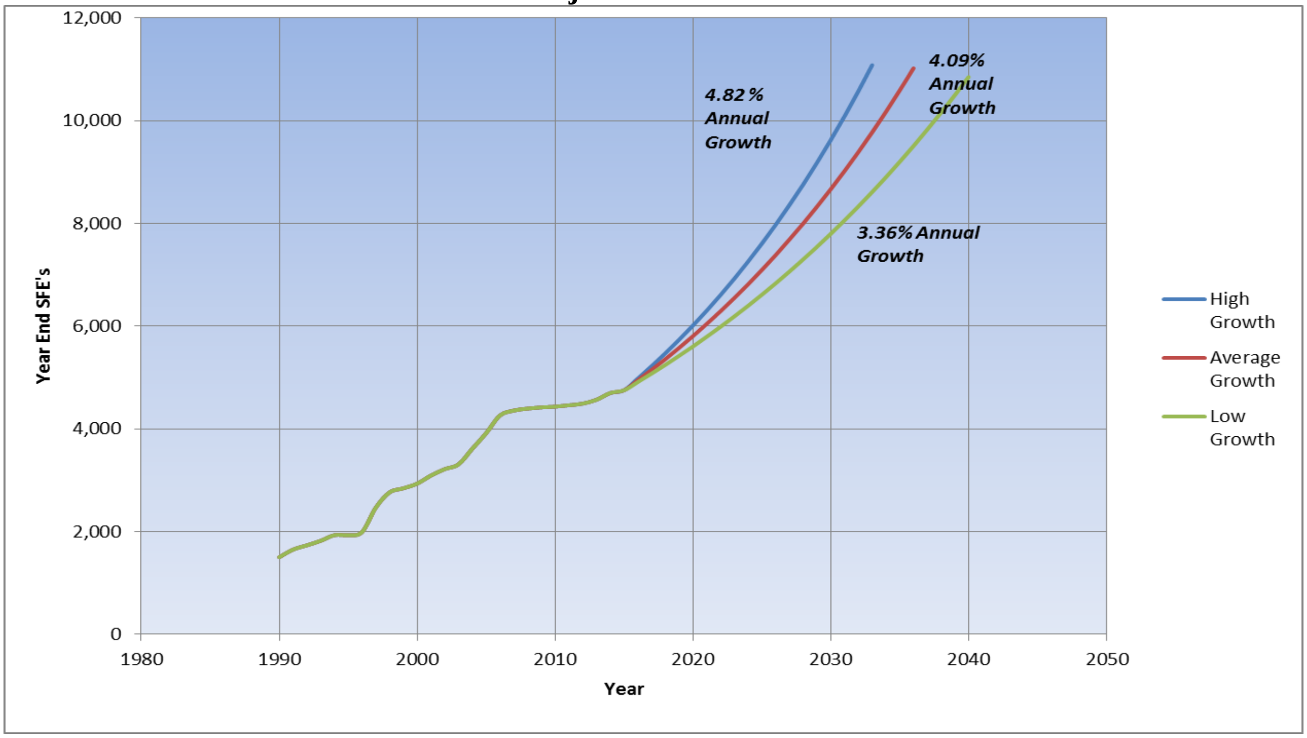
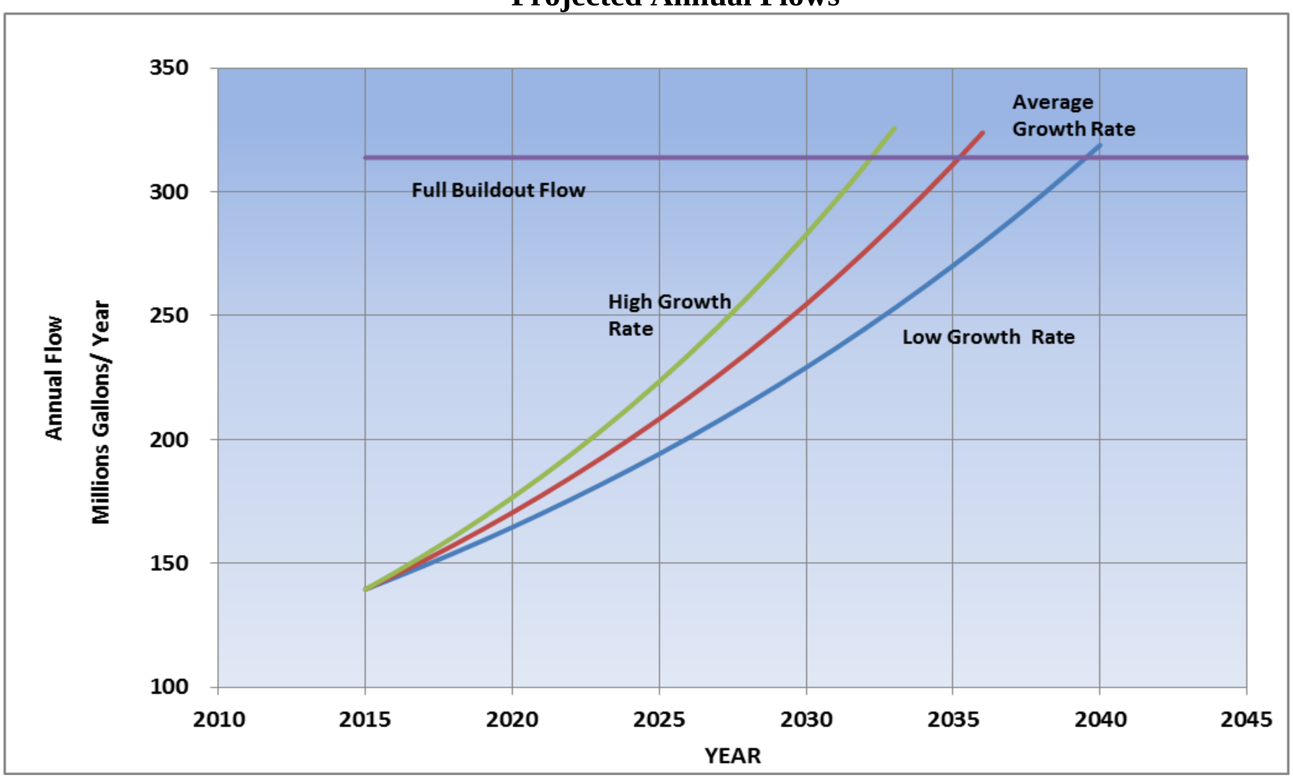


Figure 2-7
Projected Annual Flows



2.4 REGULATORY REQUIREMENTS

2.4.1 Treatment Standards and Water Classifications

Montana has defined several water classifications and treatment standards that govern the beneficial reuse of wastewater effluent (7). Table 2-8 lists the classifications pertinent to disposal of wastewater in Big Sky. Irrigation from the District's treatment process is in the Class A category. The primary difference between the Class A and Class A-1 water is related to the total nitrogen concentration. Class A-1 water must have a total nitrogen concentration of less than 5 mg/L-N whereas the limit for Class A water is not specified. The total nitrogen concentration limit on Class A-1 water allows irrigation to exceed the agronomic uptake rate of the crop and exempts any permitting requirement for a groundwater discharge permit.

Table 2-8
Water Classifications for Beneficial Reuse

	Water Classifications			
	A-1	**A**	**B-1**	**B**
BOD_5	< 10 mg/L	< 10 mg/L	Not Specified	Not Specified
TSS	<10 mg/l	<10 mg/l	Not Specified	Not Specified
Turbidity	< 2 NTU on average Does not exceed 5 NTU at any time	< 2 NTU on average Does not exceed 5 NTU at any time	Not Specified	Not Specified
Coagulation, flocculation, sedimentation, Filtration	Normally required to meet turbidity limits	Normally required to meet turbidity limits	oxidized and settled only	oxidized and settled only
Disinfection	< 2.2 total coliforms/ 100 ml (weekly limit), < 23 total coliforms/100ml in any sample	< 2.2 total coliforms/ 100 ml (weekly limit), < 23 total coliforms/100ml in any sample	< 2.2 total coliforms/ 100 ml (weekly limit), < 23 total coliforms/100ml in any sample	< 2.2 total coliforms/ 100 ml (weekly limit), < 23 total coliforms/100ml in any sample
Total Nitrogen	< 5.0 mg/L at all times	Not Specified	< 5.0 mg/L at all times	Not Specified
Monitoring Requirements	continous turbidity, weekly total coliform, bi-weekly total nitrogen, weekly disinfectant residual	continous turbidity, weekly total coliform, monthly total nitrogen, weekly disinfectant residual	weekly total coliforms, bi-weekly total nitrogen, weekly disinfectant residual	weekly total coliforms, monthly total nitrogen, weekly disinfectant residual
Land Application	Can exceed Agronomic uptake rate	Cannot exceed Agronomic uptake rate	Can exceed Agronomic uptake rate	Cannot exceed Agronomic uptake rate
Restricted or Unrestricted Access at irrigation site	Unrestricted	Unrestricted	Restricted	Restricted
Buffer Zone Requirement	None	None	50 feet	50 feet
Groundwater discharge permit	exempt from permit	permit required for >5,000 gpd discharge	exempt from permit	permit required for >5,000 gpd discharge

2.4.2 Land Application

Land application of treated wastewater is regulated based on the quality of the water and the use of the application site. At Big Sky, the effluent is classified as Class A and is suitable for use where public access is unrestricted. Application of effluent to the golf course cannot exceed the agronomic uptake rate of nitrogen unless the plant is upgraded to produce Class A-1 water. Even with an upgrade to Class A-1 water, if the percolating water is shown to be interconnected to a surface stream, the MDEQ would be required to treat the discharge as a surface water discharge with permit requirements.

2.4.3 Surface Water Discharge

A discharge to surface water will require a Montana Pollution Discharge Elimination system (MPDES) permit from the Montana Department of Environmental Quality. For a new discharge, changes in water quality at the boundary of a mixing zone must meet the nondegradation requirements with any change in water quality being considered nonsignificant as defined in the Administrative Rules (ARM 17.30.715) unless a change is water quality is authorized by the MDEQ. Degradation will only be allowed if criteria contain in ARM 17.30.707 and 75-5-303 MCA are met. Key criteria in the referenced regulations include:

- degradation is necessary because there are no economically, environmentally, and technologically feasible modifications to the proposed project that would result in no degradation;
- the proposed project will result in important economic or social development and that the benefit of the development exceeds the costs to society of allowing degradation of high-quality waters;
- existing and anticipated use of state waters will be fully protected; and
- the least degrading water quality protection practices determined by the department to be economically, environmentally, and technologically feasible will be fully implemented by the applicant prior to and during the proposed activity.

On February 13, 2014 the Montana Board of Environmental Quality adopted base numeric nutrient standards that set new limits for nitrogen and phosphorous discharged to surface waters at levels designed to protect the beneficial uses and prevent exceedances of other surface water quality standards. The Big Sky area is in the Middle Rockies ecoregion which has in-stream nutrient standards of 30 μg/L total phosphorus and 300 μg/L total nitrogen. However, the limits only apply from July 1 to September 30 each year and consequently would not apply to a winter discharge. Instead of the numeric nutrient standards, the nondegradation criteria and narrative standards contain in the Water Quality Classification Standards would apply to a winter discharge. The narrative standards have general prohibitions (ARM 17.30.637) which require a discharge to be free from any substance that will create an undesirable or nuisance condition. Narrative standards are applied when sufficient information does not yet exist to develop specific numeric standards.

The West Fork of the Gallatin River, the Middle Fork of the West Fork, and the South Fork of the West Fork are all currently classified as impaired with on-site treatment systems, site clearance, animal feeding operations, and alteration of stream-side vegetative cover from roads and bridges listed as the probable cause of the impairment. The main stem of the Gallatin River is classified as a category 1 water where all beneficial uses are fully supported.

2.4.4 Groundwater Discharge

Groundwater disposal systems with a capacity greater than 5,000 gpd are required to obtain a discharge permit per ARM 17.30.1023. However, public systems that discharge unrestricted reclaimed wastewater and have been reviewed and approved by MDEQ under ARM 17.38.101 are exempt from the permit requirements (ARM 17.30.1022(1)h unless, as noted above, the discharge is hydraulically connected to a surface stream.

3.0 EXISTING FACILITIES

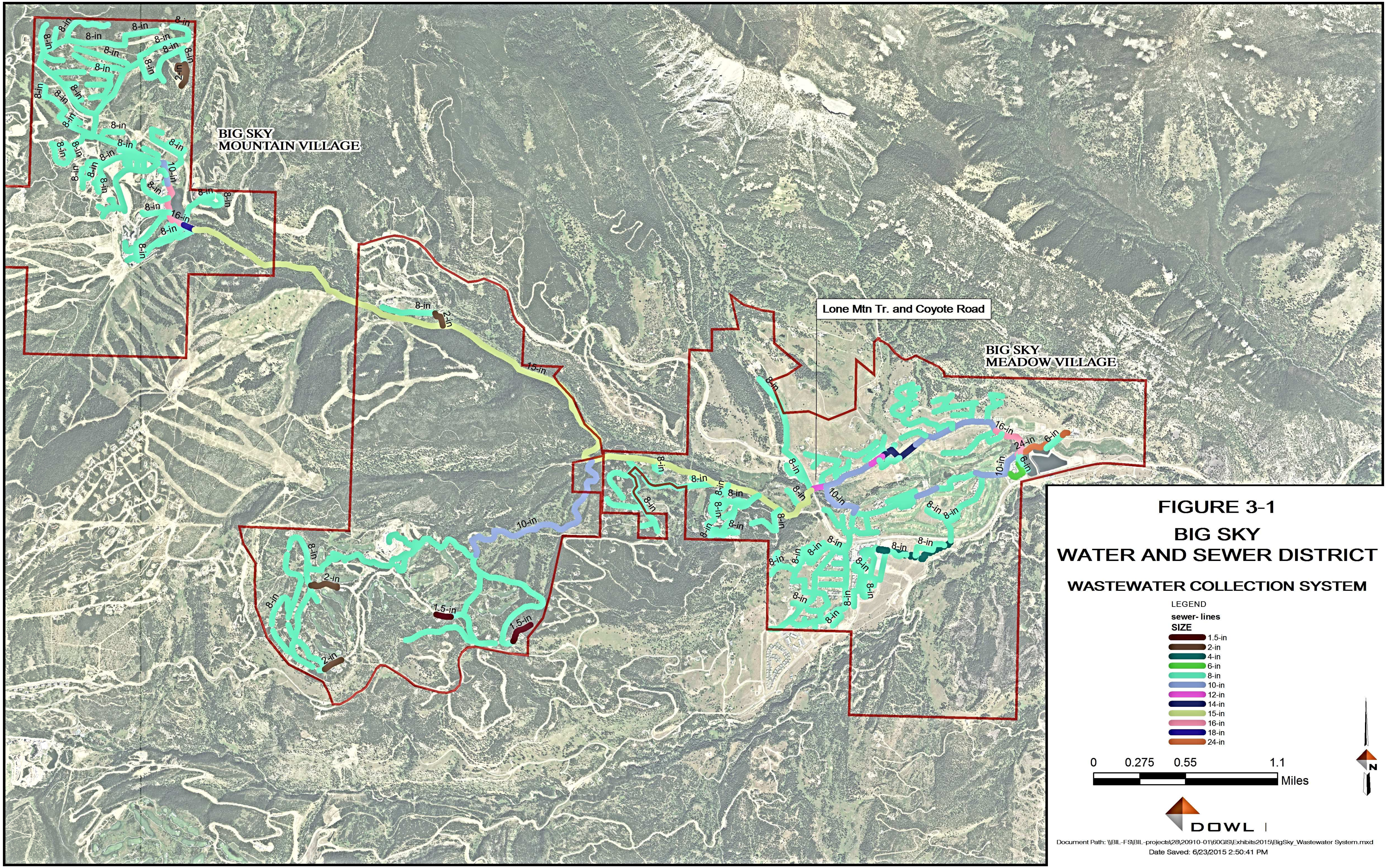
3.1 COLLECTION SYSTEM OVERVIEW

The existing collection system contains 211,094 feet of sewer line and 937 manholes. A depressed sewer (inverted siphon) consisting of two eight-inch and one six-inch HDPE lines convey wastewater under the Middle Fork. The sewer piping is made of polyvinyl chloride (PVC) pipe and was installed in the late 1960s and early 1970s when the resort was developed. High I&I events in the 1990s and 2000s led to the replacement of portions of the sewer system. There appears to be a choke point in the system where the 15-inch sewer line from the Mountain Village intersects Lone Mountain Trail at Little Coyote Road. At this point, the 15-inch constricts to a 12-inch and shortly thereafter decreases to a 10-inch. Downstream from this intersection the sewer line alternates between 10-inch, 12-inch, 14-inch, and 16-inch. Any of the locations where the diameter decreases has the potential to create a plugging point and back water conditions in the upstream pipe sections. Monitoring of these locations during high runoff events is recommended to determine if the choke points are causing the system to back up. Figure 3-1 shows the Meadow Village collection system and Table 3-1 presents a summary of the collection system's diameters and lengths.

Table 3-1
Wastewater Collection System Summary

Size (in)	Total Length (feet)	Inch-Diameter-Mile
1.5	1,284	0.4
2	3,612	1.4
4	924	0.7
6	1,368	1.6
8	189,112	286.5
10	16,251	30.8
12	911	2.1
14	1,373	3.6
15	24,178	68.7
16	3,015	9.1
18	388	1.3
24	1,187	5.4
Total	243,603	412

We recommend updating the GIS database with any information available, including: pipe material, pipe slope, age, and any other notes that may be of value. While no existing capacity issues are known, having pipe slopes in the database would allow the capacities of each pipe section to be determined.


BIG SKY
MOUNTAIN VILLAGE
Lone Mtn Tr. and Coyote Road
BIG SKY
MEADOW VILLAGE
FIGURE 3-1
BIG SKY
WATER AND SEWER DISTRICT
WASTEWATER COLLECTION SYSTEM
LEGEND
sewer- lines
SIZE
1.5-in
2-in
4-in
6-in
8-in
10-in
12-in
14-in
15-in
16-in
18-in
24-in
0
0.275
0.55
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3.2 WASTEWATER TREATMENT PLANT OVERVIEW

The existing wastewater treatment plant provides a tertiary level of treatment suitable for unrestricted irrigation. The plant consists of a Sequencing Batch Reactor (SBR) that provides secondary treatment and a tri-media filter providing tertiary treatment. Disinfection is accomplished using chlorine gas supplied in 150 pound cylinders. Irrigation at the Meadow Village golf course and the Yellowstone Club golf course are the only means of effluent disposal. Treated effluent is stored in lined storage ponds during the non-irrigation season. Biosolids generated in the treatment process are stabilized in an aerobic digester followed by an in-vessel composting system. Figure 3-2 is a schematic of the treatment process.

