WATER RESOURCES MASTER PLAN

TOWN OF BUENA VISTA, COLORADO

OCTOBER 2014

PREPARED BY:

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1.0 EXECUTIVE SUMMARY

RG and Associates, LLC (RGA) was retained to update the town's August 2006 Water Resources Master Plan. Updates, changes and additions are located throughout the entirety of the master plan. RGA completed this update with the assistance of the Town of Buena Vista staff and public works division. The planning horizon for this master plan update is generally considered to be 20-years.

The 2014 update includes, but is not limited to, the following:

- Current and projected population
- Water system demands
- Narrative of distribution system process
- Distribution system capital project recommendations
- Narrative of treatment system process
- Treatment system capital project recommendations
- Storage requirement calculations
- Water rate study and water rate model

This document was developed for the use of the town in its planning process and evaluates both current and projected future conditions. It is intended to be a working document that is used as a guideline for planning decisions and represents a best approximation of future conditions.

1.1 DEMOGRAPHICS

- 2012 Census Bureau Population Estimate: 2,662. This equates to approximately 1,618 SFE.
- Historically, in the last 15 to 20 years, population growth has been sporadic and has averaged around 1.2% per year. The town currently appears to be in a steadier, more consistent pattern that will likely continue for some time.
- Analyses suggest using 1.2% annual growth rate for planning purposes.
- The distribution system is divided into three zones: 1) Upper Zone, 2) Lower Zone and 3) Ivy League Zone.
- The Lower Zone has approximately 1,255 existing SFEs. The Lower Zone has approximately 608 remaining SFE until full buildout (full buildout being the maximum amount of development in the zone).
- The Upper Zone has approximately 363 existing SFEs. The Upper Zone has approximately 1415 remaining SFE until full buildout.
- The Ivy League contains approximately 40 SFEs and is at full buildout.

1.2 DEMANDS

To determine current water usage and estimate future demand, RGA obtained billing records for all taps served by the Town of Buena Vista for the 2012 calendar year. RGA then totaled the water usage for all records and compiled a model based on analyzing every 10th customer record. Every 10th customer was used to ensure the amount of data being analyzed was not unwieldy. The water usage analysis resulted in a current Average Day Demand and a current Maximum Day Demand. These are as follows:

- Current maximum day demand: 0.94 MGD.
- Current average day demand: 0.47 MGD.
- Currently average 1.65 persons per SFE.
- Currently use average of 293 gpd per SFE.
- Peaking Factor Used for Max Day / Average Day: 2.6

1.3 WATER TREATMENT & PRODUCTION

There are three water sources which are an Infiltration Gallery, a groundwater well (Well No. 2) and a surface water intake off of Cottonwood Creek. The current demand is met using just the Infiltration Gallery and Well #2. The net production of both of the used sources is about 1.4 MGD. A future well, Well Number 3, could provide water to the Upper Zone Tank.

Surface water from Cottonwood Creek could be treated by an existing unused water treatment plant. As currently configured, the treatment plant has a nominal capacity of 1.5 MGD.

It is currently estimated that the town could face $5M (or more) in capital costs to upgrade and expand its production/treatment facilities over the next 15 years, or so, to meet growing water demands and increasingly stringent water quality standards. This will significantly impact both tap fees and water use rates. This master plan details costs to update the existing facility, retrofit the existing facility with membranes and to completely replace the existing facility. This was done to give the town several options and ranges of costs to evaluate.

1.4 WATER RIGHTS

Through the analysis completed in the update of this master plan, it was determined that the town's existing water rights are sufficient for the foreseeable future. However, should growth and development patterns change, water rights will need to be reevaluated.

1.5 DISTRIBUTION SYSTEM

No upgrades to existing pipes appear necessary to maintain current service conditions. It is recommended that the town should begin funding an annual pipe replacement fund to replace aging infrastructure (there are some pipes in the system that are over 50 years old) and to install pipes in streets where none currently exist. It is also recommended that the town standardize their system with minimum 8" pipe. This means that all 4" and 6" pipe should be upsized to 8" when they are replaced, and all new developments should only use 8" or greater.

Preliminary investigations indicate that it is feasible to convert the Ivy League subdivision to gravity service off the Upper Zone tank. The town will need to perform a more detailed analysis during preliminary engineering to confirm service conditions given the chosen routing and flow requirements.

Finally, the majority of the future improvements herein recommended should be funded by development.

1.6 STORAGE

Storage requirements vary depending on the governing agency, however, the most common is to ensure enough storage is provided to satisfy the Maximum Day Demand for each SFE served by the tank plus the required fire flow. In this analysis the required fire flow is 3,500 gallons per minute for three hours and the Maximum Day Demand per SFE is 580 gallons per day. There are three existing tanks in the town's water supply system.

- The upper zone currently has one (1) 0.75 million gallon storage tank. According to the design requirements set forth previously, this can serve about 206 SFEs.
- The lower zone currently has one (1) 1.5 million gallon storage tank. According to the design requirements set forth previously, this can serve about 1,320 SFEs.
- The Ivy League, which is out of the current town's service area, has one (1) 0.27 million gallon storage tank to serve its full buildout population of approximately 40 SFEs. Note that all water passes through the Ivy League tank prior to flowing by gravity to the Lower Zone.
- It is recommended that an additional 0.75 million gallon tank to serve the Upper Zone.

1.7 SECONDARY WATER SUPPLY

Non-potable water use (i.e. irrigation) typically accounts for an estimated 50% of the total water demand. This significantly increases infrastructure costs, treatment costs and long-term operation costs.

The town should investigate conversion of the town ball fields and parks to dedicated non-potable well systems. The town should also encourage new developments to install, operate, and maintain their own non-potable irrigation water systems wherever this is feasible.

1.8 SUMMARY OF MASTER PLAN RECOMMENDATIONS

The following is a summary of recommendations made throughout this master plan as well as the associated dates each recommended task should be completed by. Detailed descriptions of each recommendations are included throughout the report.

1.8.1 Fire Flow Requirements

- It is recommended that a new ISO fire flow study be completed by 2016.

1.8.2 Water Rights

- As is explained in this master plan, additional water rights are not necessary for the 20-year planning horizon. If population growth accelerates beyond the current projections it is recommended that the town study the viability of constructing a new municipal well tributary to the Arkansas River.

- As described above, additional water rights, or raw water storage are not necessary, however, if population growth accelerates beyond the current projections, it is recommended that the town investigate alternative storage opportunities on Cottonwood Creek.

- The town's existing water rights are projected to be adequate to serve through the 20-year planning horizon of this master plan. If population projections should change, however, three trigger points have been developed to assist in timing new water rights acquisitions. These trigger points are as follows:

    - Trigger Point #1 occurs when the current MDD is eighty percent of 3.88 CFS (amount currently available for diversion at town intake and infiltration gallery), or 3.1 CFS. Given the town's current water rights portfolio, its best option at Trigger Point #1 is to prove dry-up of the land irrigated by the Leesmeagh as outlined in Case 83CW88 and start diverting the Leesmeagh water at the town Intake and Infiltration Gallery. This will bring the amount of usable water rights to 4.98 CFS.

    - Trigger Point #2 occurs when the current MDD is eighty percent of 4.98 CFS (amount available for diversion after Leesmeagh dry-up), or 4.0 CFS. Assuming that the town is limited to the existing water rights portfolio, the town's best option at Trigger Point #2 is to prove dry-up of the land irrigated by the Gorrel as outlined in Case 83CW88 and start diverting the Gorrel water at the town intake and infiltration gallery. This will bring the amount of usable water rights to 5.88 CFS.

    - Trigger Point #3 occurs when the current MDD is eighty percent of 5.88 CFS (amount available for diversion after Leesmeagh and Gorrel dry-up), or 4.7 CFS. At this point, additional demand will exceed the existing water rights currently available to the town. Assuming that the town is able to obtain additional water rights prior to Trigger Point #3, then the town's best option at Trigger Point #3 is to transfer the point of diversion of said water rights to the town Intake and infiltration gallery or other applicable point of diversion, or use the consumptive use from senior irrigation rights to augment depletions from wells and/or junior surface diversions.

1.8.3 Water Treatment

- Based on a projected population growth of 1.2% per year, it is expected that the water treatment plant will need to be brought back online in 2026. There are several alternatives modernizing the water treatment plant prior to bringing it back online that are discussed in this master plan. These are as follows:

    - Rehabilitate the existing treatment plant and maintain a similar operational schematic with some improvements.

- Rehabilitate the existing water treatment plant with membrane filtration to replace the existing multi-media filters.
- Completely replace the existing water treatment plant with a new state of the art membrane filtration system.

- Prior to re-starting the existing water treatment plant or starting up a new treatment plant, it is recommended that the existing pre-sedimentation pond be lined with a synthetic liner.
- Prior to re-starting the existing water treatment plant or starting up a new treatment plant, it is recommended that the existing raw water intake structure be replaced.

A summary of cost estimates for each potential project is shown in Table 1.

Table 1- Summary of WTP Upgrade/Replacement Alternatives

Alternative	Total Project Cost	Annual Payment*	Project Cost w/ Interest
EXISTING WATER TREATMENT PLANT IMPROVEMENTS AND RE-COMMISIONING	$2,300,097	$169,245	$3,384,903
3-MGD MEMBRANE FILTER RETROFIT	$5,612,097	$412,948	$8,258,958
ULTRA-FILTRATION WATER TREATMENT PLANT (3 MGD)	$6,104,897	$449,209	$8,984,180
*Assumes 20 year loan @ 2.7%.			

1.8.4 Distribution System

- It is recommended that the Ivy League be incorporated into the Upper Zone by 2015. This would allow the Ivy League Subdivision to operate on gravity from the Upper Zone Tank rather than have all pressure supplied by pumps, and have more reliable fire flow.
- After conversion of the Ivy League Subdivision to the Upper Zone, it is recommended that the Ivy League Booster Pump Station be re-purposed to pump from the Ivy League tank to the Upper Zone Tank. This would provide a backup pumping system if the Westmoore Booster Pump Station was unavailable. This project should be completed in tandem with the Ivy League conversion to the Upper Zone in 2015.
- It is recommended that the town fund an annual water main replacement project to replace old and deteriorating lines. The annual replacement projects should start in 2015.
- It is recommended that the town should also encourage new developments to install, operate, and maintain their own non-potable irrigation water systems wherever this is feasible.

A summary of cost estimates for each potential project is shown in xxx.

Table 2 - Summary of Distribution Project Costs

Alternative	Total Project Cost
ANNUAL WATER MAIN REPLACEMENT PROGRAM	$85,400
IVY LEAGUE CONVERSION TO UPPER ZONE	$363,680

1.8.5 Water Storage

- It is recommended that the town construct an additional 750,000 gallon finished water storage tank adjacent to the existing Upper Zone Tank. This project should be completed as soon as possible as the Upper Zone does not have sufficient storage capacity. The new water storage tank is expected to cost approximately $950,000.

1.8.6 Watershed Protection

- The town's watershed protection plan should be modified to include the entire watershed with a source water protection plan developed through the CDPHE process.
- Require regular maintenance inspections of OWTS systems within critical protection zones that are filed with the town.
- The town should foster the development of a Watershed Stakeholders Group.
- The Watershed Protection Plan should be expanded to include proactive water quality and quantity monitoring.

1.8.7 System Rules and Regulations

- The town should adopt a more defined SFE apportioning schedule to more accurately assess system impacts and fee assessments.
- The town should raise their water service fees now to account for known future expenditures.
- The town should conduct tap fee and service rate studies.
- The town should annually review water rights cash-in-lieu fees to ensure that fees are sufficient for current market conditions.

2.0 INTRODUCTION AND OVERVIEW

2.1 PROJECT SCOPE

RG and Associates, LLC (RGA) was retained to update the August 2006 Water Resources Master Plan. Updates, changes and additions are located throughout the entirety of the master plan. RGA completed this update with the assistance of the Town of Buena Vista staff and public works division. The planning horizon for this master plan update is generally considered to be 20-years.

This report covers current water demands, the distribution system, water storage, treatment, water rights in a general sense, watershed protection and offers recommendations for future upgrades and enhancements that will likely be necessary to meet future growth.

Numerous resources were used to develop this master plan. These resources included meetings and interviews, review of previous town studies and planning documents, evaluation of system data, and use of the previously developed water system computer model. Some of the data sources utilized include:

- Town of Buena Vista Public Works Staff
- Town of Buena Vista 2008 Comprehensive Plan
- State Demographer
- Chaffee County
- Colorado Department of Public Health and Environment
- US EPA
- US Census Bureau

2.2 PREVIOUS STUDIES AND REPORTS

The following reports were reviewed to obtain background system information:

- **Town of Buena Vista Comprehensive Plan, 2008**

  This document was developed to be the guiding document for town growth and expansion. The document represents the aggregated goals of the town and its residents as to how, what and where the town develops. Important water facts gleaned from the report are a summary of water rights and an overview of the water distribution system.

- **Upper Zone Water System Master Plan, August 1994**

  This report was the original report outlining how to provide service above the historic town Zone tank fed directly from the water treatment plant and infiltration gallery. This report details the main components of the historic system, reporting such information as blue-line elevation and estimated irrigation uses (including the golf course).

Water demand estimates based on historic data dictated using a value of 150 gallons per day (GPD) per person for average day use, with a peak day multiplier of 3.67 for Upper Zone demand calculations. The report ultimately recommends construction of a 1.25 MG tank at elevation 8235, served from a new pump station to be constructed in the Westmoor Subdivision.

- **Buena Vista WTP Comprehensive Performance Evaluation, November 2001**

  This is a summary report of a plant performance evaluation conducted in conjunction with the State of Colorado. Some of the important information contained in this report is:

  1. The infiltration gallery is not under the influence of surface water by MPA analysis.
  2. The infiltration gallery and the groundwater well have a combined capacity of about 1.4 MGD.
  3. The water treatment plant can treat about 1.5 MGD from Cottonwood Creek.
  4. WTP process flow diagram.
  5. Details of unit processes operation and design.

- **Report on the Water Treatment Plant, Town of Buena Vista, September 1995**

  This report summarizes a plant evaluation conducted in response to finished water turbidity violations. The result of this evaluation was the implementation of several of the recommendations. The report also evaluates several other treatment options that were not implemented but may be of interest in future planning. The CPE conducted in 2001 supersedes this report regarding plant performance.

- **Town of Buena Vista Water Workshop, Nov. 6, 1999**

  This report describes the town's water rights portfolio and the town's water rights, in order of seniority.

- **Town of Buena Vista Water Workshop, June 24, 2004**

  This document supersedes the 1999 Workshop. It is a synopsis of the town's water rights portfolio as of the date of the report. It lists current rights, historic demands, current status and viability of the town's portfolio, and lists future needs. Factors of Use (=estimated # of available taps from water rights, based on 2001 water use patterns, divided by the existing taps in 2001) are reported as 1.9 for current usage, 2.4 cumulative with the addition of the Leesmeagh rights, and 2.9 cumulative with the Gorrel rights.

2.3 RAW WATER SYSTEM OVERVIEW

The water supply for the Town of Buena Vista is comprised of two sources. The primary source is North Cottonwood Creek. The second source is a 100-ft deep groundwater well (Well No. 2) located near the water treatment plant. The town also has a second smaller well that serves only the rodeo grounds. Raw water from Cottonwood Creek can either flow directly to the water treatment plant through a ditch system. Water from North Cottonwood Creek can be applied to an infiltration gallery in a meadow to the west of the water treatment plant.