Figure 3-2
Process Flow Schematic

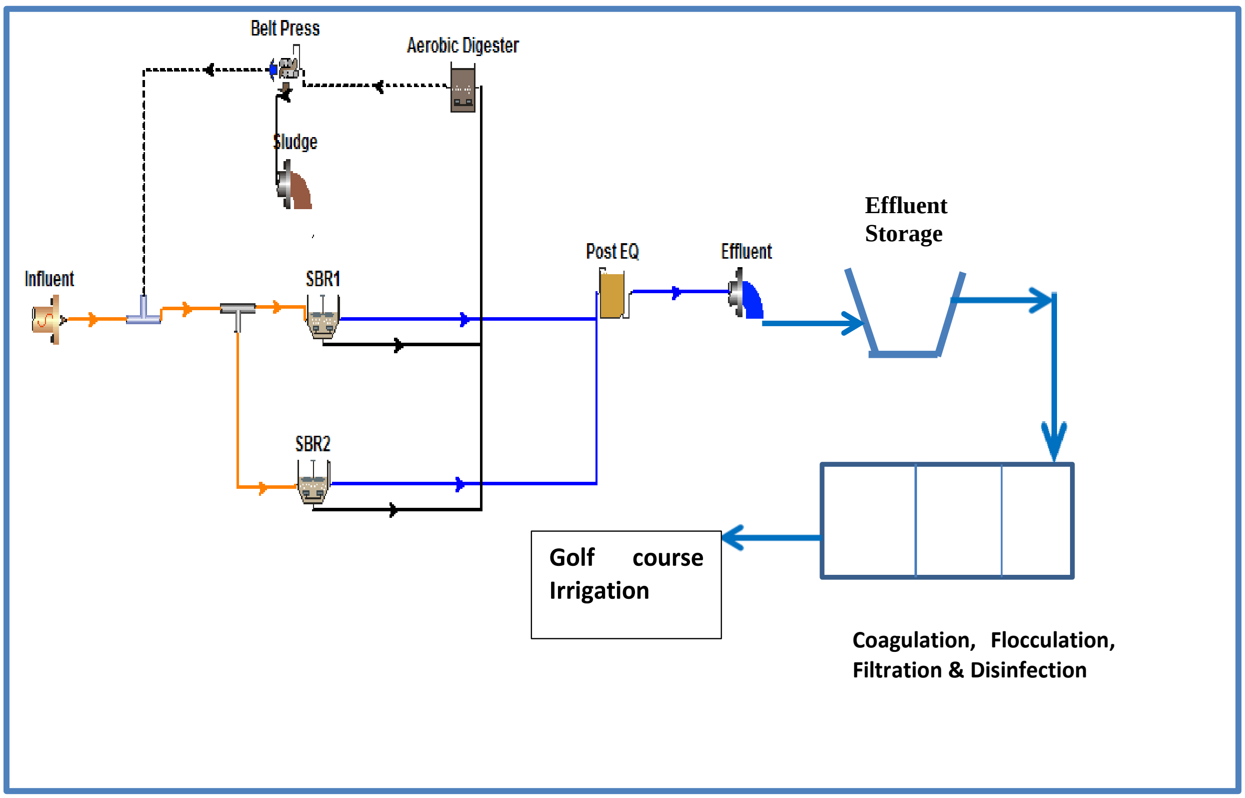


Table 3-2 lists the design criteria and equipment characteristics at the plant and operating data from 2014.

Table 3-3 lists the design effluent values.

Table 3-2
Existing Plant Design Characteristics

Design Year	2021	2014
Design Flows		
Peak Hour	1.89 MGD	
Peak Day	1.30 MGD	0.923 MGD
Average Day	0.65 MGD	0.363 MGD
BOD_5 Loading		
Peak Day	3,700 Pounds/day	
Average Day	1,294 Pounds/day	811 Pounds/day*
TSS Loading		
Peak Day	4,623 Pounds/day	
Average Day	1,368 Pounds/day	955 Pounds/day*
TKN Loading		
Peak Day	570 Pounds/day	
Average Day	190 Pounds/day	141 Pounds/day*
SBR Basins		
Basins	2	
Volume/Basin	466,000 gallon	
Cycles/Day/Basin	5	4
SBR Aeration		
Type	Fine Bubble	
Blowers	4 at 100 HP each	
SCFM/Blower	904 SCFM	
Aerobic Digesters		
Tanks	2	
Volume/Basin	133,000 gallons	
Aeration	Coarse Bubble	
Blowers	3 at 50 HP each	
SCFM/ Blower	446	
Tertiary Filtration		
Treatment trains	3	
Net Filtration Capacity/ train	0.23 MGD at 2.5 gpm/square foot (0.69 MGD total capacity)	
Coagulation &Flocculation	Horizontal Paddle Wheel	
Sedimentation	60 degree tube settlers	
Filtration	Tri-media	

* Based on median values

Table 3-3
Design Effluent Values

BOD_5	20 mg/L
TSS	20 mg/L
Total Nitrogen as N at average day	10
Fecal Coliforms	< 2.2/100mls

Table 3-4 summarizes the SBR influent and effluent data from 2010 through 2014. There are several abnormally high influent test results that are suspected to have been influenced by the aerobic digester supernatant or septage dumping that enters the plant upstream of the influent composite sampler.

Table 3-4
SBR Influent, Effluent, Filter, and Irrigation Water Data
(all data in mg/L)

	2010	2011	2012	2013	2014
Influent					
BOD $_5$					
Average	279	454	421	446	331
Median	210	325	310	270	270
Peak Day	1500	3300	1900	3000	1200
Total Suspended Solids					
Average	508	716	514	724	391
Median	264	396	295	362	318
Peak Day	3150	8200	2360	8980	1200
Total Kjeldahl Nitrogen					
Average	42.9	55	56.3	62.3	50
Median	39	56	52.5	51.5	47
Peak Day	113	192	166	236	153
SBR Effluent					
BOD $_5$					
Average	19	23	20	14	18
Median	14	6	10.5	10	13.5
Peak Day	68	170	120	100	93
Total Suspended Solids					
Average	21	23	21	8	17
Median	14	< 10	<10	<10	11
Peak Day	126	263	184	101	124
Total Nitrogen					
Average	8.61	9.1	8.8	13.4	16.8
Median	4.1	4.5	6.2	6.25	13
Peak Day	43	36	32	42	45
Total Kjeldahl Nitrogen					
Average	6.22	8.29	7.04	13.23	15.00
Median	2.30	3.30	3.05	4.90	10.85
Peak Day	43.00	36.00	32.00	42.00	45.40
Ammonia					
Average	8	5.7	4.1	10.6	12.3
Median	1.4	0.6	0.3	2.5	7.5
Peak Day	33	32	24.8	35	39
Total Phosphorous					
Average	1.27	1.69	2.39	1.71	1.52
Median	0.53	1.15	2.62	1.56	1.21
Peak Day	5.3	11.6	8.28	6.48	5.22
Filter Effluent					
Total Nitrogen_ Average	11.7	9.2	8.4	17.6	19.2
Total Kjeldahl Nitrogen_Average	10.7	8.7	8.3	17.5	18.6
Nitrate + Nitrite_Average	1.0	0.5	0.2	0.1	0.2
Total Phosphorous_Average	0.4	1.2	0.9	1.2	0.9
Irrigation Water					
Total Nitrogen_ Average	9.36	7.66	8.48	15.23	13.83
Total Kjeldahl Nitrogen_Average	9.04	6.52	8.21	14.86	13.55
Nitrate + Nitrite_Average	0.35	1.14	0.42	0.37	0.29

A statistical evaluation was completed for both the influent loading and the plant performance based on data from 2008 through 2014. Figure 3-3 shows the influent BOD_5 distributions and Figure 3-4 through Figure 3-6 show the treatment performance of the plant. The distributions of the 50.0th, 91.7th and 98.1th percentiles correspond to the estimated median, maximum month and maximum week concentrations.

Figure 3-3
Influent BOD_5 Concentrations

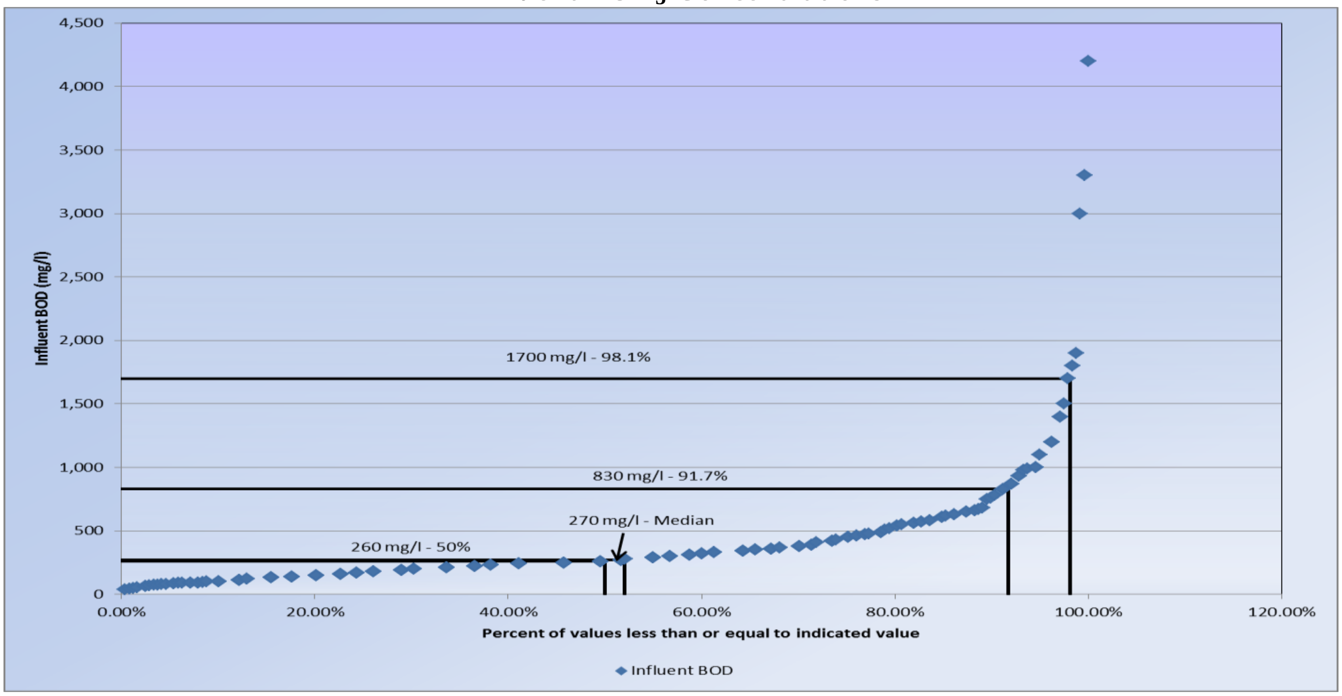


Figure 3-4
Effluent BOD_5 Distribution

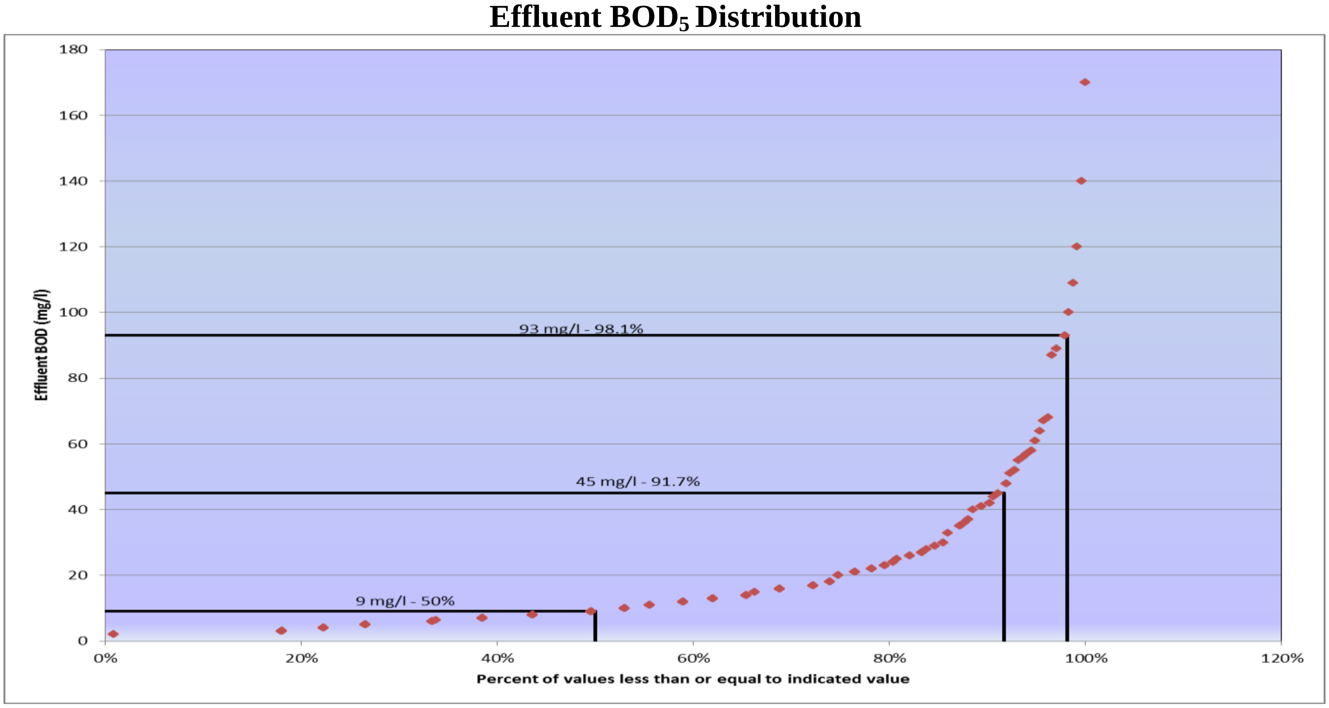


Figure 3-5
Effluent Total Nitrogen Distribution

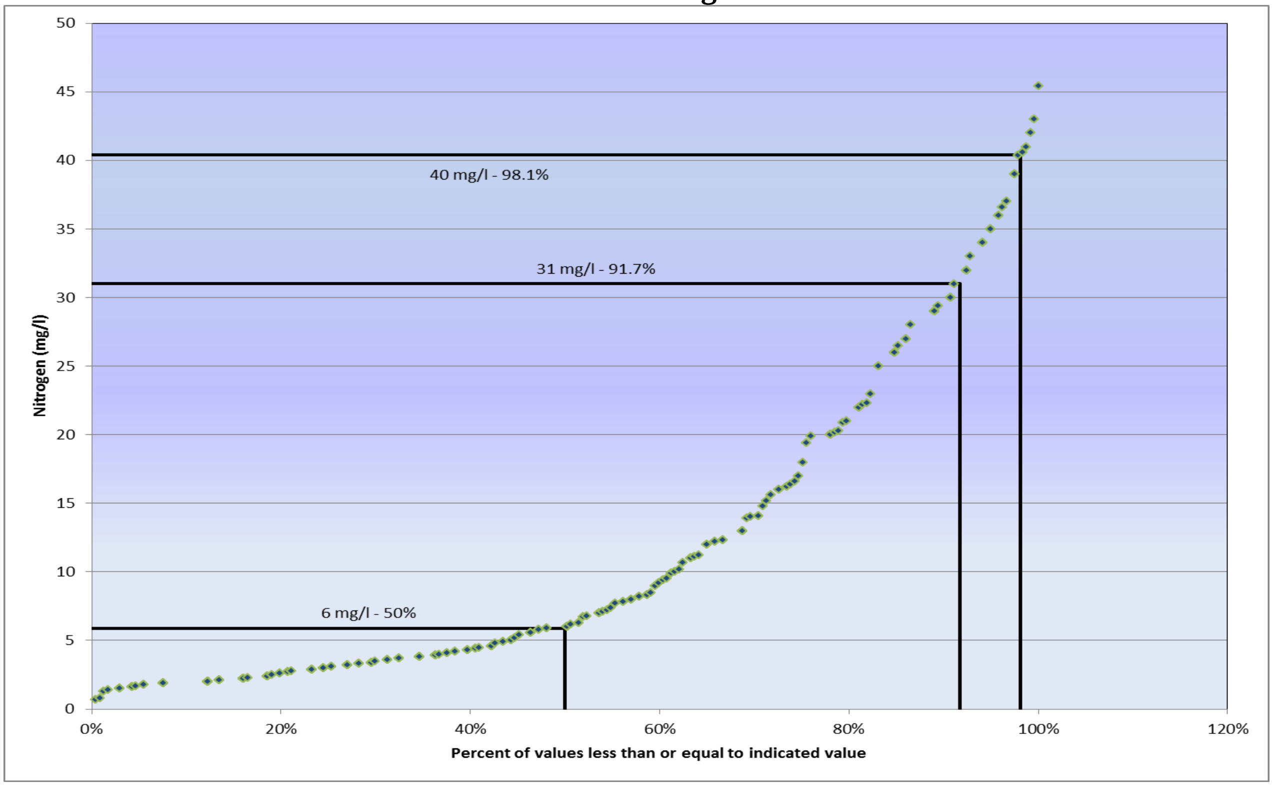
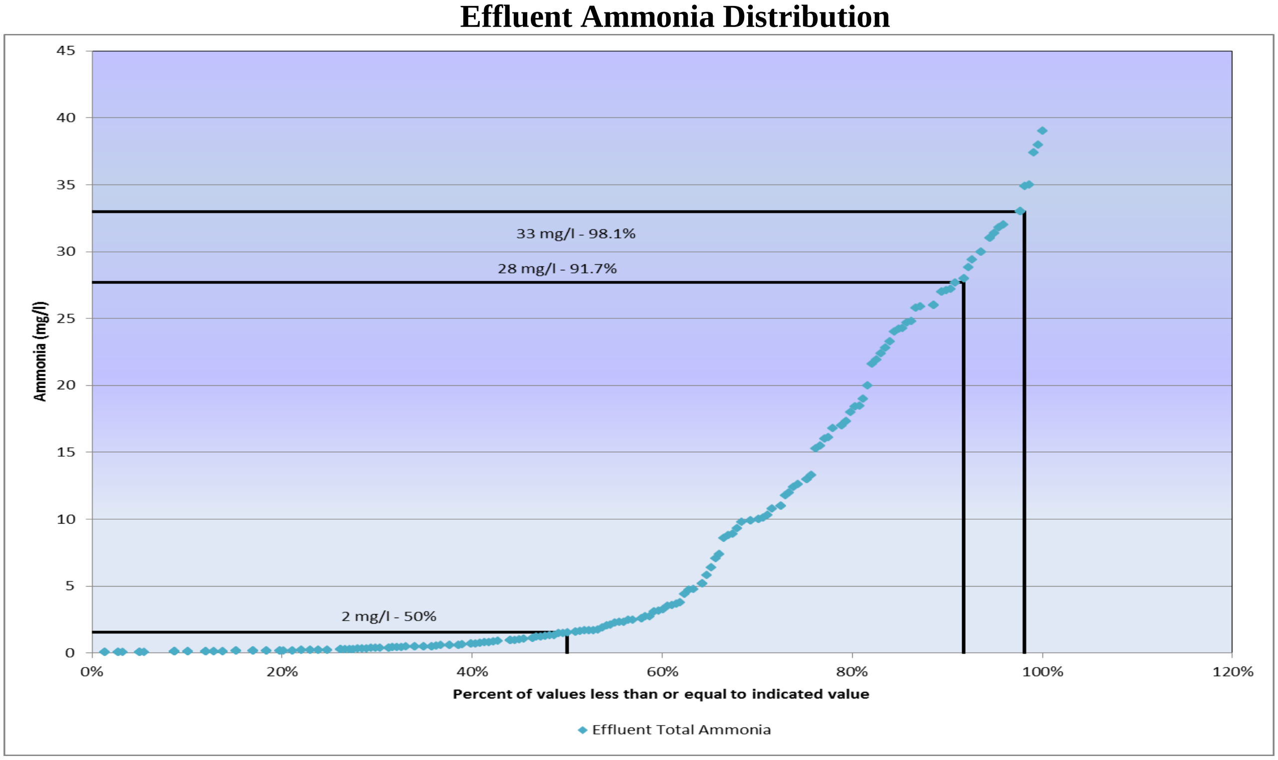


Figure 3-6
Effluent Ammonia Distribution



3.2.1 Unit Process Review

3.2.1.1 Headworks Screening and Grit Removal

The headworks screening consists of two mechanical fine screens each of which has a rated capacity of 1.87 MGD. The screens have ¼-inch diameter holes to strain material from the waste stream. Both screens are connected to the emergency generator. A dewatering and bagging system is used collect screened material. A screen washing system washes organic matter from the screened material prior to the bagging system. The washing system was designed to use either plant water or potable water but currently only uses potable water due to clogging issues with the plant water pumps.

The grit removal process uses a vortex style process with a self-priming vacuum pump used to pump grit from the bottom of the grit tank to a grit classifier. The grit tank and pump are located under a shed roof structure but are exposed to freezing temperatures which has caused problems with the pump. The District has enclosed the pump and motor in a shed but enclosing the entire grit assembly is recommended to ensure reliability and allow maintenance during cold weather conditions. The District is currently working with a contractor to enclose the grit system. Based quotes received by the District the estimated cost to enclose the grit removal process is $50,000

The headworks building has a high potential for odor due to the raw sewage in open channels, screening, and grit systems. It is recommended that an odor control system be installed inside the screening room to reduce odors. Three types of odor control systems are commonly used. One type consists of pumping air through a granular activated carbon bed , one process uses photoionization with ultraviolet light, and the third process uses an electrical discharge (corona discharge) created by ionization of the gas surrounding a conductor. The Corona discharge system can treat and recirculate the ambient air in the room and consequently does not required a high exchange rate of inside air with outside air. The Corona discharge system has an estimated installed cost of $42,000 and is the recommended odor control system.

3.2.1.2 Sequencing Batch Reactors

The two SBR basins were designed to operate on a total cycle time of 4.8 hours each resulting in 5 cycles per day per basin (10 total cycles/day). Table 3-5 shows the design cycle structure and times.

Table 3-5
Design SBR Cycle Structure

Total cycle time	=	4.8 hours	React	=	0.58 hours
Mixed fill	=	0.67 hours	Settling time	=	0.75 hours
React fill	=	1.74 hours	Decant time	=	1.07 hours

At the maximum fill level, the decant volume in each basin is 130,000 gallons. With a total of 10 cycles per day the design maximum day flow is 1.3 MGD.

The basins are currently operated on a 6 hour cycle time to improve nitrogen removal. The current cycle structure is shown in Table 3-6

Table 3-6
Current SBR Cycle Structure

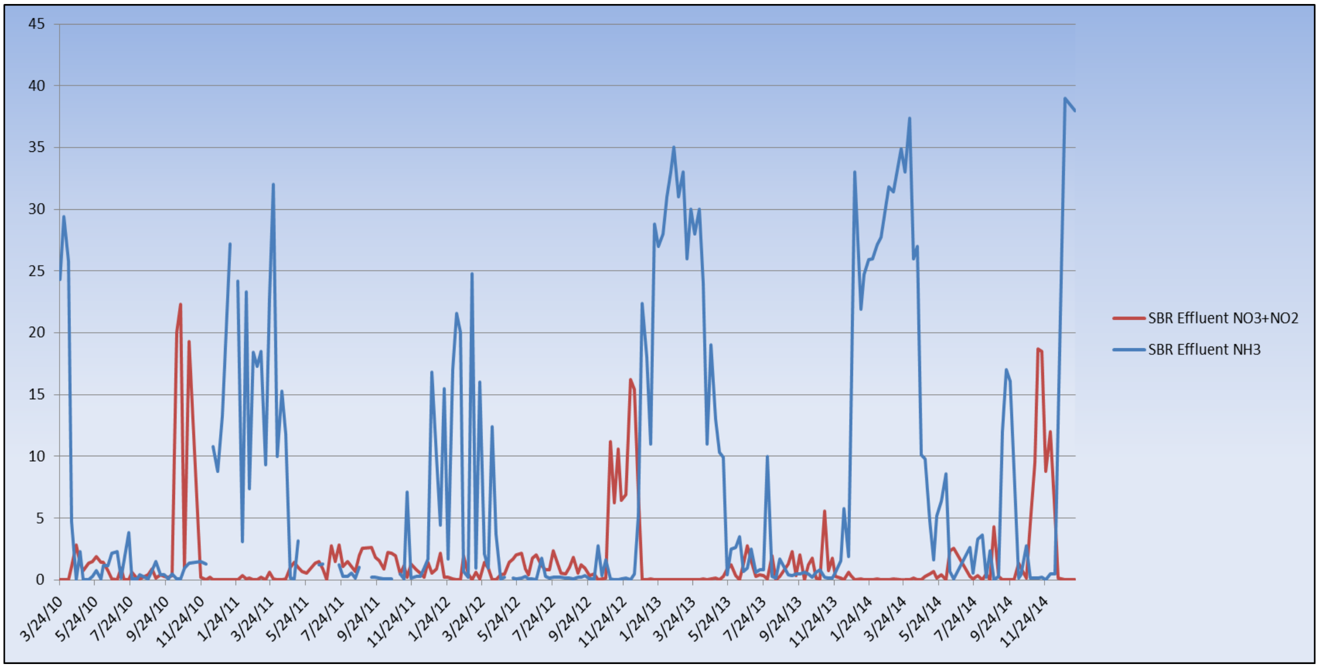
Total cycle time	=	6 hours	React	=	1 hour
Mixed fill	=	1 hour	Settling time	=	1 hour
React fill	=	2 hours	Decant time	=	1 hour

With a total cycle time of 6 hours and 2 basins (8 total cycles) the maximum day flow capacity is reduced to 1.04 MGD. The peak day flow in 2012, 2013, and 2014 has been 0.611 MGD, 0.723 MGD, and 0.923 MGD respectively.

Nitrification is a two-step process where ammonia is first converted to nitrite by ammonia oxidizing bacteria (AOB) and then nitrite is converted to nitrate by another group of nitrite oxidizing bacteria (NOB). The rate of conversion is dependent upon several factors including: pH, temperature, dissolved oxygen levels, and the concentration of AOB and NOB in the SBR basins. Both the AOB and NOB are a relatively small fraction of the biomass in the SBR basins and grow more slowly than bacteria that remove BOD. The growth rates decrease during the winter and consequently it is important to control the volume of biomass wasted from the SBR so that biomass is not wasted faster than the AOB and NOB are able to grow. This type of control is commonly referred to as maintaining the sludge age or mean cell residence time (MCRT) and is defined as the mass of sludge wasted divided by the total mass in the system. Wasting 10 percent of the mass each day results in a sludge age of 10 days. In order to achieve nitrification during the winter, a sludge age of 20-30 days will be required. Tracking the sludge age in a SBR plant is more complicated than other types of activated sludge plants because the biomass concentration and liquid volume are constantly changing. In addition, the concentration of the sludge being wasted can change during the pumping cycle. Improved process control is discussed further in Section 5.5.1

Figure 3-7 illustrates how the effluent ammonia concentration increases during the winter months when the temperature drops, the nitrification rate decreases and ammonia oxidizing bacteria are lost from the system. The performance of the SBR basins is also being impacted by the return flows from the digester supernatant and the belt press filtrate that are returned to the head of the plant with high ammonia concentrations. Grab samples of the supernatant of each digester collected in January 2015 had ammonia concentrations of 53.6 mg/L and 66 mg/L. These values correspond reasonably well with the Biowin process model developed to simulate the current operation of the plant. The operation of the digester has been modified as discussed below to reduce the ammonia levels in the return flow.

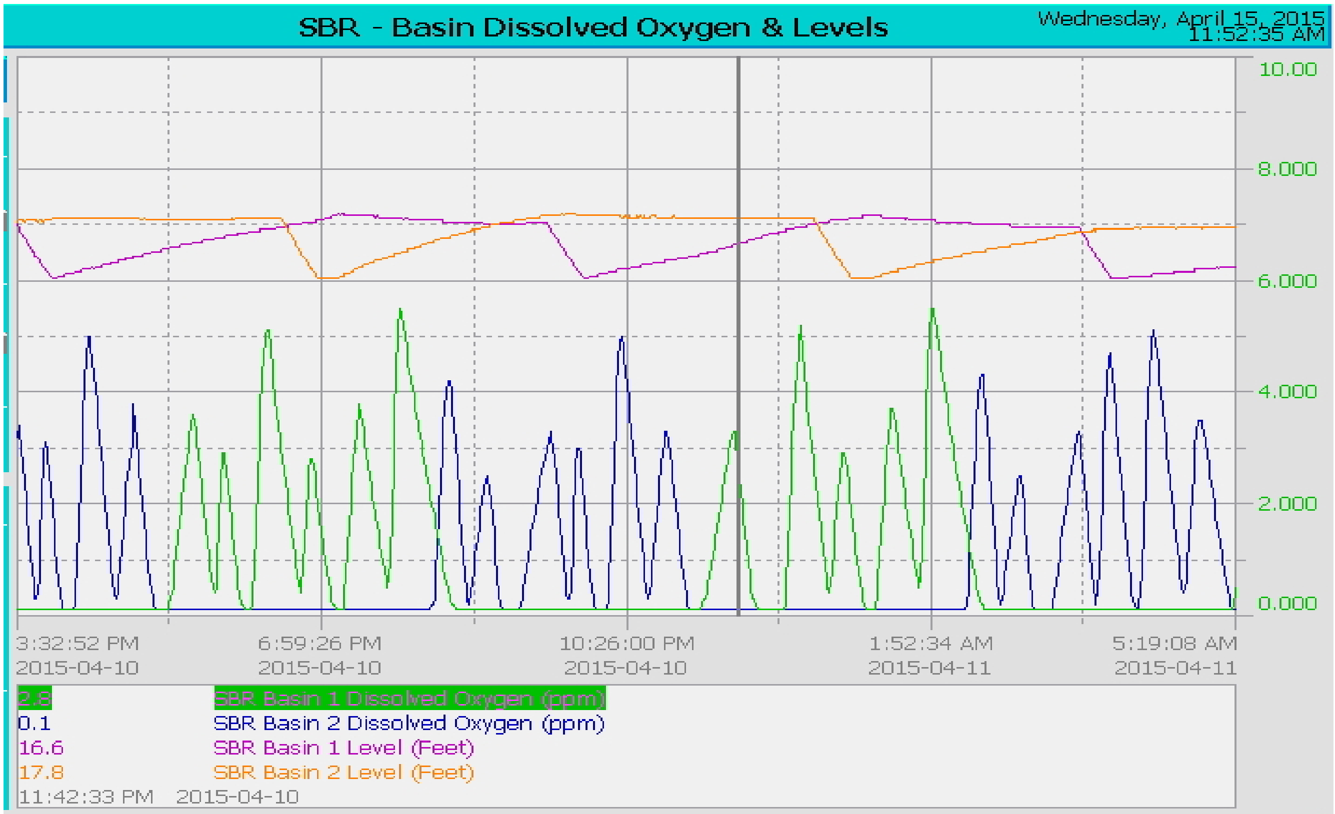
Figure 3-7
SBR Effluent Ammonia and Nitrate+Nitrite Concentrations



The SBR aeration system consists of four constant-speed 100 horsepower blowers (3 duty, 1 standby) that can be controlled either on a time basis or on the dissolved oxygen (DO) concentration in the basin. In the current operation, the blowers are being controlled by the DO in each basin. The DO set points are adjustable and are currently set at a low level of 1.0 mg/L and a high level of 3.0 mg/L with lag times between the start and stop to prevent rapid on/off cycling. Because the blowers are constant-speed, the high DO set point is often exceeded. Figure 3-8 shows plant monitoring data for the DO concentrations in both of the SBR basins and illustrates the cycling of the blowers and the inability to control the DO at the set points.

Six racks of fine bubble diffusers are located in each SBR basin. Each rack consists of 25 diffuser tubes with two EPDM membranes on each diffuser tube. The EPDM membranes have a life of approximately five to eight years. In the summer of 2014, the operators noticed an uneven aeration pattern in the SBR basins and the dissolved oxygen levels could not be maintained even with two blowers operating. The membranes had been in use for ten years and were replaced in the fall of 2014.

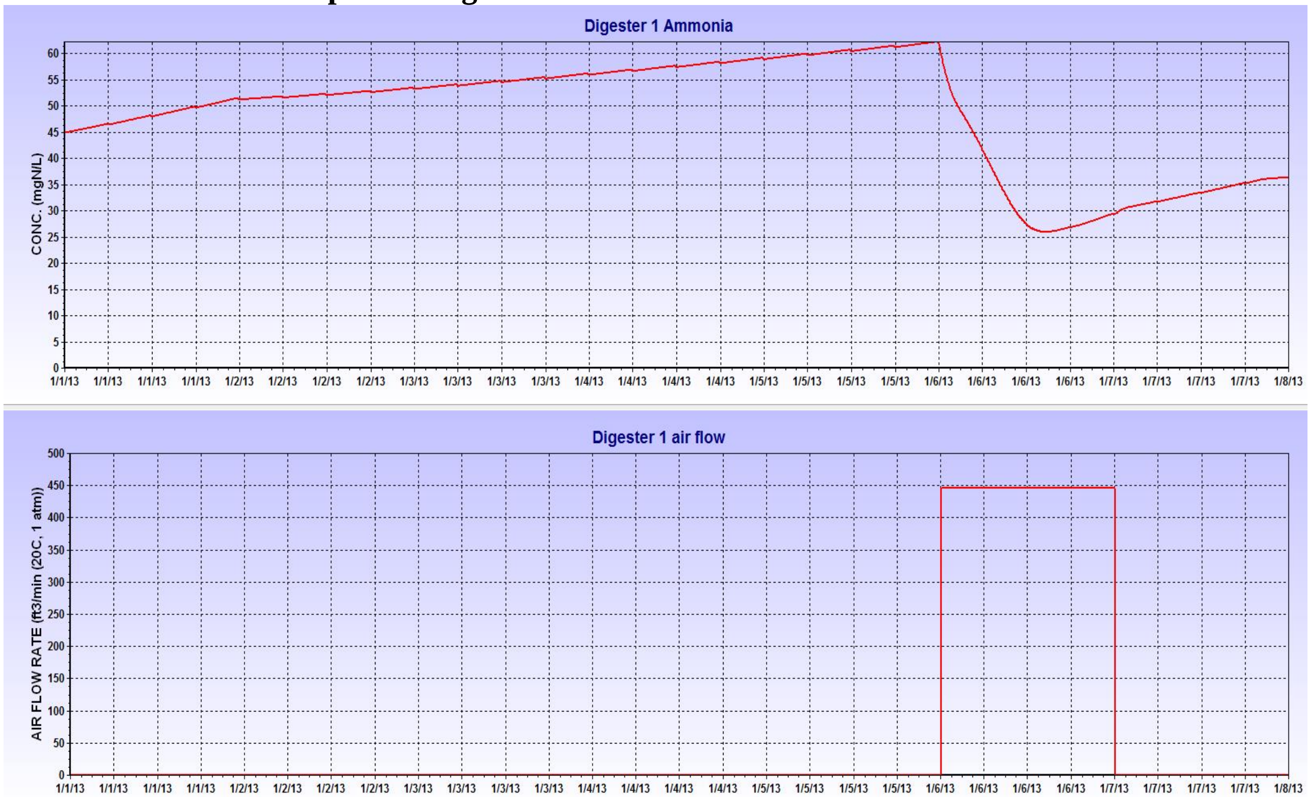
Figure 3-8
SBR Basin Dissolved Oxygen Trend



3.2.1.3 Aerobic Digesters

Excess sludge (waste sludge) generated in the SBR process is pumped to the digesters for further digestion prior to the solids disposal in the composting process. The plant has two aerobic digesters that can be operated in series or in parallel. Aeration is provided by three 50 HP blowers (2 duty, 1 standby) with one blower dedicated to each digester. The aeration also provides mixing. Each digester has a telescoping valve to allow supernatant to be decanted back to the head of the plant. Typically, aerobic digesters are operated by pumping small quantities of waste sludge from the SBR basins at the end of each decant cycle, with intermittent supernatant and digested sludge withdrawals. The basins are continuously aerated during the filling cycle. Prior to decanting supernatant, the aeration system is shut down for a short period (1-6 hours) to allow solids to settle. The unaerated settling time can vary but should be as short as possible to avoid forming anaerobic conditions that result in odors and the formation and release of ammonia as the process undergoes anaerobic digestion. Until recently, the digesters were operated by running the blowers for two to three days and then turning the blowers off for four to five days prior to decanting and composting to save energy costs. Figure 3-9 illustrates how the ammonia levels in the digester increase while the blowers are off and decrease when the aeration is restarted. The District has changed the operation to run the blowers continuously until the day before they are going to decant. The aerobic digester operation could be further improved through ammonia and nitrate monitoring to control the blower operation.