Raw water collected by the infiltration gallery is considered to be groundwater not under the influence of surface water by CDPHE and therefore requires less rigorous treatment than does surface water. The infiltration gallery is the primary source of water for the town and additional raw water needs beyond the production capabilities of the gallery are augmented by Well No. 2.

The infiltration gallery and Well No. 2 have a maximum yield of about 1.4 million gallons per day. The existing water treatment plant, which has been decommissioned and is not in use at the present time, has a maximum production capacity of 1.5 MGD.

2.4 TREATMENT SYSTEM OVERVIEW

The Buena Vista WTP (BVWTP) is a direct filtration plant with pre-sedimentation that treats water from Cottonwood Creek. The plant has a nominal capacity of 1.5 MGD based on a 4-GPM/sf filter loading rate with both filters running and about 10% daily production loss due to backwash water usage and time off-line for backwashing. The plant has not been used for production since 1998. At present, water treatment consists solely of chlorinating and injecting corrosion inhibitors to the infiltration gallery water and chlorinating well water. The town can provide water in power outages as there is a standby generator for the chlorination system.

2.5 WATER DISTRIBUTION SYSTEM OVERVIEW

The Town of Buena Vista water distribution system is comprised of two main gravity zones and one constant pressure zone. The two gravity zones are the Lower Zone and the Upper Zone. The constant pressure zone is fed from the Ivy League Pump Station and is therefore referred to as the Ivy League Zone. Water is pumped from the Lower Zone to the Upper Zone through the Westmoor Booster Pump Station.

3.0 SERVICE AREA DEMOGRAPHICS

3.1 EXISTING SERVICE AREA

The town has an adopted water service boundary that encompasses all incorporated lands. The town isalso committed to serve the Ivy League subdivision which is inside of the service district boundary but outside of the town boundary. All other areas that are currently outside service boundary that desire water service in future will be required to annex into the town and will have to either bring additional water rights or pay impact fees. The water system service area (town boundary) is shown in Figure 1. It is important to note that the town's existing water rights portfolio serves only the areas in the town limits. Developments outside of the town limits must provide water rights or cash in lieu of water rights.

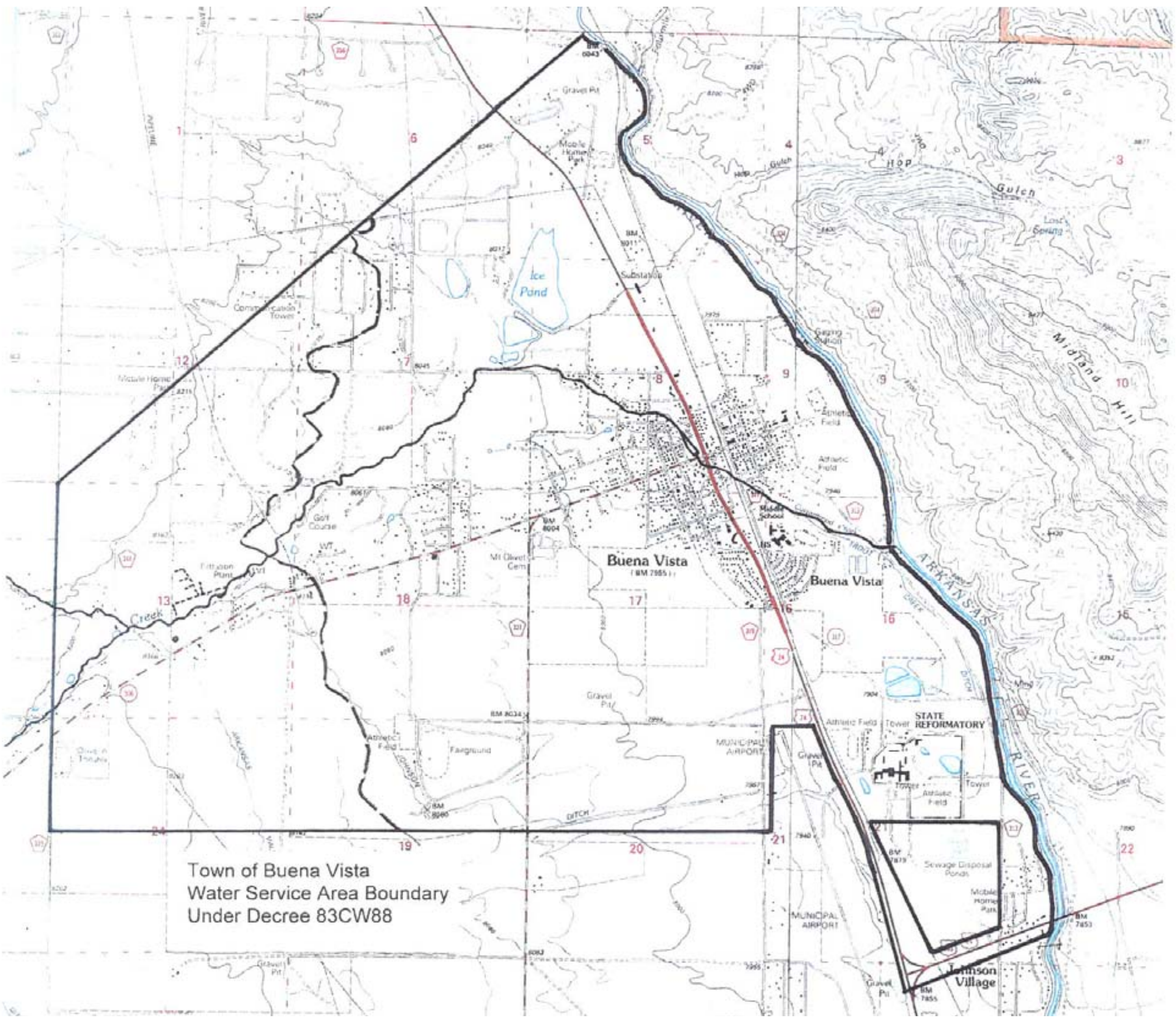

Figure 1 - Buena Vista Water Service Area

One significant planning and impact issue that the town faces is potential expansion beyond the current service boundary from future annexations. This is because the Town of Buena Vista is located in an area that lends itself to boundary expansion, constrained only by the Arkansas River and the

Collegiate Peaks. There has been discussion of annexing the area to the south, including Johnson's Village. These decisions could significantly impact future water needs projections, but are not specifically addressed in this document.

Up until 1998 the town did not use a traditional Single Family Equivalent (SFE) system and each user was billed a flat rate regardless of home size, number of plumbing fixtures, or actual use. When meters were installed in 1998, the billing system was modified to charge a tap fee based on the size of the meter (typically 3/4" or 1" depending on needs of the service) and user fees where switched from a flat fee system to a system that charges per gallon used from flow meter readings. Through analyzing water use data, it is estimated that each SFE uses an average of 293 gallons per day.

3.2 POPULATION GROWTH ESTIMATES

According to the U.S. Census Bureau, the population for the Town of Buena Vista was 2,662 in 2012. Due to the generally poor economy over the previous several years population growth has been relatively slow but is anticipated to increase as the economy grows. Population growth in the town has been sporadic since the 1980s and has been around 1.2% per year.

Future population growth trends predicted by the State of Colorado Demographers Office and Chaffee County vary. The Colorado State Demographers Office estimates a statewide population growth of around 1.7% annually. Chaffee County estimates a population growth of 2.3% annually for the next 20-25 years. The 2008 Town of Buena Vista Comprehensive Plan utilizes a continued historical population growth rate of 1.2% annually which predicts a 2030 Town of Buena Vista population of 3,087.

For the purposes of this master plan update, rate study and rate model an annual population growth of 1.2%, as projected in the town's comprehensive plan, was utilized. With this 1.2% annual growth rate from the base 2012 census population of 2,662, the predicted 20-year or 2034 Town of Buena Vista population would be 3,028. This population will be utilized for planning purposes throughout the master plan. The rate model provided by RGA is interactive however, so should evidence of a change in growth trends be witnessed, the model can be easily updated.