Figure 3-9
Impact of Digester Aeration on Ammonia Concentration



3.2.1.4 Pumps

There are several pumps located throughout the treatment plant. In most cases, the pumps are functioning properly, the exception being the plant water pumps. The plant water pumps are intended to supply non-potable water to the headworks screens and the belt press but are no longer used due to plugging problems reportedly from hair and fine material that passes through the headworks fine screens. The pumps are submersible well pumps located in the post-equalization basin with design conditions of 18 gpm at 104 psi. It is recommended that the pumps be replaced with a slightly larger pump that is equipped with a self-cleaning screen. The estimated cost of the improvement is $7,200

The District has experienced problems with the reclaimed water pumps that pump to the YMC storage pond. The pumps operate at a discharge pressure around 500 psi. The driveshaft failed on two of the pumps after short operating periods possibly from an unbalanced flow on the suction side. The pumps have recently been replaced and piping changes were completed to ensure better hydraulic conditions on the pump suction.

Three effluent pumps are used to pump effluent from the post equalization basin to the SBR effluent pond. One of the pumps recently failed due to a failed shaft bearing and wear of the pump bowl. The District has ordered replacement parts for the failed pump and a set of spare parts.

3.2.1.5 Filtration and Disinfection

Effluent from the SBR is pumped across the Middle Fork to an 8.2 MG storage pond and from there the water is pumped to the filtration process for tertiary treatment and disinfection. The filtration system uses three process trains with each train having a coagulation, flocculation, sedimentation, and filtration step. Solids removed in the sedimentation process and backwash water from the filters are returned to the collection system. Aluminum sulfate (alum) can be added to aid in the flocculation process and also for additional phosphorous removal. Other than normal maintenance issues, the filtration process continues to function as designed and no modifications or capital expenditures are anticipated in the next five years. The filtration capacity is 0.69 MGD (251 MGY) and at the high-growth rate projection the filters would be at capacity around the year 2027.

Disinfection is accomplished with chlorine gas fed from 150 pound cylinders. A chlorine gas scrubber, with an alarm system, is housed in the filtration building to scrub chlorine gas in the event of a chlorine leak.

3.3 EFFLUENT DISPOSAL

Effluent from the treatment plant is currently used for irrigation on the Meadow Village golf course with some disposal also at the Yellowstone Club course. The owners of the Spanish Peaks golf course are in the process of designing a connection to the effluent reuse line the District uses to pump from the Meadow Village to the storage pond at the Yellowstone Club. The disposal capacity of the golf courses can be limited by any of the following factors and various values have been used in past studies:

1. the vertical permeability (percolation rate) of the soils,
    - a limiting hydraulic rate of 4% of the measured percolation rate is normally used to calculate the hydraulic capacity of the golf course soils. Three ring permeameter tests were conducted on the Meadow Village course in 1995 and the weighted average of the soil type and the minimum permeability observed in the soil type were used in calculating the hydraulic capacity. The limiting permeability for both the Spanish Peaks and Yellowstone Club golf courses were estimated based on soil permeability rates published by the Soil Conservation Service and the Natural Resources Conservation Service. The limiting rates used to calculate the capacity were (8) (9) (10):
        - **Meadow Village:** 4% x 1.58 inches /hour x 8 hr/day = 0.50 inches per day
        - **Spanish Peaks** 4% x 0.2 inches / hour x 24 hr/day = 0.19 inches per day
        - **Yellowstone Club** 4% x 2 inches / hour x 24 hr/day = 1.92 inches per day
2. the agronomic uptake rate of the turf grass
    - Various estimates have been used in past reports (8) (9) (10) regarding the nitrogen agronomic uptake rates but all of capacity calculations have been based on a 137 day growing season. It is unlikely that the Spanish Peaks and Yellowstone Club course have a growing season as long as the Meadow Village course.
        - **Meadow Village** -150 pound/acre with 108 pound / acre being supplied by irrigation, 36 pounds/acre from decomposition of grass clippings, and 5.5 pounds/acre of commercially applied fertilizer
        - **Spanish Peaks** 178.4 pounds/acre
        - **Yellowstone Club** 147 pounds/acre
        -

3. nitrogen concentrations in the applied water
    - Different nitrogen concentrations were used in calculating the allowable application based on agronomic uptake rates:
        - **Meadow Village** DEQ approval based on 15 mg/L –N in applied water
        - **Spanish Peaks** Allowable volume based on 5 mg/L-N in applied water
        - **Yellowstone Club** Allowable volume based on 24 mg/L-N in applied water
4. weather conditions: limiting capacity is based on the wettest year in 10.
    - **Meadow Village**: A 10-year recurrence precipitation value of 16.19 inches was used based on records from station 0775 (Big Sky 3S)
    - **Spanish Peaks and Yellowstone Club:** A 10-year recurrence precipitation value of 12.96 inches was used based on records from Lake Yellowstone
5. playability of the course: This criteria depends on the underlying soils and geology and the course operation.

Table 3-7 summarizes the various factors used in prior estimates of irrigation capacity of each golf course.

Table 3-7
Summary of Factors used in Prior Golf Course Capacity Estimates

Factors Used in Prior Capacity Calculations	Meadow Village Course	Spanish Peaks Course	Yellowstone Club Course
Limiting Permeability	0.5 inches/day	0.19 inches/day	1.92 inches/day
Agronomic Uptake of Nitrogen Supplied through Irrigation	108 pounds/acre	178.4 pounds/acre	147 pounds/acre
Total Nitrogen Concentration in Irrigation Water	15 mg/L-N	5 mg/L-N	24 mg/L-N
Irrigation time/ day	8 hours/day	24 hours/day	24 hours/day
Irrigation Season	137 days	137 days	137 days
Precipitation during Irrigation Season	16.19 inches	12.96 inches	12.96 inches

Table 3-8 lists the capacities of the courses based on the permitted value for the Meadow Village course and the calculated values in the Preliminary Design Reports for the Yellowstone and Spanish Peaks courses.

Table 3-8
Golf Course Irrigation Capacities Based on Permitted and Calculated Values

	Wettest Year in 10 Irrigation Capacity
Meadow Village Course	206.MG
Yellowstone Club	100.8 MG
Spanish Peaks	76.0 MG
Total	**382.8 MG**

The original approval from the Montana DEQ for irrigation for the Meadow Village course was based on a total nitrogen concentration of 15 mg/L and a disposal volume of 206 million gallons per year. The Nutrient Management Plan (8) calculated the disposal capacity at 308.2 million gallons in a wet cool year and 317.3 million gallons in an average year with a total nitrogen level of 10 mg/L-N although it was noted that groundwater mounding and course playability would limit the irrigation volumes to less than the calculated volumes. At lower nitrogen concentration more effluent could be applied and at higher concentration less volume could be applied. As shown in Table 3-4, in 2013 and 2014 the total nitrogen content in the irrigation water has been 13.4 mg/L-N and 16.8 mg/L-N respectively. The 2013 and 2014 TN values increased significantly from the prior three years when the TN concentrations were below 10 mg/L-N. It is suspected that the increase was mostly due to failed air diffusers in the SBR basins that were replaced in the fall of 2014 as discussed in Section 3.2.1.2.

The irrigation capacity of the Yellowstone Club golf course was estimated to be 100.8 MGY based on an applied nitrogen concentration of 24 mg/L (10). The estimated capacity was reduced to approximately 95 MGY based on the ability of the soils to drain. (11). In actual practice the Yellowstone Club course currently uses little reuse water from the District.

While not currently connected to the District's reuse irrigation system, the Spanish Peaks golf course owners have expressed an interest in using reuse water for their course. The Preliminary Design Report (9) estimated the irrigation capacity at 76 MGY but it appears the calculation did not include the 3:1 dry-wet ratio required by the Montana DEQ. The course is approximately 95 acres and the historical irrigation has been between 20-30 MGY.

Under a 2001 agreement between the District and Yellowstone Mountain Club, the District has the right to dispose of up to 160 MGY of treated wastewater on land owned by YMC. The 160 MGY includes the irrigation on the YMC golf course and the potential irrigation of the Spanish Peak's golf course.

Table 3-9 shows the results of normalizing the various input data to an applied nitrogen concentration of 10 mg/L-N, a nitrogen uptake of 150 pounds per acre with 108 pounds being supplied by nitrogen in the applied effluent, and a dry/wet ratio of 3:1, and assuming a sprinkler efficiency of 90 percent. For Table 3-9 the limiting hydraulic rates discussed above were applied.

Table 3-9
Calculated Golf Course Capacities (Normalized values)

	Wettest year in 10 and TN of 10 mg/L-TN
Meadow Village Course	218 MGY
YMC	184 MGY
Spanish Peaks	30 MGY
Total	432 MGY

It is emphasized that actual testing of the soil percolation rate has only been completed on the Meadow Village course and even then only three tests were completed. The high percolation rate (2 inches/hour) published for the YC golf course seem unusually high and is not supported by the actual irrigation volumes used at the course or the opinion of the golf course manager. In addition, the irrigation values calculated for both the YC course and the Spanish Peaks course were based on a 24 hour irrigation cycle and consequently would not provide the required 3:1 dry wet cycle. **It is recommended that additional testing be completed on all three courses to better define the true irrigation capacity.** Based on the irrigation volumes currently being used, what is perhaps a more realistic estimate of the irrigation capacity of the courses is shown below.

Table 3-10
Empirical Golf Course Capacities

	Wettest year in 10 and TN of 10 mg/L-TN
Meadow Village Course	140-160 MGY
YMC	22 -28 MGY
Spanish Peaks	20-30 MGY
Total	182-218 MGY

Groundwater mounding under the application area can also impact the volume of irrigation water that can be applied. A detailed groundwater hydraulic model was outside the scope of this study but a simplified calculation based on an EPA manual, indicates mounding will not occur on the Meadow Village course at the application rates listed. Data was not available to estimate potential mounding at the YMC or Spanish Peaks course.

The main lines from the filtration pump house to the Meadow Village golf course consists of a 10-inch AC pipeline that serves the southern side of the course and a 12-inch line that serves the northern side of the course. When the velocity in the lines is limited to 5 feet per second to control pipeline surges, the capacity of the 10-inch line is 1,224 gpm and the capacity of the 12-inch line is 1,762 gpm. Based on an 8-hour irrigation cycle, the delivery pipeline capacity to the course is limited to 1.46 MGD.

3.4 STORAGE

Since the District's only current disposal alternative is summer irrigation, treated wastewater must be stored from fall until irrigation season begins in the late spring. The storage volume available to the District consists of 79.7 MG at the Meadow Village and 80 MG at the YMC for a total volume of 159.7 MG. An 8.2 MG SBR effluent storage pond is also located in the Meadow Village but it has limited storage capacity due to the configuration of the piping and remains nearly full at all time.

In the 2001 agreement, YMC agreed to construct ponds with a total storage capacity of 130 MG of which 80 MG has been constructed and another pond is currently being designed at Spanish Peaks that will have a volume of approximately 11.5 MG, leaving a volume of approximately 38.5 MG of storage volume left to be constructed under the terms of the agreement.

Table 3-11 illustrates the calculated storage requirement assuming a full buildout influent flow of 314 MGY, a monthly influent flow pattern similar to the existing pattern, and a cool-wet year that would limit irrigation, and no winter disposal of effluent. In addition to the 314 MGY of wastewater flow, precipitation for the wettest year in ten in both the Meadow Village area and Mountain Village area has been added to the required volume that must be stored. The assumed pumping schedule to YMC is set to maintain the volume in the Meadow Village storage just below the maximum volume of 79.7 MG.

As indicated in the water balance, a minimum storage volume of 147.74 MG is needed at the YMC, or another location, leaving a shortfall of 17.74 MG from the 130 MG contained in the 2001 agreement.

Table 3-11
Storage Requirements Based on Permitted and Committed Irrigation Volumes

	Inflow At Full Buildout MG	Meadow Village Precipitation Inches [1]	Precipitation Volume MG [2]	Meadow Village Irrigation Volume Maximum Cool-wet Year MG	Pumped Volume to YMC Storage MG	Meadow Village Storage Volume MG	Mountain Village Precipitation Inches	Precipitation Volume MG [3]	YMC or Spanish Peaks Disposal MG	YMC Storage MG
Oct	16.4	1.80	0.94	0.00	12.00	5.32	2.92	1.40		13.40
Nov	18.6	1.74	0.91	0.00	18.00	6.87	3.00	1.43		32.83
Dec	51.5	1.84	0.96	0.00	22.00	37.28	3.27	1.56		56.39
Jan	30.5	1.70	0.89	0.00	22.00	46.64	3.07	1.47		79.86
Feb	35.9	1.35	0.70	0.00	20.00	63.21	2.60	1.24		101.10
Mar	35.6	1.91	1.00	0.00	25.00	74.82	3.76	1.80		127.90
Apr	20.9	1.94	1.01	0.00	18.00	78.72	3.86	1.84		**147.74**
May	15.2	3.46	1.80	11.78	5.00	78.94	5.68	2.71	9.05	146.41
Jun	21.7	3.96	2.06	47.60		55.10	5.07	2.42	37.59	111.24
Jul	19.5	2.30	1.20	66.38		9.44	2.80	1.34	47.34	65.24
Aug	21.4	2.20	1.15	57.62		-25.64	2.70	1.29	43.26	23.27
Sep	26.7	2.47	1.29	22.60		-20.28	3.64	1.74	23.23	1.78
	314		13.91	205.98				20.25	160.47	
1. Wettest year in 10 Station 0775										
2. Based on pond storage area of 19.2 Acres in Meadow						Total Disposal Volume (MGY)=			366.45	
3. Based on pond surface area of 17.6 acres for 160 MG of storage						Total Inflow Volume including precipitation (MGY)=			347.95	

It should be noted that the Meadow Village irrigation volumes shown in Table 3-11 are based on the DEQ permitted volume of 206 MG for the Meadow Village, 160 MG of disposal at YMC or Spanish peaks, and are the maximum volumes that could be applied if irrigation water was available. In the water balance shown, the Meadow Village ponds would be emptied during July and irrigation of the golf course would be limited. Reconfiguring the piping in the SBR effluent storage pond to allow the pond level to fluctuate would provide an additional 8.2 MG of storage.

An existing 12-inch pipe connects the SBR effluent pond overflow to the recirculation pump station located just east of the SBR effluent pond. The exact depth of the line is not shown on the existing drawings but it appears to be approximately 3-1/2 to 4 feet below the high water level in the SBR pond which limits the drawdown of the pond. The line is laid in the embankment between the SBR effluent pond and pond 1 along with another 8-inch force main. Due to the narrow width of the embankment, the other pipe in the alignment, liner anchor trench, and the inability to drain the ponds it will be very difficult to replace the line in its current location. The cost estimate to reconfigure the piping is based on installing a new 12-inch line to the north of the existing embankment between the SBR effluent storage pond and the fence line. The total project cost to reconfigure the SBR piping is estimated at $187,000

3.5 METERING

There are several discrepancies in the water balance between the metered flows in the treatment plant. Figure 3-10 shows the total flows recorded by the various meters from January 1, 2012 to December 31, 2014. Based on total flows from January 1, 2012 through December 31, 2014 the flow recorded at the treatment plant totaled 376.6 million gallons (SBR effluent meter), the flow to the filters totaled 298.3

million gallons, while the flow pumped to the Meadow Village and YC golf courses totaled 484.5 million gallons. Based on the meters, only 76.8 percent of the SBR effluent was pumped to the filters and flow pumped to the golf courses was 128.6 percent of the SBR effluent flow. The source(s) or cause(s) of the discrepancies are not known at this time. It is recommended additional investigation be completed to resolve the discrepancies. The SBR effluent meter, filter meters, and irrigation meters are all magnetic style meters which have a reported accuracy of 0.5% when properly installed. Potential sources of error include improper grounding, stray electrical currents being conveyed through the power or signal cables, lines flowing part full, or meters that are not sized properly for the flow being measured.

Parshall flumes, when properly installed, have an accuracy of ±3-5%. The accuracy of the Parshall flume will be impacted if the flume becomes submerged which could result if water backs-up in front of the screen before the screen operates. This impact will result in the flume reading being higher than the actual flow.

3.6 NEED FOR PROJECT

Even though the irrigation of the Meadow Village golf course with treated wastewater meets the DEQ requirements for irrigation at rates not exceeding the agronomic nitrogen uptake rate, water quality monitoring by the Blue Water Task Force (BWTF) suggests that the irrigation on the Meadow Village golf course is impacting the West Fork as the stream travels through reach bounded by the golf course (12). Monitoring by the BWTF showed increasing concentrations of nitrate and chloride as the stream bisects the golf course. Therefore, improving the treatment process to lower the concentration of total nitrogen in the applied wastewater would be expected to reduce the water quality impacts on the West Fork.

The reported disposal capacity of the YMC golf course appears to be overstated. The course currently uses much less than the estimated capacity of 95 MGY and is more in line with the 25-30 MGY used at the Spanish Peaks course.

The District has two irrigation water rights from the West Fork of the West Gallatin River to irrigate the Meadow Village golf area from June 1 to October 19. Each right is for a maximum volume of 720.00 acre-feet for a total of 1440 acre-feet (469.2MG). In order to maintain the irrigation right, the District needs to put the right to use to show a historical record of beneficial use. Using water from the West Fork to irrigate the golf course occasionally would also help to flush salts from the soil that can accumulate with the continued use of effluent for irrigation.

The need for the project is three-fold: 1) reduce the level of nutrients applied to the Meadow Village golf course and 2) provide additional disposal in the spring of the year to allow the District to irrigate with water from the Middle Fork to show a beneficial use of their water right, and 3) the current disposal capacity is approximately 95.8 to 131.8 MGY short of the projected full buildout flow.