Population and growth trends assist in predicting future treatment needs, additional storage requirements and predicting the need to acquire additional water rights. However, a more convenient method to express current water use and determine future water demand is the SFE method.

The SFE method equates water use of a typical single family residential structure with a ¾" tap as one (1) SFE to tap sizes above ¾", (1", 1.5", 2", etc.). These are calculated based on the ratio of the area of the tap area to the area of a ¾" tap. RGA has calculated that the Town of Buena Vista had 1,618 SFEs in 2012 which equates to approximately 1.65 persons per SFE. Applying the ratio of 1.65 SFEs per person to the projected 2034 population results in a predicted 1,835 SFEs by the year 2034. A summary of population and SFE growth estimates from 2012 through 2034 is shown in Table 3.

Table 3 - Population and SFE Growth Predictions

Town Population and SFE Growth		
Year	Population	SFE
2012	2,662	1,618
2013	2,675	1,621
2014	2,691	1,631
2015	2,708	1,641
2016	2,725	1,651
2017	2,741	1,662
2018	2,758	1,672
2019	2,775	1,682
2020	2,792	1,692
2021	2,809	1,702
2022	2,826	1,713
2023	2,843	1,723
2024	2,860	1,733
2025	2,876	1,743
2026	2,893	1,754
2027	2,910	1,764
2028	2,927	1,774
2029	2,944	1,784
2030	2,961	1,794
2031	2,978	1,805
2032	2,995	1,815
2033	3,011	1,825
2034	3,028	1,835
Note: Assumes 1.6 persons per SFE.		

There are approximately 1,255 existing SFEs in the Lower Zone and approximately 363 existing SFEs in the Upper Zone. There are approximately 40 SFEs in the Ivy League Zone which is at full build-out. It is estimated that the full build-out potential for the lower zone is 1,863 SFE and 1,780 SFE for the upper zone. It is unknown how future development will be split between the two zones at this time. Table 4 and Table 5 show the lower zone and upper zone population growth respectively and assumes that growth is split evenly between the two zones.

Table 4 - Lower Zone Population and Growth Estimates

Lower Zone Population and SFE Growth		
Year	Population	SFE
2012	2,008	1,255
2013	2,011	1,257
2014	2,019	1,262
2015	2,027	1,267
2016	2,035	1,272
2017	2,043	1,277
2018	2,051	1,282
2019	2,059	1,287
2020	2,067	1,292
2021	2,076	1,297
2022	2,084	1,302
2023	2,092	1,307
2024	2,100	1,313
2025	2,108	1,318
2026	2,116	1,323
2027	2,125	1,328
2028	2,133	1,333
2029	2,141	1,338
2030	2,149	1,343
2031	2,157	1,348
2032	2,165	1,353
2033	2,174	1,359
2034	2,182	1,364
Note: Assumes 1.6 persons per SFE.		

Table 5 - Upper Zone Population and Growth Estimates

Upper Zone Population and SFE Growth		
Year	Population	SFE
2012	654	363
2013	664	365
2014	673	370
2015	681	375
2016	690	380
2017	699	385
2018	707	390
2019	716	395
2020	725	400
2021	733	405
2022	742	410
2023	751	415
2024	760	421
2025	768	426
2026	777	431
2027	786	436
2028	794	441
2029	803	446
2030	812	451
2031	820	456
2032	829	461
2033	838	467
2034	846	472
Note: Assumes 1.6 persons per SFE.		

3.3 ESTIMATED FUTURE DEVELOPMENT BY AREA

Note that the estimated future development section remains largely unchanged from the original 2006 report as the development areas studied remain unchanged. Development densities for each area were established in conjunction with town staff. The probable development areas and allotted densities are shown on the map attached as Appendix E of this report. A summary of the project development areas is presented in Table 6 and Table 7. It should be noted that exact densities and SFE assessments may change from what is shown, but the overall impact from minor deviations is expected to have minimal effect on infrastructure; large deviations will require re-evaluation.

The listed developments shown in Table 6 and Table 7 are projected to ultimately add 1,533 SFE. These developments will more than double the existing SFE service that the town currently has. Using the previously established 1.2% annual growth rate, and assuming that the development rate is equal to the growth rate, it will take approximately 26 years to reach this size.

Table 6 - Upper Zone Potential Development Areas

Upper Zone Development		
Name	**No. Units**	**SFE**
Southard (south)	300	285
Sunset Vista III	50	48
Sunset Vista IV*	274	260
Meadow Ridge II	16	15
Unnamed Upper Zone	225	214
Other Residential Infill	50	48
Total	**915**	**870**

Note: Residential SFE projections assume average 4:1 ratio of single family to townhome = 0.95 SFE on average.

Table 7 - Lower Zone Potential Development Areas

Lower Zone Development		
Name	**No. Units**	**SFE**
Crossman	94	89
South Main Addition*	400	380
College Heights Area	50	48
Other Residential Infill	75	71
Industrial Park	50	50
Other Commercial	25	25
Total	**694**	**663**

*Portions of these developments are already approved

Note: Residential SFE projections assume average 4:1 ratio of single family to townhome = 0.95 SFE on average.

4.0 WATER SYSTEM DEMANDS

4.1 EXISTING DEMAND

There are three demand conditions that are used for analyzing domestic water systems. The first is Average Daily Demand (ADD), which is equivalent to the total year production divided by 365 days. The second is Maximum Day Demand (MDD), which is equal to the average day demand during the maximum month of demand in a year. The third demand is the Peak Hour Flow (PHF), which is defined as the peak flow during any 24-hour period. Irrigation demands during the hottest part of the irrigation season usually cause PHF conditions. The MDD is utilized primarily in the analysis of treatment and pumping facilities. PHF is used for waterline sizing and modeling and fire flow.

To analyze all demand conditions, billing records were reviewed for the 2012 calendar year. 2012 data was used as a baseline in this report as it is the latest year for U.S. Census data. Billing records were broken down for business, residential, school and government (including irrigation). Table 8 shows the results of the billing analysis.

Table 8 - Existing Billing and Demand

Month	Population	SFE	Residential	Business	School and Gov.	Total (Gal per Month)
January	2,662	1,618	4,429,000	2,278,000	161,000	6,868,000
February	2,662	1,618	3,726,000	1,806,000	190,000	5,722,000
March	2,662	1,618	4,156,000	1,759,000	201,000	6,116,000
April	2,662	1,618	9,198,003	2,817,000	2,339,000	14,354,003
May	2,662	1,618	10,828,900	2,952,000	2,537,000	16,317,900
June	2,662	1,618	18,134,002	5,008,000	3,448,000	26,590,002
July	2,662	1,618	14,032,000	4,891,000	3,490,000	22,413,000
August	2,662	1,618	21,158,000	4,998,000	2,946,000	29,102,000
September	2,662	1,618	11,854,000	4,325,000	3,148,000	19,327,000
October	2,662	1,618	7,786,000	3,463,000	2,241,000	13,490,000
November	2,662	1,618	4,319,000	2,246,000	193,000	6,758,000
December	2,662	1,618	4,282,000	2,017,000	257,000	6,556,000
Totals	**-**	**-**	**113,902,905**	**38,560,000**	**21,151,000**	**173,613,905**

This data can be further broken down as shown in Table 9 to calculate the total gallon per day usage and the calculated gallons per day per SFE based on an existing 1,618 SFEs.

Table 9 – Existing Demand Breakdown

Month	SFE	Gal per Day	Gal per Day per SFE
January	1,618	221,548	137
February	1,618	204,357	126
March	1,618	197,290	122
April	1,618	478,467	296
May	1,618	526,384	325
June	1,618	886,333	548
July	1,618	723,000	447
August	1,618	938,774	580
September	1,618	644,233	398
October	1,618	435,161	269
November	1,618	225,267	139
December	1,618	211,484	131
Average	**-**	**474,358**	**293**

The average day demand is 474,358 gallons per day or 293 gallons per day per SFE. Using the peaking factor calculated in previous master planning document of a 2.6 time the average day demand, the peak hour demand is calculated to be 1,242,818 gallons per day or 767 gallons per day per SFE. The maximum month demand occurs in August and is 886,333 gallons per day or 580 gallons per day per SFE. A summary of the average day, maximum month and peak hour demands is shown in Table 10.

Table 10 – Summary of Existing Demands

Criteria	Demand (GPD)	Demand (GPD/SFE)
Average Day Demand	474,358	293
Maximum Day Demand	938,774	580
Peak Hour Demand	1,233,332	762

4.2 FUTURE DEMANDS

Town growth at the predicted rate of 1.2% per year will add roughly 217 SFEs over the next 20 years. To calculate increased demands which will be seen as a result of population growth, the existing average gallons per SFE per day can be used and multiplied by the estimated SFE each year. As shown in Table 11 the estimated average day demand future demand in 2034 is 537,955 gpd for a total of 1,835 SFEs.

Table 11 - Future Average Day Demand Projection

Year	Population	SFE	Average Day Demand (GPD)
2012	2,662	1,618	474,358
2013	2,675	1,621	475,300
2014	2,691	1,631	478,181
2015	2,708	1,641	481,090
2016	2,724	1,651	484,029
2017	2,741	1,661	486,996
2018	2,758	1,671	489,994
2019	2,775	1,682	492,992
2020	2,791	1,692	495,989
2021	2,808	1,702	498,987
2022	2,825	1,712	501,984
2023	2,842	1,722	504,982
2024	2,859	1,733	507,979
2025	2,876	1,743	510,977
2026	2,893	1,753	513,975
2027	2,910	1,763	516,972
2028	2,926	1,774	519,970
2029	2,943	1,784	522,967
2030	2,960	1,794	525,965
2031	2,977	1,804	528,962
2032	2,994	1,814	531,960
2033	3,011	1,825	534,958
2034	3,028	1,835	537,955

Note: Average day demand based off of historical average day demand of 292 gpd per SFE increased proportional to SFE growth.

The maximum day and peak hour demand can also be calculated in a similar fashion based on the 2012 maximum day and peak hour per SFE values multiplied by the number of SFEs predicted in 2034. Table 12 shows the maximum month and peak hour demands through 2034.

Table 12 - Future Max Day and Peak Hour Demand Projection

Year	Population	SFE	Max Day Demand (GPD)	Peak Hour Demand (GPD)
2012	2,662	1,618	938,774	1,233,332
2013	2,675	1,621	940,638	1,235,780
2014	2,691	1,631	946,339	1,243,270
2015	2,708	1,641	952,097	1,250,834
2016	2,724	1,651	957,912	1,258,474
2017	2,741	1,661	963,786	1,266,191
2018	2,758	1,671	969,718	1,273,984
2019	2,775	1,682	975,650	1,281,778
2020	2,791	1,692	981,583	1,289,572
2021	2,808	1,702	987,515	1,297,365
2022	2,825	1,712	993,447	1,305,159
2023	2,842	1,722	999,379	1,312,953
2024	2,859	1,733	1,005,312	1,320,746
2025	2,876	1,743	1,011,244	1,328,540
2026	2,893	1,753	1,017,176	1,336,334
2027	2,910	1,763	1,023,109	1,344,127
2028	2,926	1,774	1,029,041	1,351,921
2029	2,943	1,784	1,034,973	1,359,715
2030	2,960	1,794	1,040,906	1,367,509
2031	2,977	1,804	1,046,838	1,375,302
2032	2,994	1,814	1,052,770	1,383,096
2033	3,011	1,825	1,058,703	1,390,890
2034	3,028	1,835	1,064,635	1,398,683

Note: Max day demand based off of historical max month demand of 580 gpd per SFE increased proportional to SFE growth. Peak hour demand based off of historical peak hour demand of 768 gpd per SFE.

In summary, the estimated 2034 average day, max month and peak hour demands are shown in Table 13.

Table 13 - Future Demand Summary

Criteria	Demand (GPD)	Demand (GPD/SFE)
Average Day Demand	537,955	293
Maximum Day Demand	1,064,635	580
Peak Hour Demand	1,398,683	762

Growth will impact the town's distribution system differently in each service zone. In general, development in the Lower Zone is more or less limited to that already known (South Main Addition) and infill. The Upper Zone, however, will likely see the majority of the growth. Table 14 and Table 15 present demand projections through 2034 for both the upper and lower zones respectively.

Table 14 - Upper Zone Future Demand Projection

Upper Zone Future Demand Projection				
Year	**SFE**	**Avg Day (GPD)**	**Max Day (gpd)**	**Peak Hour (gpd)**
2012	363	106,423	210,615	276,699
2013	365	106,894	211,547	277,923
2014	370	108,348	214,426	281,706
2015	375	109,818	217,333	285,526
2016	380	111,302	220,270	289,384
2017	385	112,800	223,236	293,281
2018	390	114,299	226,203	297,178
2019	395	115,798	229,169	301,075
2020	400	117,297	232,135	304,971
2021	405	118,795	235,101	308,868
2022	410	120,294	238,067	312,765
2023	415	121,793	241,033	316,662
2024	421	123,292	243,999	320,559
2025	426	124,791	246,966	324,456
2026	431	126,289	249,932	328,352
2027	436	127,788	252,898	332,249
2028	441	129,287	255,864	336,146
2029	446	130,786	258,830	340,043
2030	451	132,285	261,796	343,940
2031	456	133,783	264,763	347,837
2032	461	135,282	267,729	351,733
2033	467	136,781	270,695	355,630
2034	472	138,280	273,661	359,527

Note: Average day demand based off of historical average day demand of 292 gpd per SFE .Max day demand based off of historical max month demand of 580 gpd per SFE. Peak hour demand based off of historical peak hour demand of 768 gpd per SFE.

Table 15 - Lower Zone Future Demand Projection

Lower Zone Future Demand Projection				
Year	SFE	Avg Day (GPD)	Max Month (gpd)	Peak Hour (gpd)
2012	1,255	367,935	728,159	956,632
2013	1,257	368,406	729,091	957,857
2014	1,262	369,861	731,970	961,639
2015	1,267	371,330	734,878	965,459
2016	1,272	372,814	737,814	969,317
2017	1,277	374,313	740,781	973,214
2018	1,282	375,812	743,747	977,111
2019	1,287	377,311	746,713	981,008
2020	1,292	378,809	749,679	984,904
2021	1,297	380,308	752,645	988,801
2022	1,302	381,807	755,611	992,698
2023	1,307	383,306	758,578	996,595
2024	1,313	384,805	761,544	1,000,492
2025	1,318	386,303	764,510	1,004,389
2026	1,323	387,802	767,476	1,008,285
2027	1,328	389,301	770,442	1,012,182
2028	1,333	390,800	773,408	1,016,079
2029	1,338	392,298	776,375	1,019,976
2030	1,343	393,797	779,341	1,023,873
2031	1,348	395,296	782,307	1,027,770
2032	1,353	396,795	785,273	1,031,667
2033	1,359	398,294	788,239	1,035,563
2034	1,364	399,792	791,205	1,039,460

Note: Average day demand based off of historical average day demand of 292 gpd per SFE. Max day demand based off of historical max month demand of 580 gpd per SFE. Peak hour demand based off of historical peak hour demand of 768 gpd per SFE.

4.3 UNACCOUNTED WATER

Every water system has unaccounted water. Some common sources of unaccounted water are inaccurate flow meters, system leaks and un-metered users. The benefits of reducing unaccounted water include full use of water rights, increased revenues from billing, lower annual operating costs, and less expenditure for capital projects. Unaccounted water may typically ranges from 10 to 15% of production in many systems. This percentage or less are typically considered acceptable.