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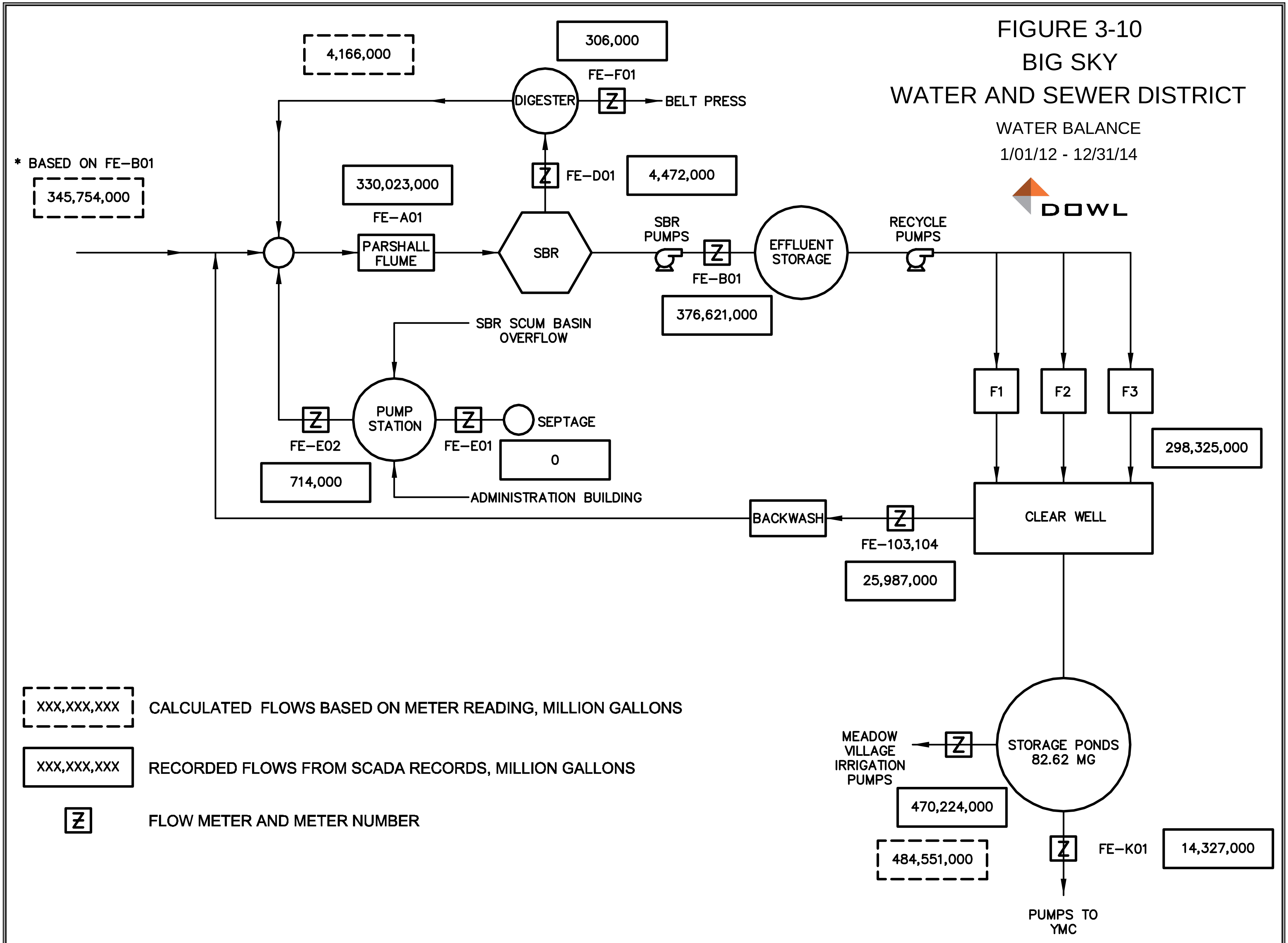

4.0 ALTERNATIVES CONSIDERED

4.1 SCREENING OF ALTERNATIVES

In order to reduce the mass of nutrients applied to the golf course and provide additional disposal options, the following alternatives were considered:

- Alternative 1 Jack Creek Disposal: In this alternative wastewater from the Mountain Village would be collected and pumped to a new treatment plant and disposal site located on the alluvial fan on the west side of the Madison Range near Ennis.

- Alternative 2: Purple Pipe System: This alternative consists of installing a purple pipe system in the Meadow Village area to allow the use of highly treated wastewater for landscape irrigation in residential, commercial, and public areas.

- Alternative 3: Indirect Potable Reuse: Indirect Potable Reuse (IPR) is defined as the augmentation of a drinking water source with reclaimed water followed by an environmental buffer that precedes drinking water treatment. The State of Montana allows the use of either Class A or A-1 water for indirect potable reuse. However, if Class A water is used the environmental buffer must provide at least 180 days of retention before use. Aquifer recharge or aquifer injection, where reuse water is injected or conveyed directed to the aquifer, must meet either the primary drinking water standards, or the nondegradation requirements at the point of discharge (MCA 75-5-410). This alternative consists of injecting highly treated wastewater into an aquifer to supplement the potable water supply currently used by the District. In this alternative the reuse water would have to meet Class A-1 standards and the primary drinking water standards or nondegradation requirements.

- Alternative 4: Subsurface Disposal: This alternative consists of locating suitable grounds to utilize a drainfield or subsurface infiltration bed for winter or summer and winter disposal for a portion of the treated wastewater.

- Alternative 5: Upgrade Existing Treatment Plant: In this alternative, potential improvements at the existing treatment plant to reduce the level of nutrients in the water applied to the golf course(s) are considered. In particular it would be desirable to reduce the total nitrogen level to less than 5 mg/L so the water would be classified A-1 and could be applied to land at greater than the agronomic uptake.

- Alternative 6: Surface Discharge: Utilizing a surface discharge to the Gallatin River during the winter periods to reduce storage requirements and land disposal volumes.

- Alternative 7: Snowmaking: This alternative consists of using snowmaking during the winter to effectively “store water” instead of constructing new ponds. The State reuse guidelines allow snowmaking as a reuse alternative with either Class A1 or B-1 water where the melt water is designed for discharge to groundwater and Class A-1 water when the discharge is in an area of unrestricted access. Additionally any discharge to surface water must be authorized under a discharge permit.

- Alternative 8: Direct Potable Reuse: Direct Potable Reuse (DPR) is defined as the introduction of water from a reuse facility into a raw water supply immediately upstream of, or directly into, a drinking water treatment plant or directly into a potable water distribution system.

4.1.1 Alternative 1. Jack Creek Disposal

This alternative is an update of an alternative that was considered in the 1997 Long Term Compliance Work Plan (13). As discussed in the 1997 plan, land suitable for spray irrigation in the vicinity of Big Sky is limited. Lands suitable for irrigation are also highly valued for residential and/or commercial development. Typically, the most favorable location for a spray irrigation system is on agricultural land where the landowner benefits from increased crop production and the potential for human contact with the irrigation water is minimized. Wastewater used for irrigation of agricultural land does not require the high level of treatment needed for the irrigation of a golf course where the potential for human contact is high. The nearest agricultural land that has suitable site characteristics is near the mouth of Jack Creek in the Madison Valley.

In this option, wastewater from the Mountain Village would be intercepted in the existing sewer line just below Lake Levinsky at an elevation of approximately 7,400 feet. The raw wastewater would be pumped in 8-inch cement-lined ductile-iron pipe to the drainage divide which is at an elevation of approximately 7,820 feet. From the divide, wastewater would flow by gravity to the mouth of Jack Creek in an 8-inch PVC line. A new treatment plant could be located in the Big Sky area or near the disposal site. This alternative is considered further in Section 6.

4.1.2 Alternative 2 Purple Pipe System

This alternative basically consists of an expansion of the golf course irrigation system to include residential, commercial, and other public areas. The phrase "purple pipe system" is commonly used to describe the process where non-potable water is used for un-restricted irrigation where the public is likely to come into contact with the water because the distribution system pipes are impregnated with a purple dye to distinguish them from potable water lines and sewer lines. This type of system is being used more frequently in arid regions. The reuse provides two advantages 1) it provides a greater area for irrigation disposal and 2) it reduces the amount of potable water used. This alternative is considered a viable alternative and will be evaluated in more detail in Section 6 of this report.

4.1.3 Alternative 3 Indirect Potable Reuse

Indirect Potable Reuse (IPR), as used in this report, refers to constructing an aquifer injection system to supplement the potable water supply. In this alternative the reuse water would have to meet Class A-1 water, and primary drinking water standards.

The governing authority for the permitting and operation of injection wells is the Environmental Protection Agency (EPA) which has five classifications for underground injection wells. Several factors determine which type of well will be required and the main issue is the aquifer water quality and wastewater quality. Another key component to an injection well program is to define the lowermost

Underground Source of Drinking Water (USDW). The EPA considers an aquifer an USDW if the concentration of total dissolved solids (TDS) is less than 10,000 mg/L.

An injection well used for disposal of municipal wastewater may be considered either a Class I or Class V injection well. A Class I well injects wastewater into a bedrock formation that is far below the lowermost USDW. A well that injects wastewater above or into a USDW or into shallow aquifer composed of unconsolidated deposits is considered a Class V injection well.

For purposes of discussion, shallow injection into unconsolidated deposits and deep injection into bedrock whether it is above or below the lowermost USDW (Class I and V wells) are presented separately.

DEEP INJECTION WELLS - CLASS I WELL

The geology in the Big Sky area consists of a thick sequence of bedrock varying widely in age that have been deformed by a complex series of folds and faults. The bedrock is overlain by Quaternary deposits consisting of landslide, glacial, alluvial, and colluvial material. Comprehensive hydrogeologic reports of the Big Sky area have been prepared by Baldwin (14). In 2005 HKM prepared a feasibility study for drilling a new water supply well for the Big Sky Water and Sewer District (15). Within the HKM report, the hydrostratigraphic units, associated water quality information, and aquifer hydraulic characteristics for the Big Sky area were presented in detail. The units discussed include Quaternary deposits, Cody Shale, Frontier formation, Mowry Shale, Muddy Sandstone, Thermopolis Shale, Kootenai formation, Morrison formation, and the Madison Group. There are water supply wells completed in all of these formations. Water quality information for the formations above the Madison Group indicate that most of the units have TDS concentrations below 1,000 mg/L.

The Madison Group is regionally an important aquifer and recharges the Gallatin River within the study area (16). Using the GWIC database, Schaffer identified 12 wells in the Big Sky area that terminate in limestone (presumably Madison limestone). HKM reported that the Madison limestone is at a depth of about 2000 feet below Meadow Village. HKM indicated that no water quality information was available for the Madison Group in the area. The GWIC database contained one water quality sample for a site at Snowflake Springs located approximately 9 miles south of the current report's study area. The source water for the spring is reported to be from the Madison and had a TDS of 290 mg/L.

Based on published data, it appears that all aquifers above and including the Madison Group in the Big Sky area would be considered USDW's and in fact are currently being used as such. Because of the variability in topography and the complex geology it is difficult to assess whether formations older than the Madison are being utilized as sources of water in the area of review. However, a review of the GWIC database for Township 6 South, Range 4 East indicates that numerous water supply wells are completed in older Paleozoic and Precambrian rocks. The GWIC database contained three water quality samples for these wells and the TDS ranged from 215 to 313 mg/L indicating that older aquifers are also of a high enough quality to be considered USDW.

A search of the MBMG database revealed that no wildcat oil wells have been drilled in the area and no additional information on deeper formations was obtained.

As reported by HKM the hydrogeology is complicated by major and minor faulting and fracturing. The fracturing provides secondary porosity for many of the formations and the faulting and fracturing also create communication between aquifers. The wide range in yield and hydraulic parameters seen in the bedrock formations may be a result of the degree of fracturing and amount of interconnection.

In order to permit a Class I well, a geologic study of the injection and confining zones would be required to determine:

- The receiving formations are sufficiently permeable, porous, homogeneous, and thick enough to receive the effluent at the proposed injection rate without requiring excessive pressure
- Formations are large enough to prevent pressure buildup and injected effluent would not reach aquifer recharge areas
- There is a low-permeability confining zone to prevent vertical migration of effluent
- The effluent is compatible with the well materials and with rock and fluid in the injection zone
- The area is geologically stable
- The injection zone has no economic value.

Due to the complex geology in the area, the conditions listed above could not be met and it is unlikely a Class I well could be permitted. For these reasons, deep well injection is not considered a viable alternative and will not be considered further.

SHALLOW INJECTION - CLASS V WELL

There are over 20 different types of class V wells. For the disposal of treated effluent, the well is termed a Sewage Treatment Effluent (STE) well. These types of wells are used throughout the county for the shallow disposal of treated sanitary waste from publicly owned treatment works. In Montana, the discharge from a STE well will be regulated through the authority of MCA 75-5-401, which requires a permit for discharges into groundwater. Shallow aquifers in the Big Sky area are of generally high quality, Class I, waters with conductivity of less than 560 µS/cm. ARM 17.30.1006 requires that Class I groundwater be maintained for beneficial use with little or no treatment for public and private water supplies and that the nondegradation requirements of 75-5-303 MCA be met.

The only surficial deposits that are of sufficient lateral extent to be considered for possible shallow injection are the Quaternary glacial and alluvial deposits located along the West Fork and the Gallatin Rivers. Based on well log information and geologic research these deposits generally range in thickness from 20 to 60 feet, are several thousand feet wide, and can be several miles in length. These units typically consist of coarse grained material described as mostly gravel, cobbles, and boulders. The aquifer associated with these deposits appears to be the most commonly used source of drinking water in the area including private and public water supply wells, however this in itself does not prohibit the construction of a Class V STE well. This option, in essence, would constitute indirect potable reuse where treated effluent is mixed with the drinking aquifer. This type of system has been used in the United States since the 1960’s to prevent salt water intrusion along the coasts and for groundwater augmentation in water short regions. The use of a Class V STE well will be evaluated further in the alternative analysis section.

4.1.4 Alternative 4 Subsurface Disposal

Alternative 4 consists of screening sites in the Big Sky area for their suitability for subsurface disposal. This alternative is intended to provide a means of effluent disposal for a portion of the effluent during the winter to minimize storage requirement and reduce the volume of wastewater applied to the Meadow Village golf course so that the District can use their existing irrigation right in the spring of the year to apply fresh water to the course. Depending on the location(s) of the disposal site(s), this alternative may involve providing further treatment to removal additional nitrogen prior to disposal. In addition, this alternative could be combined with the purple pipe system where additional irrigation is provided during the summer and during the winter subsurface disposal is utilized. This alternative is evaluated further in Section 6.

4.1.5 Alternative 5 Upgrade the Existing Treatment Plant

Four options are considered in this alternative. Option 5A consists of adding a fixed film media to each of the basins to allow a larger biomass to develop. Nitrifying bacteria have a slow growth rate, especially during cold weather, and consequently maintaining a large enough biomass to nitrify the ammonia load is difficult. A fixed film media allows nitrifying bacteria to be maintained within the system for longer periods. The problem with this option relates to the ability of the media to withstand the shear forces created by the SBR mixer. The high shear forces created by the mixer could tear the media itself and would most likely shear the biomass from the media at a high enough rate that it would not function as intended. The vendor for the SBR (Aqua-Aerobics) indicated they have considered this type of installation but do not have any operational system. Option 5A will not be considered further.

Option 5B consists of adding a third SBR basin to the existing system. The addition of a third basin would increase the capacity of the plant to a peak day flow of 1.96 MGD when operated at 5 cycles per day or to 1.57 MGD when operated at 4 cycles per day. This option is evaluated further in Section 5.

Option 5C consists of modifying the operational control of the system by adding additional probes to control the operation of the blowers to achieve better total nitrogen removal. Additional control/ treatment of the side-streams returned to the plant are also considered in the option. This option is evaluated further in Section 5.

Option 5D consists of replacing the existing tertiary filters with an algae based nutrient removal (ABNR) process designed to reduce nitrogen and phosphorous levels and which incorporates a membrane microfiltration process. This technology is currently in the testing and development stage but there are a few plants in operation that show promising results, including a test facility located in Missoula. This option is evaluated further is Section 5.

4.1.6 Alternative 6 Surface Discharge

This alternative consists of constructing a new pipeline to the Gallatin River and discharging a portion of the District's effluent during the winter. This alternative could also include the option of installing a new sewer along the lower canyon area to remove the existing septic tanks and drainfields located near the Gallatin River. This alternative is evaluated further in Section 5.

4.1.7 Alternative 7 Snowmaking

This alternative has been evaluated in past studies (2) and has been considered a promising alternative but has been limited by the need to prevent surface runoff to surface streams to limit nutrient loading. With the adoption of base numeric criteria that only apply from July1st to September 30th each year coupled with proposed improvements at the existing treatment plant to improve ammonia and total nitrogen removal snowmaking continues to stand out as a viable alternative.

Snowmaking with reclaimed water is being done in the United States, Canada, and Australia (17). Snowmaking using reclaimed water is occurring in Main, Pennsylvania, and California. Given the limited potable water supply in the Big Sky area it is hard to ignore the use of reclaimed water for snowmaking.

This alternative is evaluated further in Section 5.

4.1.8 Alternative 8 Direct Potable Reuse

This alternative would consist of pumping water from the reuse plant directly into the potable water distribution system at one of the storage reservoirs to allow reuse water to mix with water in the tank prior to distribution. To date, no regulations or criteria have been developed or proposed specifically for DRP in the United States (18) however there are several existing potable reuse projects in the United States and abroad. A report published by the WateReuse Research Foundation identified four case studies where cities are serving as case studies to evaluate differing logistical and treatment challenges with DPR but emphasized that the case studies does not imply that DPR is imminent or pending for any of the four cities (19). DPR will most likely be a viable alternative in the future but with the state of current research, lack of full risk assessment, DRP is not considered a viable alternative at this time and will not be evaluated further.

5.0 ALTERNATIVE EVALUATION

5.1 ALTERNATIVE 1 JACK CREEK DISPOSAL

Disposal of water from the Mountain Village by irrigation on a site near the mouth of Jack Creek has been re-evaluated to determine the economic viability of this option. Figure 5-1 shows the conceptual pipeline route overlaid on Montana Cadastral land ownership data and Figure 5-2 shows the conceptual pipeline route overlaid on NRCS Soils Data for irrigation application of wastewater. The route shown is only for cost estimate purposes to test the economic viability of the option. The conceptual route is a total of 14.3 miles long and passes through Commercial Rural (6.7 miles), Agricultural Rural (3.2 miles), Farmstead Rural (2.4 miles), Residential Rural (1.7 miles), and Vacant Land Rural (0.3 miles) property types. Geological conditions have not been explored fully; however, NRCS soils data shows no limiting soil layer within 200 feet of the surface. The pipeline would be buried with 6.5 feet of cover to prevent freezing as prescribed in *AWWA D100-11 Welded Carbon Steel Tanks for Water Storage*. Geotechnical, environmental conditions, and right-of-way acquisition may require realignment changes if this disposal option is selected.