Water production and billing records were reviewed for the 2012 calendar year. Over this period the average monthly unaccounted water varied from 1,582,000 gallons in January to 5,761,100 in May. As a percent of production these values represent a range of 18.0% to 26%. This is significantly greater than the 10%-15% of production benchmark that is usually deemed acceptable by most water system operators. The fact that the annual volume and monthly volume of unaccounted water are generally consistent with only minor seasonal fluctuations is significant. This strongly suggests that losses are from a single, constant source that is unaffected by production and/or system pressures. The SFE use calculations included in this report use billed data to calculate overall demand and therefore do not include the unaccounted water.

The town has performed system-wide leak detection studies every three years. The first leak study found leaks that have since been repaired. Ensuing studies have not detected any major leaks. Despite the lack of detection of significant leaks, production and billing records still show significant unaccounted water volumes.

Un-metered usage could be a source of unaccounted water. The potential for such usage exists given that the town only started requiring meters in 1998. Evidence to support this is the fact that total losses are generally constant throughout the year with a slight increase in the irrigation season. The fact that system-wide leak detection studies have not detected any major leaks and unaccounted water still exists helps support this theory. Another theory is that older water meters in the town may be reading low. This is a common issue with disk meters as they age.

Discrepancies between production meters and billing records could also be a source of unaccounted water. This hypothesis has merit for two reasons: the production meter may be reading accurate flows due to its installation which would result in over-estimation of production, and/or the production meter has not been calibrated since installation. While this is a potential source of discrepancy, it does not appear to be the sole source because the unaccounted water volume does not change with production; it is always around 150,000 gallon per day.

Town staff discovered that the emergency feed line to the Department of Corrections (DOC) was not included in previous leak detection studies. Staff has subsequently isolated this line and it appears to have significantly reduced daily production requirements which suggests that this was is leaking. Staff will need to continue to monitor the system and compare billing and production records as they become available. Another possible area where significant leaks may be present is the low pressure 18-inch ACP transmission main. This main was put into service in 1975 and should be evaluated for integrity and leaks.

The town should remain vigilant in its endeavors to monitor the system and minimize unaccounted water as it potentially affects revenues, water rights and operating costs.

4.4 SYSTEM INTERCONNECTION

The Town of Buena Vista has an emergency service connection with the Department of Corrections (DOC) facility located to the south. This connection is one-way only and is rarely used by DOC since it has its own dedicated supply and distribution system. In the past, the town has delivered as much as 150,000 GPD through this connection. Future use of this connection is anticipated to only occur under emergency conditions. The DOC water tower can back-feed into the town's distribution system in an emergency.

4.5 FIRE FLOW REQUIREMENTS

All existing service areas were evaluated for fire flow capacity. Fire flow capacity was evaluated under max day demand conditions in each area. Historically the design fire flow requirement used in town was probably on the order of 500 to 1,000 GPM for two hours. However, under new Uniform Fire Code (UFC), while most residences still only require 1,000 GPM, commercial and high density residential properties (e.g. townhomes and hotels) now require flows of 1,500 to 3,500 GPM for up to three hours. It should be noted that actual fire flow requirements are still set by the local fire chief, and that he has the authority to reduce requirements on a case-by-case basis. If the fire chief is unavailable, then the town engineer may set fire flow requirements based on his/her interpretation of the prevailing UFC.

The town completed an Insurance Services Office (ISO) study to establish fire insurance ratings in town. For the purposes of this report, all areas were checked against the flow rates set in the ISO study, which ranged from 750 GPM to 3,500 GPM. The individual ISO flow requirements are presented in Appendix A. It is recommended that a new study be completed.

5.0 LEGAL WATER RIGHTS

5.1 SUMMARY OF TOWN WATER RIGHTS

Note that the water rights sections remain largely unchanged from the original 2006 report. Water rights in Colorado are allocated based on a system of prior appropriation. This "first in time-first in right" system gives senior priority on a stream to the first person who diverted water from the stream and put the water to beneficial use. Under this system junior priorities are cut off entirely to the extent required to provide a full supply of water to downstream, senior priorities that have placed the call. There is no pro rata curtailment during shortages. This allocation system can place legal limitations on the ability of the town's junior water rights to reliably meet demand.

The town's senior water rights (Thompson, Cottonwood Irrigating 1866 priority, and Prior Right) can be counted on to be available, with rare exceptions, during most of the irrigation season, and the Buena Vista Water Works Right can be counted on in the non-irrigation season. The Supply Ditch, the Town Ditch and the Cottonwood Irrigating 1872 priority, being more junior, are less reliable. The Leesmeagh and Gorrel Ditches, although senior water rights, are constrained by stipulations made in Case No. 83CW88, as discussed below.

The Town of Buena Vista water rights are summarized in Table 16

Table 16 - Water Rights Summary

(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)
Water Right	Decreed Amount	Appr. Date	Adj. Date	Ownership by Town	Ownership by Town	Allowed Diversion @ Town Intake and Infiltration Gallery	Season of Use
	[CFS]			[CFS]	[MGD]	[CFS]	
Leesmeagh	4	11/30/1864	06/19/1890	1.833	1.19	Varies*	Irr.
Thompson	4	12/19/1864	06/19/1890	2	1.29	2	Irr.
Prior Right	2	04/30/1866	06/19/1890	1	0.65	1	Irr.
Gorrel	4	05/31/1866	06/19/1890	2.66	1.72	Varies**	Irr.
Cottonwood Irr.	6	07/31/1866	06/19/1890	0.88	0.57	0.88	Irr.
Cottonwood Irr.	13	12/31/1872	06/19/1890	0.12	0.08	0.12	Irr.
Town	4	06/01/1880	7/14/1903	4	2.59	2	Irr.
Supply	2	06/01/1880	7/14/1903	2	1.29	2	Irr.
Buena Vista Water Works	10	06/01/1883	9/10/1904	10	6.46	10.0***	Non-Irr.****

*Ranges from 0.8 to 1.1 CFS during May-Sept See Case 83CW88 for details.
**Ranges from 0.6 to 0.9 CFS during May-Sept See Case 83CW88 for details.
***Town Well No. 1 is alternate point of diversion of 0.1 CFS of the Buena Vista Water Works right. This 0.1 CFS can only be diverted during the irrigation season.
****Year-round, but generally in priority in non-irrigation season.

The columns in Table 16 are described as follows:

(a) The name of the water right.

(b) The decreed flow rate taken from Case 83CW88.

(c) The Appropriation Date is the date that the water right was first put to beneficial use. This is the date that determines the water right's priority on the stream.

(d) The Adjudication Date is the date that the Water Court entered the Decree.

(e) Decreed flow rate of the water right in cubic feet per second.

(f) Decreed flow rate of the water right in millions of gallons a day.

(g) The allowable flow rate of diversion from case 83CW88 in cubic feet per second.

(h) Allowable season of use.

During the non-irrigation season (November – March) the town has the most senior water right on Cottonwood Creek, the Buena Vista Water Works right. Historically Cottonwood Creek has had sufficient flow to satisfy all of this right.

During the irrigation season (April - October) the Buena Vista Water Works right is too junior to be a reliable supply, and therefore, the town has acquired a number of senior irrigation rights on Cottonwood Creek. These irrigation rights are the Leesmeagh, Thompson, Prior Right, Gorrel, Cottonwood Irrigating, town and Supply rights. In water court cases over the years the town has changed many of these irrigation rights to allow the water to be diverted at the town Intake and Infiltration Gallery. In particular, in 1989, the Water Court approved a complex water rights change case in Case No. 83CW88.

Case No. 83CW88 confirmed and corrected the terms of previously changed irrigation rights, and decreed changes of use and point of diversion for others. In addition to changing the points of diversion of the irrigation rights to include the town Intake and Infiltration Gallery, this change case changed the use of these rights to include "irrigation for lawns, gardens and green spaces, municipal use, domestic use, fire protection use, recreation use and all other beneficial uses, including historical crop irrigation." The decree also changed 0.1 CFS of the Buena Vista Water Works rights so that it can be diverted at the town Well No. 1. In order to obtain this decree the town was required to make agreements with other Cottonwood Creek and Arkansas River water rights users to ensure non-injury. The town made an agreement with St. Charles Mesa to protect St. Charles Mesa's interest in the Cottonwood Irrigating Ditch. The town also reached an agreement with the Colorado Water Conservation Board (CWCB) that protects the Board's in-stream flows in Cottonwood Creek by subordinating the town's interest in the Town and Supply Ditches to the 20 CFS instream flow. Additionally, prior to the use of the Leesmeagh and Gorrel rights for municipal purposes, the town is required to demonstrate dry-up of approximately 111 acres of land historically irrigated by these two ditches.

At present the town has not needed to implement all of the changes outlined in Case No. 83CW88. During the non-irrigation season the town uses the Buena Vista Water Works right, diverted at the town Intake and Infiltration Galley, as its source of water. During the irrigation season the town is currently exercising its 0.1 CFS Buena Vista Water Works right at the town Well No. 1 (when it is in priority) in addition to the Thompson (2.0 CFS), Prior Right (1.0 CFS), and the senior Cottonwood Irrigating (0.88 CFS) water rights, which are all diverted at the town Intake and Infiltration Gallery. The town has not yet started diverting the Leesmeagh and Gorrel rights at the town Intake and Infiltration Gallery and has not yet dried up the associated irrigated lands. The town has attempted in the past to dry up the Leesmeagh Meadow, but was not able to, therefore, the town has not been able to exercise those rights. The remaining rights (i.e. the junior Cottonwood Irrigating (0.12 CFS), the town (2.0CFS), and the Supply (2.0 CFS)) are too junior to be reliable sources of water during dry periods. As the water system is presently operated, the town can reliably divert up to 3.88 CFS, or 2.51MGD, of water at the town Intake and Infiltration Gallery during the irrigation season. This water consists of the Thompson, Prior Right, and Cottonwood Irrigating rights.

5.2 SYSTEM LIMITATIONS

There are a number of factors that could place a limitation on the ability of the town's water rights to adequately meet the system demand. These factors can be broken down into either physical limitations or legal limitations. Physical limitations occur when the town has legal rights to divert water from the stream but is unable to divert because of factors that limit the quantity or quality of water available at the town's diversion structures. Legal limitations exist when there is water present in the stream, but the town's water rights are out of priority so it has no legal right to divert water. Legal limitations, rather than physical limitations, currently impose the more significant limitation on Buena Vista's water supply system.

It is prudent to analyze the likelihood that physical limitations during the non-irrigation season due to low stream flow could limit the town's ability to provide sufficient supply. While the town owns the senior winter right on Cottonwood Creek (Buena Vista Water Works), there is no guarantee that there will be adequate water in the stream to meet the demand. The maximum non-irrigation season (Nov.-Mar.) per tap daily production for 2012 was approximately 293 GPD per SFE. Given the projected population growth, this would result in a maximum non-irrigation season production rate of 493,615 gpd, or 0.76 CFS, in the year 2034 (~1,835 SFE). This amount is less than the Buena Vista Water Works decreed rate of 10.0 CFS. Analysis of USGS records for the stream gage located on Cottonwood Creek below the Cottonwood Hot Springs (USGS #07089000) indicates that during the period 1911 to 1986 the minimum flow on Cottonwood Creek was 13 CFS. This gage was located above the confluence of Cottonwood Creek and North Cottonwood Creek and therefore underestimates the flow in Cottonwood Creek at the town Intake. Given the projected demand and the stream gage record it seems unlikely that the Buena Vista Water Works right will be limited by physical availability of water when it is in priority.

Another potential physical limitation is the quality of water available at the town's intake structures. The Colorado Department of Public Health and Environment, in the 2004 Source Water Assessment Report, rated the total susceptibility of Buena Vista's water sources to contamination as Moderate. Water sources with total susceptibility ratings of Moderately High or High are at greater risk for potential contamination than those receiving lower ratings. Cottonwood Creek currently has very high water quality the majority of the time, with the exception of an occasional problem with high turbidity during the spring runoff. Historically this high turbidity has only impacted the town Intake for a few days a year. The issue is easily addressed by diverting water through the infiltration gallery, which is not impacted by the turbidity. As long as the town is vigilant in enforcing the watershed protection ordinance there will be minimal chance of physical limitations as a result of water quality. In particular the town should monitor the proliferation of septic systems in the Cottonwood Creek watershed. Water quality related issues are discussed more thoroughly in Chapter 5.

Cottonwood Creek is an internally controlled stream, meaning that the senior water rights diverted from Cottonwood Creek, as opposed to those diverted on the main-stem of the Arkansas River, tend to control the call on Cottonwood Creek. Unfortunately, reliable historic call records for Cottonwood Creek and the Arkansas River have not been maintained. Attempts to locate some record of the calls have been unsuccessful. Due to this lack of data, it is difficult to predict the probability that a senior call will require the town to curtail diversions. The town's staff has indicated that to-date the town has been able to withdraw all the water it needs, even in the drought years of 1977 and 2002. As the demand on the system increases, the chances that the town will have to curtail diversions will increase. If only the town's most senior changed irrigation rights – the Thompson Ditch (1864) and the Leesmeagh Ditch (1864) – are in priority, the town could divert at its intake and infiltration gallery a total of 2.0 - 3.1 CFS (1.3 – 2.01 MGD), depending on the month. The town must demonstrate dry up of historically irrigated land in order to use the Leesmeagh water right at its intake. These 1864 rights will not be called out by Cottonwood Creek rights, but Buena Vista's agreements with CWCB and St. Charles Mesa (discussed below) may reduce the amount of the Leesmeagh water right Buena Vista may divert. There may be times when a Cottonwood Creek call senior to 4/30/1866 would require Buena Vista to curtail its diversion of the Prior Right, and it could only use the Thompson Ditch water right. If a senior Arkansas River right calls, and the average day demand is greater than 1.3 MGD Buena Vista can release stored transmountain Fryingpan-Arkansas water from Twin Lakes to meet that call. Historically, no Arkansas River call has called out Buena Vista's most senior rights.

Other potential legal limitations on the use of Buena Vista's changed irrigation rights stem from the decree outlined in Case 83CW88. The town made three agreements with objectors to protect the objectors' water rights from injury. The first of these agreements is with St. Charles Mesa (St. Charles), an urbanizing area near Pueblo, CO, on the Arkansas River. St. Charles owns 1.2 CFS of the senior Cottonwood Irrigating right (Appr. 7/31/1866) and 2.6 CFS of the junior Cottonwood Irrigating right (Appr. 12/31/1872). The town agreed that it would not place a call against these rights under the town's interest in the Thompson (2.0 CFS, Appr. 12/19/1864) and Prior Right (1.0 CFS, Appr. 4/30/1866) rights. Additionally, the town agreed that when water is physically and legally available to the Cottonwood Irrigating Ditch rights, it will subordinate its 0.88 CFS of the senior Cottonwood Irrigating priority, and its 0.12 CFS of the junior Cottonwood Irrigating priority to St. Charles' 1.2 CFS in the senior Cottonwood Irrigating priority and 2.6 CFS in the junior Cottonwood Irrigating priority, respectively. The town also agreed to curtail its diversions or provide water to compensate for losing stream conditions between the Cottonwood Irrigating's original point of diversion and St. Charles' downstream measuring gage. Lastly the town has agreed to limit its diversions under the Leesmeagh and Gorrel to the schedule outlined in Table 17. In an effort to protect St. Charles by providing water on the Arkansas River, the town has contracted with the Fryingpan-Arkansas Project for 1000 ac-ft of water stored in Twin Lakes reservoir, although this is not provided for in the decree in Case No. 83CW88.