In this option, wastewater from the Mountain Village would be intercepted in the existing sewer line just below Lake Levinsky at an elevation of approximately 7,400 feet. The raw wastewater would be pumped in an 8-inch cement lined ductile-iron pipe to the drainage divide which is at an elevation of approximately 7,820 feet. The pump station would consist of three 100 horsepower pumps (2 duty 1 standby) pumps. Each pump would have a capacity of approximately 550 gallons per minute. The pump station discharge pressure would be 215 psi. An overflow to the existing gravity collection system would be provided to handle flows in excess of peak hour flows and to act as an emergency overflow during power outages. An emergency generator would not be provided for the lift station.

From the divide, wastewater would flow by gravity to the mouth of Jack Creek in an 8-inch PVC line. For this cost estimate we have assumed an aerated lagoon could provide the required treatment. An aerated lagoon located near the mouth of Jack Creek would provide treatment before spray irrigation of the effluent. The lagoon would be sized to treat the projected average day flow from the Mountain Village (0.26 MGD). The projected BOD_5 is approximately 300 mg/l or 650 pounds per day. A 2-cell, 15 foot deep aerated lagoon would be used with a total detention time of 17 days. An area of approximately 3 acres would be required for the aerated lagoon.

A 10 acre storage pond with a depth of 20 feet would be required to store treated water during the non-irrigation season. The storage pond size has been estimated on providing 200 days of storage. The storage volume is estimated on the annual average flow rate for the Mountain Village (0.26 MGD). A storage volume of 52.0 million gallons would be required.

A minimum of 70 acres would be required for irrigation. However, in order to provide reserve capacity and utilize standard equipment, a minimum of 100 acres should be leased or purchased for irrigation with a 1,000 foot center pivot. One 1,000 foot center pivot has the capacity to irrigate 72 acres. Approximately 14 acres would remain untouched (100 acres – 72 acres (irrigation) – 10 acres (storage) – 3 acres (lagoons) – 1 acres (additional space between components) = 14 acres). The 2014 price for purchasing land in Madison County is estimated to be $5,000 per acre. The cost estimate for this alternative is shown in Table 5-1

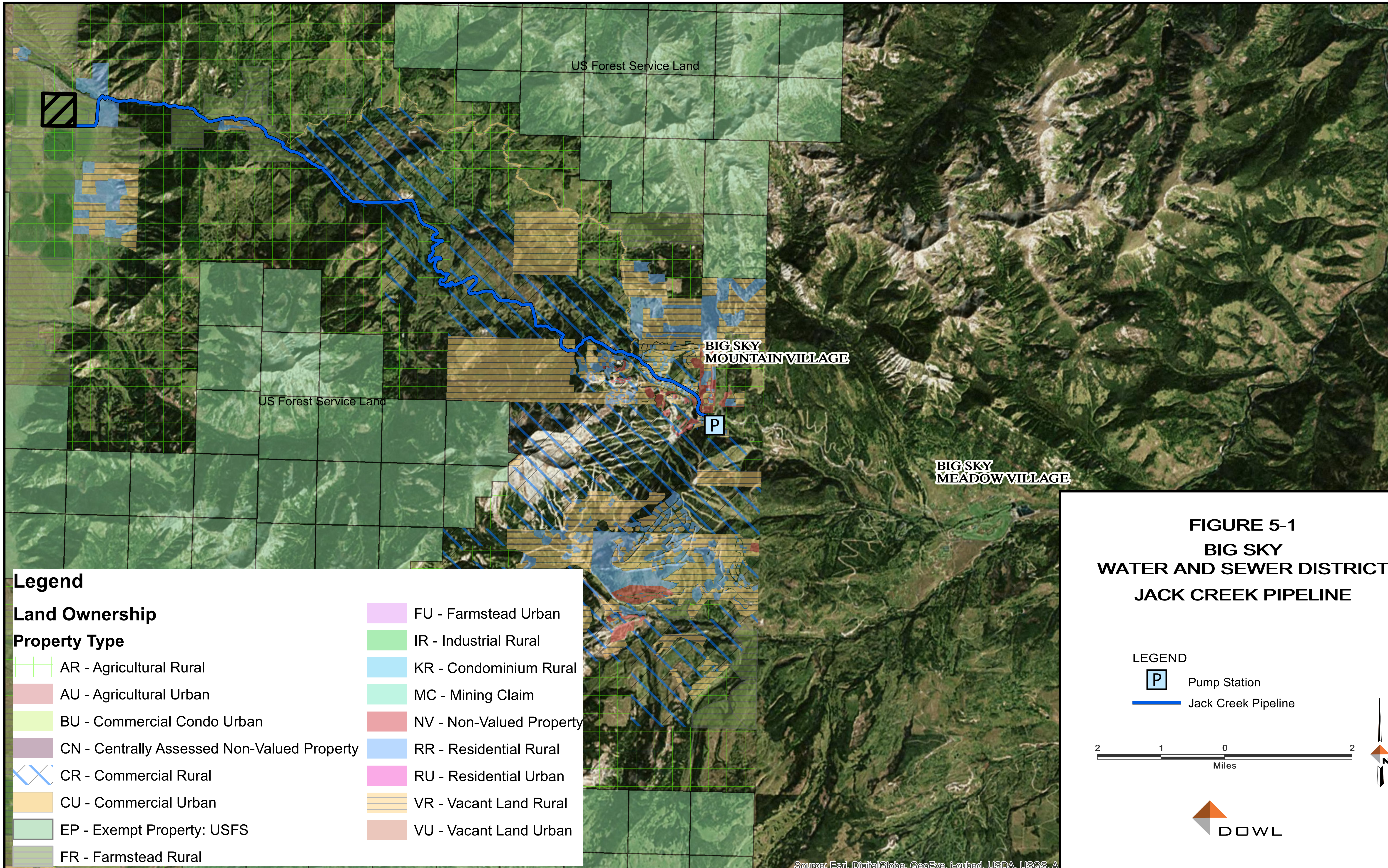

FIGURE 5-1
BIG SKY
WATER AND SEWER DISTRICT
JACK CREEK PIPELINE

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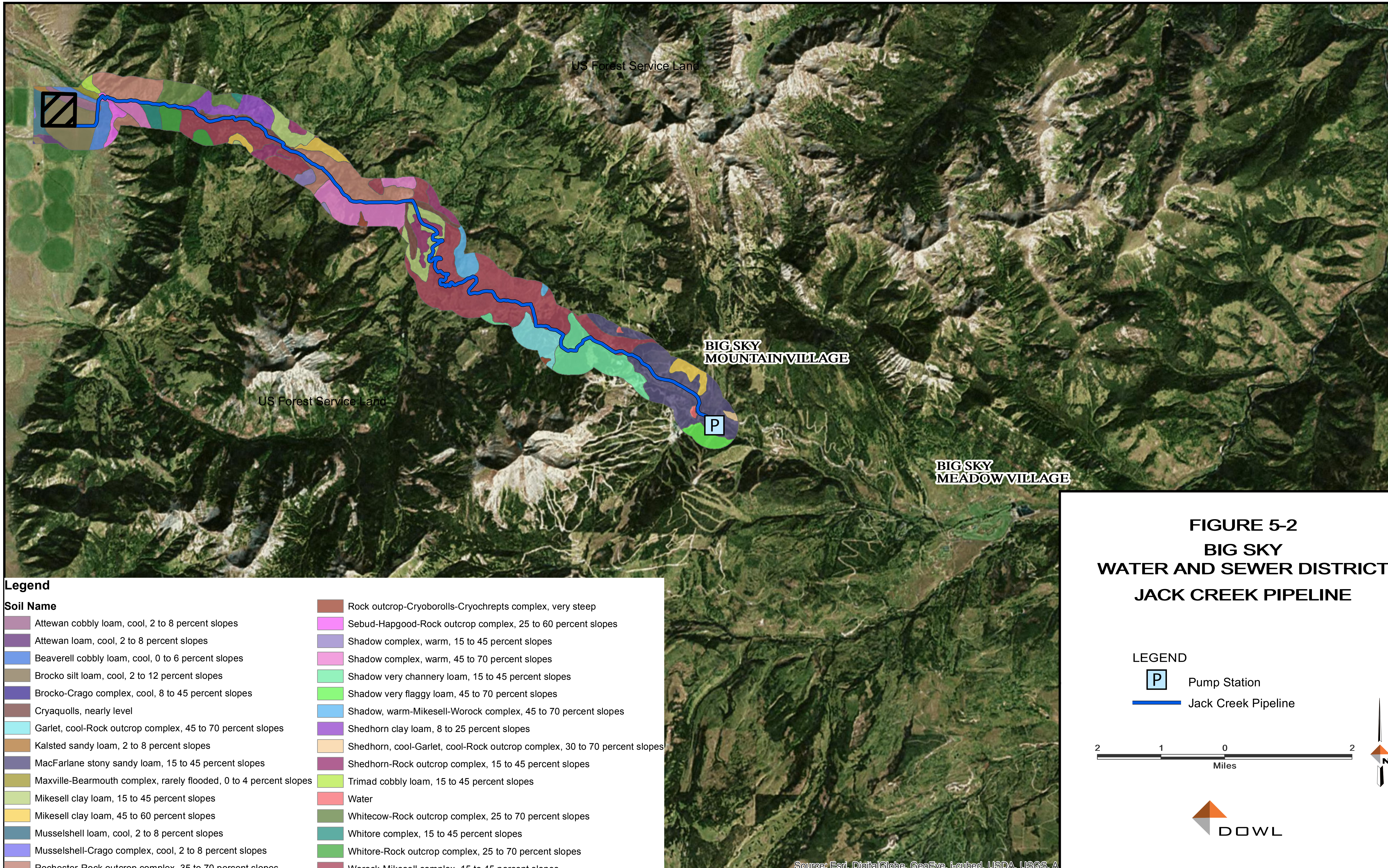

FIGURE 5-2

BIG SKY WATER AND SEWER DISTRICT

JACK CREEK PIPELINE

LEGEND

P Pump Station

Jack Creek Pipeline

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Table 5-1
Cost Estimate for Jack Creek Disposal

Item Description		Estimated Quantity	Units	Unit Cost	Total Cost
A. PUMP STATION					
	Pumps Piping and fittings	3	ea.	$37,000.00	$111,000
	Building: HVAC, Electrical and Construction	1000	SQ FT	$175.00	$175,000
	subtotal				$286,000
B. FORCE MAIN					
	8" Ductile Iron Pipe Material	9,700	LF	$18.20	$176,540
	Trench Excavation, Backfill & Compaction	9,700	LF	$40.14	$389,397
	Pipe Bedding	9,700	LF	$8.42	$81,713
	Pipe Laying (DIP)	9,700	LF	$11.71	$113,591
	Tracer Wire	9,700	LF	$0.18	$1,715
	Non-detectable Warning Tape	9,700	LF	$0.03	$303
	Surface Restoration	9,700	LF	$17.00	$164,900
	Subtotal				$928,158
C. GRAVITY LINE					
	Manholes	105	EA	$4,000.00	$420,000
	8" Ring Tite SDR 35 PVC Pipe Material	69,200	LF	$10.05	$695,211
	Trench Excavation, Backfill & Compaction	69,200	LF	$40.14	$2,777,965
	Pipe Bedding	69,200	LF	$8.42	$582,941
	Pipe Laying (PVC)	69,200	LF	$2.95	$204,389
	Tracer Wire	69,200	LF	$0.18	$12,235
	Non-detectable Warning Tape	69,200	LF	$0.03	$2,159
	Stream Crossings	2	EA	$15,000.00	$30,000
	Surface Restoration	69,200	LF	$17.00	$1,176,400
	Subtotal				$5,901,299
D. AERATED LAGOON					
	Earthwork: Excavation	17,600	CUYD	$4.11	$72,301
	Earthwork: Embankment	17,600	CUYD	$4.19	$73,765
	Liner and Cusion Fabric	134,400	SQ FT	$1.00	$134,400
	Blower Building	500	SQ FT	$200.00	$100,000
	Lagoon Aeration	1	LS	$350,000.00	$350,000
	Site Piping	2,680	LS	$27.00	$72,360
	Valves and Tees	1	LS	$148,000.00	$148,000
	subtotal				$950,826
E. STORAGE LAGOON					
	Earthwork: Excavation	62,000	CUYD	$4.11	$254,696
	Earthwork: Embankment	62,000	CUYD	$4.19	$259,854
	Liner and Cusion Fabric	418,700	SQ FT	$1.00	$418,700
	Site Piping	1,020	FT	$27.00	$27,540
	Valves and Tees	1	LS	$5,000.00	$5,000
	subtotal				$960,790
F. SPRAY IRRIGATION					
	Center Pivot	1	LS	$45,000.00	$45,000
	Pump	1	LS	$1,000.00	$1,000
	Pipeline	1000	LF	$25.00	$25,000
	subtotal				$71,000
G. LAND PURCHASE					
	Land	100	Acre	$5,000.00	$500,000
H. ELECTRICAL TO THE SITE		1	LS	$100,000.00	$100,000
	SUBTOTAL				$9,698,074
	15% CONTINGENCY				$1,454,711.04
	SUBTOTAL				$11,152,784.64
	ENGINEERING/CONSTRUCTION MANAGEMENT				$2,007,501.24
	TOTAL				$13,160,285.88

5.2 ALTERNATIVE 2 PURPLE PIPE SYSTEM

This alternative would connect to the existing golf course irrigation system to include irrigation of residential areas near the course. Figure 5-3 shows the extent of the area identified for potential reuse. Montana allows the use of class A or A-1 reclaimed water for unrestricted irrigation. Class A water can only be applied at less than or equal to the agronomic uptake rate while Class A-1 reclaimed water can be applied at rates greater than the agronomic uptake rate. When used for residential irrigation, the application rates would be difficult for the District to control and consequently the installation of a purple pipe system would only be practical if the existing treatment system is upgraded to produce Class A-1 water.

The proposed purple pipe system is broken into two different areas shown in Figure 5-3: the first includes 37 acres of residential land application area around the homes on the golf course and the second area includes 45 acres in the West Fork development south-west of the golf course. The main distribution line for the golf course residences would be constructed as a 6-inch PVC pipe along Black Otter Road and Curley Bear Road with secondary lines along Yellowtail Road and Two Moons Road. The main line for the development south-west of the golf course would be a 4-inch PVC extension from the 6-inch main line serving the golf course residences. It is possible to complete the purple pipe alternative in phases, constructing the pipelines serving the golf course residences first followed by a second phase adding the West Fork Subdivision. The total cost would be approximately $2,703,000 for phase 1 and an additional $2,331,000 for phase 2. The cost estimate to complete the purple pipe system for both areas is provided in Table 5-2.

Table 5-2
Cost Estimate for Purple Pipe System

Description	Quantity	Units	Unit Costs	Extended Cost
Taxes, Bonds and Insurace at 8 %	1	%	$296,800.00	$296,800
Mobilization at 10%	1	%	$371,000.00	$371,000
2"PVC Material	26,720	LF	$1.10	$29,392
4"PVC Material	12,730	LF	$6.04	$76,889
6"PVC Material	8,510	LF	$11.96	$101,780
2" Fittings and Valves	1	LS	$41,480.00	$41,480
4" Fittings and Valves	1	LS	$40,736.00	$40,736
6" Fittings and Valves	1	LS	$34,040.00	$34,040
Trench Excavation, Backfill & Compaction	47,960	LF	$19.45	$932,822
Pipe Bedding	47,960	LF	$1.62	$77,695
Pipe Laying	47,960	LF	$0.82	$39,327
Tracer Wire	47,960	LF	$0.18	$8,479
Non-detectable Warning Tape	47,960	LF	$0.03	$1,496
Surface Restoration	47,960	LF	$30.50	$1,462,780
Connection to Residential	260	EA	$750.00	$195,000
Subtotal				$3,709,717
15% CONTINGENCY				$556,458
SUBTOTAL				$4,266,174
ENGINEERING/CONSTRUCTION MANAGEMENT (18%)				$767,911
TOTAL				**$5,034,000**

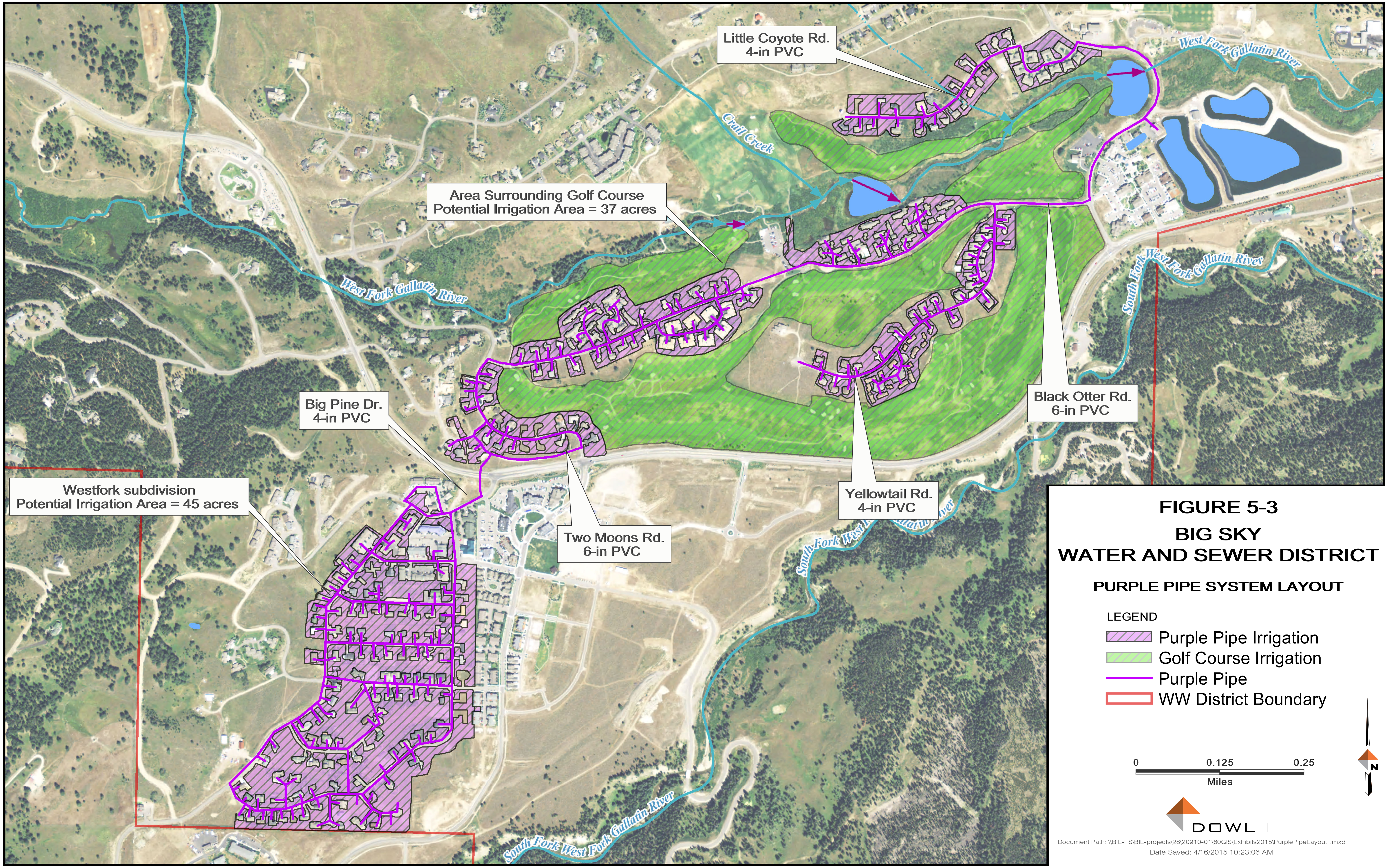

FIGURE 5-3
BIG SKY
WATER AND SEWER DISTRICT
PURPLE PIPE SYSTEM LAYOUT

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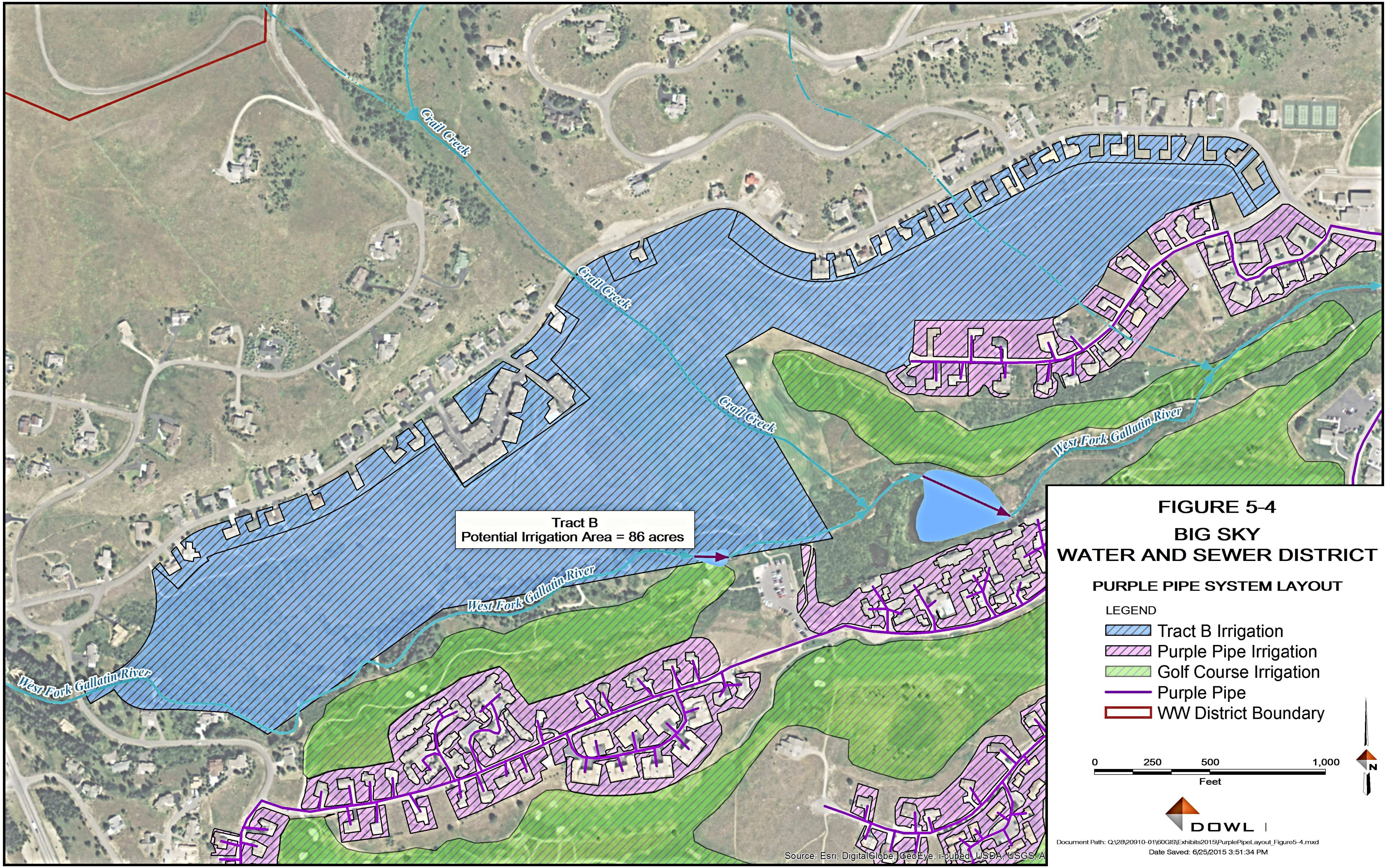
An option that could also be considered in this alternative consists of expanding the current irrigation system to Tract B, as shown in Figure 5-4 which would add an additional 86 acres of land for irrigation. The irrigation expansion could be completed either as part of a purple pipe system or more simply as just an expanded irrigation system. Boyne has expressed the possibility of expanding the golf course into Tract B and if the course is expanded, the irrigation system would be installed as part of the expansion. The added irrigation area would increase the total acreage irrigated from 176.5 acres to 262.5 and would increase the calculated capacity of the course to approximately 324 MG, assuming the same weighted average percolation rate. While outside the scope of this study, it is recommended that additional soils testing be completed to determine the percolation rate of soils in the expanded area.

5.3 INDIRECT POTABLE REUSE- SHALLOW INJECTION WELLS

As discussed in the alternative screening, the only surficial deposits that appear feasible for a shallow injection well are located along the West Fork, ie. the Meadow Village area, and along the Gallatin River. Injection wells located along either site would in effect be an indirect potable reuse since there are currently both public and private potable drinking water wells located in the glacial and alluvial deposits. Because of the public health concerns related to the potential presence of pathogens and trace constituents in the indirect potable reuse, this alternative would only be considered after an advanced treatment process was constructed. The existing filtration plant could function as the first step in an advanced plant but additional treatment would be required following the filters to provide redundancy and remove pharmaceuticals and personal care products (PPCP) and endocrine disrupting compounds (EDC) that are not significantly removed during conventional wastewater treatment processes. An order of magnitude cost to install reverse osmosis and an advanced oxidation process following the existing filtration process is 7 to 9 million dollars. Reclaimed water from this treatment process would be free of pathogens and most trace contaminants would be below detection levels.

A determination of the actual volume of water that could be injected into the aquifer is beyond the scope of this evaluation but for cost estimation purposes it has been assumed that a flow of 200 gpm would be injected through 5 wells (50 gpm/well with one standby) located in the Meadow Village vicinity. A rough order of magnitude cost for constructing and permitting 5 wells injecting approximately 50 feet below ground surface is estimated to range from $350,000 to $500,000. This cost would be in addition to the advanced treatment cost.


Crail Creek
Crail Creek
Crail Creek
West Fork Gallatin River
West Fork Gallatin River
West Fork Gallatin River
Tract B
Potential Irrigation Area = 86 acres
Source: Esri, DigitalGlobe, GeoEye, i-cubed, USDA, USGS, A
FIGURE 5-4
BIG SKY
WATER AND SEWER DISTRICT
PURPLE PIPE SYSTEM LAYOUT
LEGEND
Tract B Irrigation
Purple Pipe Irrigation
Golf Course Irrigation
Purple Pipe
WW District Boundary
0
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5.4 ALTERNATIVE 4 SUBSURFACE DISPOSAL

An initial screening of potential disposal sites was completed by overlaying GIS data onto a map to exclude areas above or directly below mapped landslide areas, slopes greater than 35 percent, wetland and riparian areas, and within a 100 foot buffer of surface streams. The mapped data is shown on Exhibit A bound at the back of this report. Potential sites, within 1 mile of the existing effluent disposal pipeline were then categorized. The categorizing of sites as “good”, “potential”, or “maybe” is subjective but was completed base on slopes, area size, and distance to effluent pipeline. All of the areas are on private property and no weight was given to the parcel’s potential based on specific ownership. Exhibit B shows the mapping of potential sites with identifying numbers assigned to the site.. Soils data was not available in a GIS database so the mapping does not take the type of soils into account. Addition screening was completed on sites that appeared to have a good potential for subsurface disposal by referencing published data from the Madison and Gallatin County Soil Surveys. The published soil survey lists all of the sites as having severe limitations for absorption fields but that classification does not necessarily mean the site cannot be used as evidenced by the numerous drainfields already in use in the area.

Based on the assumption that any drainfield would be limited to a slow application rate (0.15 gpd/ft^2) , with 3 foot wide absorption trenches, and 4 feet between trenches, an area of approximately 35 to 40 acres (including buffers) will be required to dispose of 100,000 gpd, It is expected that several site may be needed due to site limitations such as slopes and proximity to surface streams. Table 5-3 lists the sites identified as possible subsurface application sites along with the soil type. In this table, the sites have been ranked in order with some weight given to the land ownership and the size of the parcel. Site 213 (horse pasture area west of the driving range) was ranked the best potential site since it already has been used for spray irrigation and is owned by Boyne. Site 165 has been listed second because it is a large tract of relatively flat undeveloped property. Site 165 is listed as being owned by a family trust and it is not known if the land owners have any interest in selling a portion of the land for a subsurface drainfield.

It is beyond the scope of this study to evaluate each potential site to determine the volume of wastewater that could be disposed. However, it is recommended that additional soil testing be completed on 3 or 4 of the sites with the best potential so preliminary drainfields designs could be evaluated and potential impacts to surface streams could be determined.

The capital cost to install a drainfield will vary depending on the site, distance from the effluent pipeline or source of effluent, but for sites with slow percolation rates an order of magnitude cost estimate is in range of $10.50 to $15.00 per gallon per day of disposal, excluding any land cost and conveyance cost to the site.

Table 5-3
Potential Subsurface Disposal Sites

Number	Category	ACRES	Soil Map Number	Soil Name	Septic Tank Absorption Fields
213	Potential Site	33.3	608B	Beehive-Mooseflat	Severe: flooding, wetness, poor filter
165	Potential Site	173.9	64-2A	Typic Cryoborolls and Argic Cryoborolls	Severe
206	Good potential	57.8	280B	Libeg cobbly loam	Severe: Large Stones
195	Good potential	13.2	796E	Loberg, very stony-Yellowmule complex	Severe: Percs slowly, slope
199	Good potential	14.7	492E	Yellowmule-Ouselfal complex	Severe: depth to rock, percs slowly
198	Good potential	6.2	207/492E/192/492F	Yellowmule-Ouselfal complex	Severe: depth to rock, percs slowly
197	Good potential	4.8	72	Loberg very stony loam	Severe: Percs slowly, slope
196	Good potential	2.7	72	Loberg very stony loam	Severe: Percs slowly, slope
121	Good potential	49.6	779E/293E	Bridger-Libeg/Stemple cobbly sandy loam	Severe: Percs slowly, slope
145	Good potential	16.6	280B	Libeg cobbly loam	Severe: Large Stones
204	Good potential	12.2	779E	Bridger-Libeg	Severe: Percs slowly, slope
159	Potential Site	6.9	482C/608D	Philipsburg-Libeg/Beehive-Mooseflat	Severe: flooding, wetness, poor filter
160	Potential Site	52.5	280B	Libeg cobbly loam	Severe: Large Stones
161	Potential Site	17.6	779E/608B	Bridger-Libeg/Beehive-Mooseflat	Severe: Percs slowly, slope
192	Potential Site	185.8	80	Mikesell clay loam	Severe: Percs slowly, slope
200	Potential Site	53.1	207	Yellowmule-Ouselfal complex	Severe: Percs slowly, slope
212	Potential Site	7.7	608D	Beehive-Mooseflat	Severe: flooding, wetness, poor filter
215	Potential Site	12.7	779E	Bridger-Libeg	Severe: Percs slowly, slope

5.5 ALTERNATIVE 5 UPGRADE EXISTING PLANT

5.5.1 Option 5B Addition of Third SBR Basin

In this option a third SBR basin would be added to the south of the existing basins. Maintaining the current 6 hour cycle time in all three basin would increase the peak day capacity to 1.57 MGD. With a 5 hour cycle time, the plant capacity would be 1.96 MGD. The new basin would be the same size as the existing basins. Three additional 100 HP blowers with variable speed drives installed on two blowers would be installed in a separate blower building to provide additional air to the basins. Variable speed drive controllers would also be added to two of the existing blowers. Four additional fine bubble racks would be added to each of the existing basins to meet the additional air requirement. New aeration control based on the ORP, DO, and ammonia probes would be added to help maintain the nitrification process and DO levels.

Table 5-4 presents the estimated project cost to add a third SBR basin.

Table 5-4
Cost Estimate for Addition of Third SBR Basin

Description	Quantity	Units	Units Costs	Extended Costs
Taxes, Bonds, and Insurance at 8%	1	LS	$240,700.00	$240,700
Mobilization at 10%	1	LS	$300,900.00	$300,900
SBR Equipment	1	LS	$768,000.00	$768,000
VFD Drives	4	EA	$25,000.00	$100,000
Harmonic Filters	4	EA	$13,000.00	$52,000
New Blower Building	600	Sq. Ft	$400.00	$240,000
Excavation	2000	CY	$5.00	$10,000
Concrete	600	CY	$1,450.00	$870,000
Handrails	220	LF	$95.00	$20,900
Grating	465	Sq. Ft	$45.00	$20,925
Aeration Piping	209	LF	$103.00	$21,527
Reset Existing Diffuser Racks	12	Ea	$2,000.00	$24,000
Site Piping	1	LS	$50,000.00	$50,000
DO and ORP probes	2	EA	$1,500.00	$3,000
Ammonia probe	1	EA	$18,000.00	$18,000
TSS probe	1	EA	$2,800.00	$2,800
SCADA programming	1	LS	$40,000.00	$40,000
Electrical and Controls 25% of Equipment	1	%	$198,250.00	$198,250
Remove and Reset Fence	160	LF	$20.00	$3,200
Site Grading and Landscaping	1	LS	$25,000.00	$25,000
Subtotal				$3,009,202
15% CONTINGENCY				$451,400
SUBTOTAL				$3,460,602
ENGINEERING/CONSTRUCTION MANAGEMENT (20%)				$692,100
TOTAL				**$4,152,702**

5.5.2 Option 5C Modification of Existing Aeration and Process Control

This option does not expand the plant capacity but does make changes in the aeration and process control to improve nitrogen removal in the plant. The existing blowers are constant-speed blowers and consequently even when the dissolved oxygen level is being monitored and turning the blowers on and off in response to the DO level, the dissolved oxygen level is often much higher than the set point and cycles lower than the blower on set point as shown previously in Figure 3-8. Replacing two of the constant-speed blowers with VFD drives would allow much better control of the dissolved oxygen levels, result in less blower cycling, and a reduced energy requirements caused by providing too much aeration. The addition of ORP and ammonia probes would allow much better control of the nitrification and denitrification process in the basins. In this option, ammonia probes would be added to the existing DO and ORP probes currently installed in the SBR basins and DO and ORP probes would be added to each of the aerobic digesters.

In additional a Total Suspended Solids (TSS) probe would be added to the waste sludge line to track the concentration of biomass being wasted. The TSS probe would be coupled with the existing flowmeter to allow the mass of sludge being wasted to be calculated. The addition of the TSS probe would allow the operators to have better control of the process sludge age which is critical in controlling the nitrification process.

A similar situation exists for the digester blowers. Return flows from the digester and belt press represent a substantial ammonia load being returned to the SBR basins and modifying the digester operation to maximize nitrification and denitrification in the basins will improve the treatment process.

Table 5-5 lists the estimated cost to make the aeration improvements to the SBRs and digesters.

Table 5-5
Estimated Cost of Modifications to Existing SBRs and Digesters

Description	**Quantity**	**Units**	**Units Costs**	**Extended Costs**
Taxes, Bonds, and Insurance at 8%	1	LS	$4,160.00	$4,160.00
Mobilization at 10%	1	LS	$5,200.00	$5,200.00
VFD Drives 40 HP	2	EA	$15,000.00	$30,000.00
New 40 HP Motors	2	EA	$3,500.00	$7,000.00
DO and ORP Probes (digesters)	4	EA	$1,500.00	$6,000
Ammonia Probes (SBR basins)	2	EA	$18,000.00	$36,000
TSS Probe	1	EA	$3,000.00	$3,000
Harmonic Filters	4	EA	$13,000.00	$52,000
VFD Drives 100 HP	2	EA	$25,000.00	$50,000
New 100 HP Motors	2	EA	$8,000.00	$16,000
Programming	1	LS	$25,000.00	$25,000
Electrical and Controls (30% of equipment)	1	LS	$26,000.00	$26,000
Subtotal				$214,000
15% CONTINGENCY				$32,100
SUBTOTAL				$246,100
ENGINEERING/CONSTRUCTION MANAGEMENT (20%)				$49,200
TOTAL				**$295,300**

The existing SBR basins are not covered which has resulted in some freezing problems with the foam control system and occasional freezing of the decanters. Biological treatment basins are not typically covered but with the heavy snows and frequent cold temperatures, covering the basins would provide some benefit both in terms of less freezing problems and in safer working conditions for the operators. The cost and an evaluation of different covering methods was outside the scope of this study, but it is recommended the District consider undertaking a study to evaluate options to cover the basins.

5.5.3 Option 5D Algae Based Nutrient Removal (ABNR) Addition

This option consists of replacing the existing filtration process with a new process that has the ability to further reduce the nitrogen and phosphorous levels in the effluent and incorporates a microfiltration step for solids separation. The process is an emerging technology that consists of growing algae in a vertical arrangement of glass tubes and then using a microfiltration membrane to separate the algae. Figure 5-5 shows the vertical arrangement of glass tubes in the test facility located in Missoula.

The CLEARAS Company is developing the process and provided the following test data from several test sites throughout the nation and for sites located in Montana.

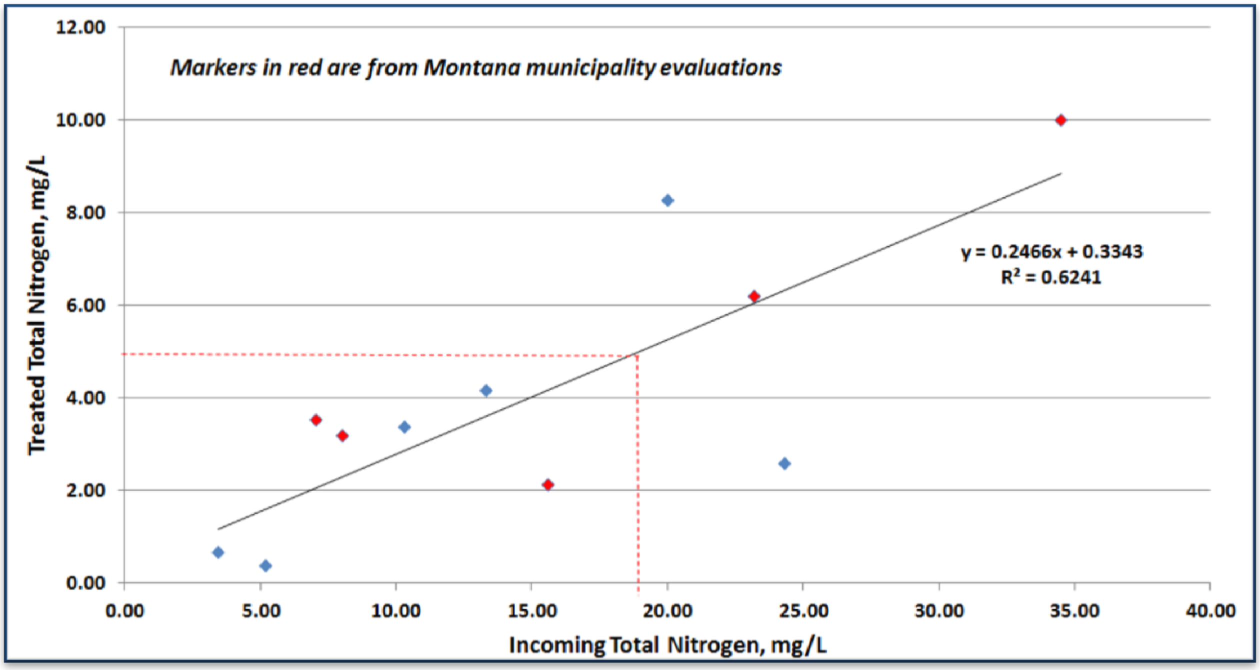
The tubes containing the algae biomass are housed in a greenhouse structure to allow the use of natural sunlight to promote algae growth. During darkness artificial lighting is used to maintain the process. A potential process layout for a system capable of treating a flow of 0.3 MGD is shown below

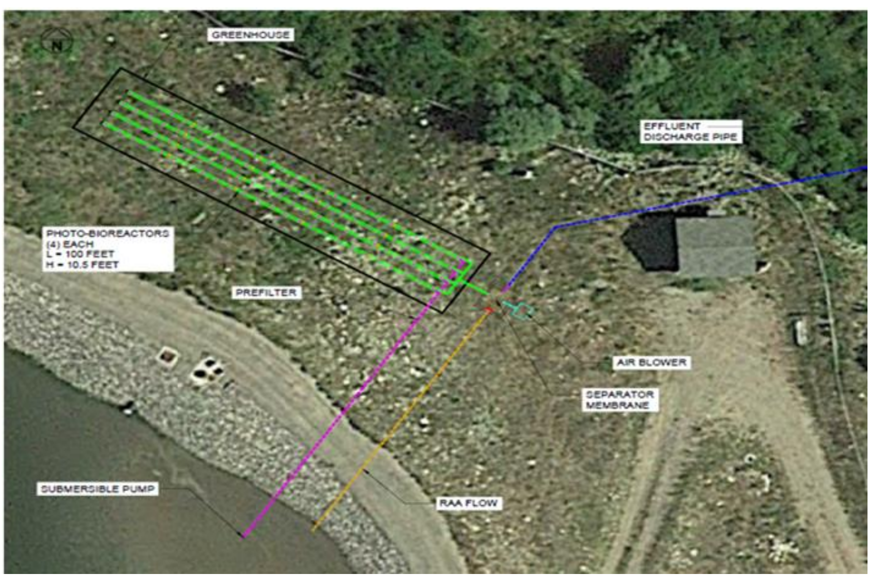
Figure 5-5 Algae Based Nutrient Removal System

Table 5-6 Algae Based Nutrient Removal Test Data
(data provided by CLEARAS Company)

Site	Category	EPA Region	TP		TN	
			RAW	TREATED	RAW	TREATED
1	Municipal	1	0.11	0.01	5.18	0.36
2	Municipal	4	2.56	0.03	3.42	0.64
3	Municipal	8	2.17	0.02	15.60	2.13
4	Municipal	8	3.01	0.01	24.30	2.55
5	Municipal	8	0.75	0.03	8.03	3.19
6	Municipal	5	0.77	0.02	10.30	3.36
7	Municipal	8	2.57	0.01	7.05	3.53
8	Municipal	8	2.02	0.04	13.30	4.13
9	Municipal	8	4.54	0.02	23.20	6.20
10	Municipal	10	0.18	0.03	20.00	8.24
11	Municipal	8	5.10	0.06	34.50	10.00
		Average	2.16	0.03	14.99	4.03
		Percent reduction	98.8%		73.1%	
		Count	11		11	

Figure 5-6
Nitrogen Reduction
(data provided by CLEARAS company)





Potential site layout for algae based nutrient removal system. The algae tubes are contained within a greenhouse structure to allow natural light promote the algae growth. At night artificial lighting is used.

The system is reported to have a cost of approximately $2.00 per gallon treated. Consequently as system sized for the current average day flow of 0.363 MGD would have an installed cost of $726,000. When piping, a small building to house the filtration membranes, and solids handling are included the installed cost is likely to be in the $1,000,000 to $1,500,000 range. The system is highly expandable by simply adding additional tubing and microfiltration. In order to evaluate this option in more detail, the vendor recommends running a Nutrient Recovery Test (NRT) which would consist of the vendor visiting the site to collect a sample that would be run through the test facility in Missoula to confirm the mass balances and potential nutrient removal capabilities. A key element in this alternative relates to the solids handling of the excess algae biomass produced. The vendor estimates the ABNR system will yield 15 pounds of algae per pound of nitrogen removed. For 0.4 MGD facility, reducing the nitrogen level from 10 mg/L-N to 3 mg/L-N, the estimated algae yield would be 350 pounds per day. Under the vendor's current business model, they collect the excess biomass and market the product which would relieve the District from that responsibility.

5.6 Alternative 6 Surface Discharge

The criteria for determining nonsignificant changes are contained in ARM 17.30.715 and for a surface discharge consists of five conditions. The criteria were evaluated in a prior report (2) but are summarized here for convenience.

1. *The increase or decrease in the mean monthly flow must be less than 15% or the change in the seven-day 10 year (7Q10) flow must be less than 10%.*

The 7Q10 flow for the Gallatin, above the confluence with the Middle Fork, was listed in the EIS as 138 cubic feet per second (CFS) (20). The allowable discharge while meeting the 10% increase criterion is 13.8 CFS (8.9 MGD). The estimated minimum mean monthly flow is 216 CFS which would allow a discharge of 32 CFS (20.6 MGD).

2. *Discharges containing carcinogenic parameters or parameters with a bioconcentration factor of greater than 300 must be less than or equal to the concentration in the* receiving water.

There are twenty two pollutants listed in Circular 7 that have bioconcentration factors greater than 300. The majority of the pollutants are pesticides such as Aldrin, Chlordane, Dieldrin, Heptachlor or organic compounds such as PCB's and Benzene. No test data was located for any of the bioconcentrating parameters in the main stem of the Gallatin or its tributaries. In addition, no test data is available from the District's wastewater plant for these parameters. It is expected, that if present in the Gallatin, the concentrations of any bioconcentration parameters would be very low. It is also expected that the treated wastewater would be very low in any bioconcentrating pollutants. Testing of both the Gallatin and the treated wastewater would have to be completed in order to determine if the nonsignificance criteria could be met.

3. *Discharges containing toxic parameters or nutrients must be less than or equal to the trigger values specified in Circular DEQ 7. If the trigger value is exceeded the change is still considered non significant if the resulting concentration outside of the mixing zone does not exceed 15% of the lowest applicable standards.*

On February 13, 2014 the Montana Board of Environmental Quality adopted base numeric nutrient standards that set new limits for nitrogen and phosphorous discharged to surface waters at levels designed to protect the beneficial uses and prevent exceedances of other surface water quality standards. The Big Sky area is in the Middle Rockies ecoregion which has in-stream nutrient standards of 30 µg/L total phosphorus and 300 µg/L total nitrogen. However, the limits only apply from July 1 to September 30 each year and consequently would not apply to a winter discharge. Instead of the numeric nutrient standards, the nondegradation criteria and narrative standards contain in the Water Quality Classification

Standards would apply to a winter discharge. The narrative standards have general prohibitions (ARM 17.30.637) which require a discharge to be free from any substance that will create an undesirable or nuisance condition. Narrative standards are applied when sufficient information does not yet exist to develop specific numeric standards.

It has been estimated in a prior study (2) that a discharge of 0.891 MGD could be discharged under this criteria.

4. *The change in water quality for any harmful parameter for which water quality standards have been adopted other than nitrogen, phosphorous and carcinogenic, bioconcentrating, or toxic parameters if the changes outside the mixing zone are less than 10% of the applicable standard and the existing water quality level is less than 40% of the standards.*

Circular DEQ7 contains the list of numeric water quality standards. No test data is available for the majority of the parameters listed in the circular so it not possible to evaluate this criterion until specific pollutants of concern are identified and tested.

5. *Changes in the quality of water for any parameter for which there are only narrative standards must not have a measurable effect on any existing or anticipated use or cause measurable changes in aquatic life or ecological integrity.*

The most applicable narrative standard for parameters that would have a measurable effect would concern the impact of nutrients on the receiving stream. Since the District would only be contemplating a winter discharge, increased algae growth is not anticipated as causing a measurable effect.

5.7 Alternative 7 Snowmaking

In this alternative, treated wastewater would be used to make snow as one part of the overall disposal capacity. With Class A-1 water, the snowmaking operation could be used on the ski runs or other areas where public access is not restricted. The wastewater would be highly treated and disinfected to protect the public health and to meet water quality criteria during melt conditions. An acre-foot of medium density snow has an equivalent water volume of approximately 146,000 gallons and consequently a large area is required to dispose of large volumes of reclaimed wastewater. In a previous study, a snowmelt model was developed to project the in-steam ammonia concentrations based on applying 250 million gallons of reclaimed water with an effluent ammonia concentration of 10.3 mg/L-N to 542 acres (2). It is expected that upgrades to the SBR process and operation control will be able to produce an effluent with a total nitrogen concentration of 5 mg/L- N of which approximately 0.5 mg/L will be in the form of ammonia-N, 1-2 mg/L in the form of nitrate-N and 1-2 mg/L as organic nitrogen-N. Table 5-7 shows the results of the model when an effluent with a total nitrogen concentration of 5 mg/L- N is applied. In this model, the background in-stream ammonia concentration is assumed to be 0.047 mg/L ammonia and as shown the projected impact to the stream due to ammonia from the melt water is minimal and well below the 0.01 mg/L-N nondegradation trigger value.

Table 5-7
Projected In-stream Ammonia Concentrations with Snowmaking and Improved Plant

	Volume of Snow Applied			**250 MG**					
Ammonia Concentration in Applied Snow				**0.5 mg/l**					
	Projected Stream Flow With Snowmaking CFS	Project Stream Flow Without Snowmaking CFS	Runoff due to snowmaking CFS	Recharge inches	Recharge Volume in application area acre-feet	Volume of Meltwater Acre-feet in this month	Cumulative meltwater volume	Resulting Ammonia Concentration in melt water mg/L-N	Instream Ammonia Concentration mg/L-N
Jan	4.45	4.45	0.00	0.00	0.0	0.0	0.0	0.047	0.047
Feb	4.42	4.42	0.00	0.00	0.0	0.0	0.0	0.047	0.047
Mar	5.10	5.10	0.00	0.00	0.0	0.0	0.0	0.047	0.047
Apr	17.82	17.35	0.47	7.19	324.5	352.5	352.5	0.049	0.047
May	79.05	76.24	2.81	2.06	93.1	265.7	618.2	0.065	0.048
Jun	80.51	77.57	2.94	2.32	104.7	279.8	898.0	0.020	0.046
Jul	24.96	24.11	0.85	0.48	21.8	74.2	972.1	0.020	0.046
Aug	10.72	10.72	0.00	0.80	36.1	36.1	1008.3	0.000	0.047
Sep	7.65	7.65	0.00	1.31	59.0	59.0	1067.3	0.000	0.047
Oct	7.51	7.51	0.00	0.00	0.0	0.0	1067.3	0.000	0.047
Nov	6.18	6.18	0.00	0.00	0.0	0.0	1067.3	0.000	0.047
Dec	5.51	5.51	0.00	0.00	0.0	0.0	1067.3	0.000	0.047

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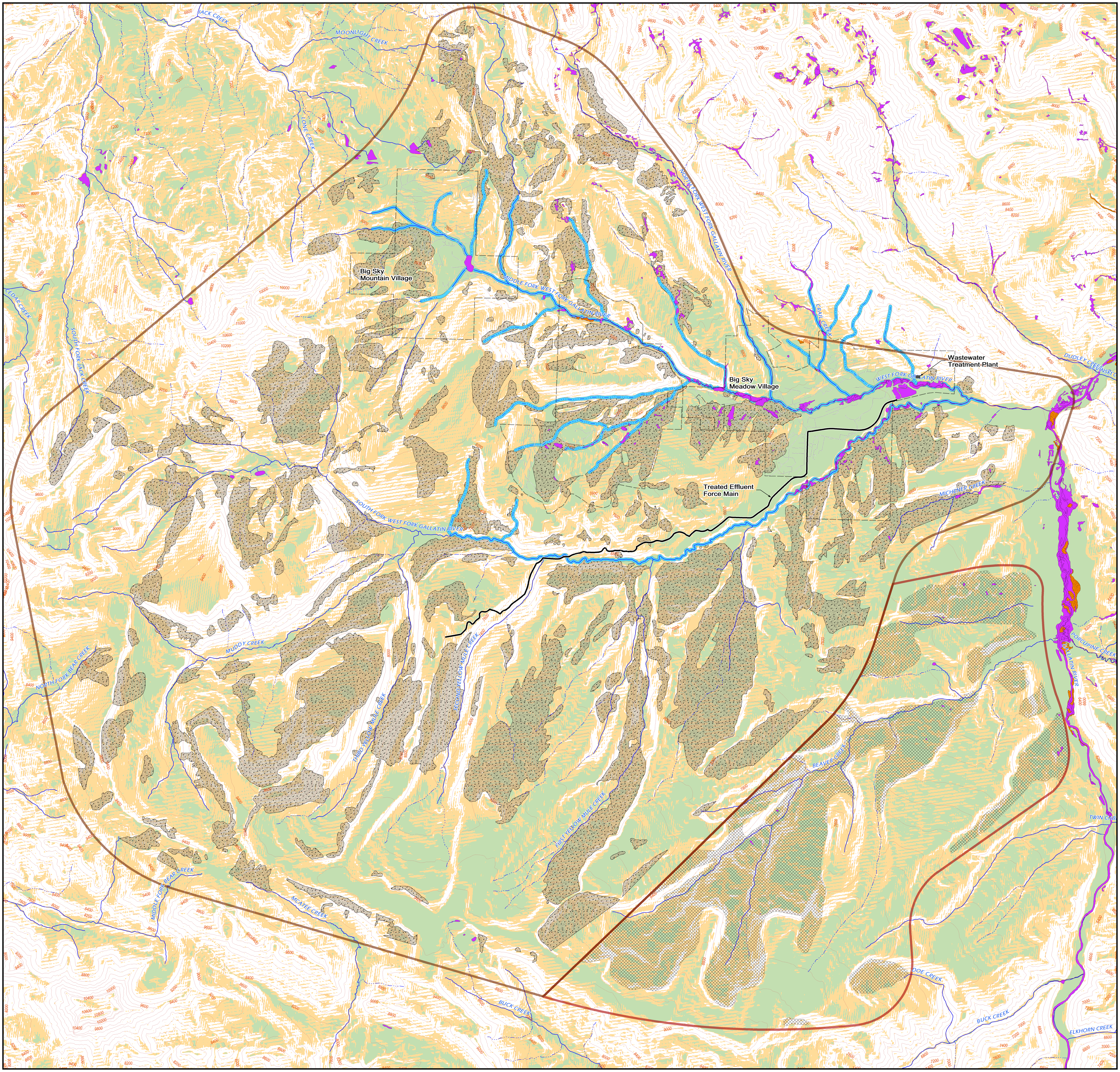
7.0 APPENDICES

Exhibit A Mapping of Land Characteristics for Subsurface Disposal



DOWL

2,000 1,000 0 2,000 4,000
Feet

LEGEND

- Treated Effluent Force Main
- Sewer Lines
- Wastewater District Boundary
- Area of Interest
- Wetlands
- Riparian
- Landslides
- Contacts - MBMG Code = Qls

Percent Slope

- 0 - 15%
- 15 - 35%
- >35%

100ft Hydrography Buffer

EXHIBIT A

BIG SKY SEWER AND WATER STUDY
BIG SKY, MONTANA

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Exhibit B Potential Application Sites


MADISON CO.
GALLATIN CO.
Exhibit B - Potential Water Application Sites
Big Sky Sewer and Water Study
Big Sky, MT
Legend
Restrictions
Potential Areas
Landslide
Steep Slopes
Hydrologic Buffer
Wetlands
Application Site Ranking
Good Potential
Potential Site
Maybe
Already Receives Treated Water
Probably Not
Treated Effluent Force Main
Sewer Lines
DOWL
0
0.25
0.5
Miles
N
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