Table 17 - Leesmeagh and Gorrel Schedule

Period	Total Leesmeagh* Gorrel Diversion	Leesmeagh Diversion**	Gorrel Diversion***
Jan.-Apr.	Zero	Zero	Zero
May	1.4 CFS	0.8 CFS	0.6 CFS
June	2.0 CFS	1.1 CFS	0.9 CFS
July	2.0 CFS	1.1 CFS	0.9 CFS
Aug.	1.5 CFS	0.8 CFS	0.7 CFS
Sept.	1.4	0.8	0.6
Oct.-Dec	Zero	Zero	Zero

* Taken From Case 83CW88.
**The Town owns 1.833 CFS of the Leesmeagh Right.
***The Town owns 2.66 CFS of the Gorrel Right.

The second agreement outlined in Case 83CW88 is an agreement with the CWCB to protect its 20.0 CFS in-stream flow right at the mouth of Cottonwood Creek (decreed in Case No. 79CW115). Upon transfer of the Leesmeagh and Gorrel to the town Intake and Infiltration Gallery, the town has agreed to subordinate its Leesmeagh and Gorrel water rights to the 20.0 CFS in-stream flow during the months of April, May, September and October. When the flow below the town Intake and Infiltration Gallery is less than 20.0 CFS, the town will limit its diversion under the Leesmeagh and Gorrel to a total from both ditches of 1.4 CFS in May and 1.4 CFS in September. The decree in Case No 83CW88 provides that the town will not divert the Leesmeagh and Gorrel at its intake or infiltration gallery at all in April and October, as shown in Table 17 above. Additionally, the town has agreed to subordinate its entire 4.0 CFS of the town Ditch right and 2.0 CFS of the Supply Ditch right to the CWCB's 20 CFS in-stream flow if the instream flow is not being met immediately downstream of the town Intake and Infiltration gallery.

The third agreement outlined in Case 83CW88 also pertains to the Leesmeagh and Gorrel rights. Under this agreement the town has agreed to dry-up all or a portion of the areas historically irrigated by water under these rights prior to diverting water under these rights at the town Intake and Infiltration Gallery for municipal uses. Approximately 48 acres of land previously irrigated by the Gorrel and 63 acres of land previously irrigated by the Leesmeagh must be dried up under this agreement. The town has not yet needed to use these rights and has not dried up these lands. When the town requires the additional water to meet demand it will need to obtain credit for the dry-up of these lands by constructing observation wells at the lowest point of any tract and demonstrating that the water table is kept a minimum of four feet below ground surface. If the town is unable to meet these dry-up requirements, it will not be able to legally divert any water under these rights for municipal purposes. These legal limitations present the greatest future supply challenge to the town.

5.3 TIMING OF WATER RIGHTS ACTIONS

The town has the following water rights during the months of peak flow: 3.88 CFS from the Thompson Right, Prior Right, and Cottonwood Irrigating Right (currently being used); 1.1 CFS from the Leesmeagh Right (currently not being used, requires dry-up); and 0.9 from the Gorrel Right (currently not being used, requires dry-up). (The junior Cottonwood Irrigating Ditch priority (0.12 CFS) and the Town and Supply Ditches are too junior to be considered a consistently reliable source of water to meet flow requirements.)

As Buena Vista's population increases past the 20-year planning horizon of this master plan, and more people look to the Arkansas River for water, the town will inevitably need to locate new sources of water in addition to fully utilizing the water rights it currently possesses. This section outlines the benefits and drawbacks to a number of potential sources of water.

5.3.1 Existing Water Rights

At present the town's irrigation season water supply comes from diversions made under the Cottonwood Irrigating, Prior Right, and Thompson water rights. These rights constitute 3.88 CFS of reliably available water for all municipal uses. When the town's MDD is eighty percent of this supply, or 3.1 CFS, the town should be prepared to divert additional water. The first logical source for additional water is diversion under the Leesmeagh water right. The Leesmeagh right was decreed for diversion at the town Intake and Infiltration Gallery in Case 83CW88. The use of this right at the town Intake and Infiltration Gallery is contingent upon the town meeting the dry-up requirements outlined in 83CW88.

The Gorrel water right was also decreed for diversion at the town Intake and Infiltration Gallery in case 83CW88. The use of this right at the town Intake and Infiltration Gallery is also contingent upon the town meeting dry-up requirements outlined in 83CW88. The Gorrel water right presently (and historically) irrigates the Gorrel Meadow, which is also the location of the town's Infiltration Gallery. As currently operated, after the Gorrel right has irrigated the Gorrel Meadow, the tail water serves to recharge the town's Infiltration Gallery. This configuration has proven to be desirable for the town because it allows them to produce more water from the Infiltration Gallery while limiting Water Treatment Plant operations. This present configuration does not, however, entitle the town to divert any water in excess of the Cottonwood Irrigating, Prior Right, and Thompson water rights. The town is required to prove dry-up of the Gorrel Meadow if any of the Gorrel water is diverted by the town for municipal purposes. If the town chooses to continue managing the Gorrel water right and associated meadow as it presently does, and the Leesmeagh right comes into production as described above, then the town's reliable, consumable water rights will be limited to 4.98 CFS. The alternative is to dry-up the Gorrel Meadow, which may will limit infiltration gallery production to 300 GPM, or so, and divert that amount plus the Gorrel right water though the water treatment plant. Based on the report titled Hydrogeology and Quality of Ground Water in the Upper Arkansas River Basin from Buena Vista to Salida, Colorado, 2000-2003, it appears that this minimum infiltration gallery production of 300 GPM is unlikely to change in the future. The report states:

"Currently (2003), annual withdrawal of ground water by an estimated 3,443 domestic and household wells completed in the alluvial-outwash and basin-fill aquifers in the Buena Vista Salida structural basin is about 690 to 1,240 acre-feet. By 2030, projected annual withdrawals to supply an additional 4,000 to 5,000 domestic and household wells are estimated to require an additional 800 to 1,800 acre-feet. During September 2003, estimated storage of drainable water in the upper 300 feet of the alluvial-outwash and basin-fill aquifers was about 472,000 acre-feet. However, in some areas little water is available within 300 feet of the land surface. Current and projected rates of consumptive use by domestic and household wells are unlikely to substantially affect water supplies because of current augmentation plans and because most new wells require an augmentation plan to replace consumptive use. In densely populated areas, well interference could result in decreased water levels and well yields, which may require deepening or replacement of wells."

Under the system of prior appropriation as administered in Colorado, a water right can be quantified based on its diversion rate as well as the rate at which it is consumed, called consumptive use (CU). Due to high evapotranspiration losses, water that is used for irrigation is typically consumed at a relatively high rate, on the order of 70%-80% of the water supplied for sprinkler irrigation (much lower efficiency for flood irrigation). Water that is used for domestic, in-home purposes is typically consumed at a much lower rate, estimated at 5%-10% of the water supplied. It is assumed that the unconsumed water is returned to the stream and is available to downstream water users. Many communities in Colorado keep records of water production volumes as well as WWTP flow volumes. The difference between the wastewater treatment plant flow volume and the water production volume is the amount consumptively used through the community water system. Typically, the WWTP return flows are credited against the diversions. The Town of Buena Vista's current water rights from Cottonwood Creek are (1) the Buena Vista Water Works Right, and (2) the changed irrigation rights. Neither of these is a reusable water right. The Buena Vista Water Works Right allows water to be diverted at the town Intake or Infiltration Gallery, used once for municipal purposes, and returned to the river as wastewater effluent or lawn irrigation return flows. On the occasions when Buena Vista is able to exchange its Fryingpan-Arkansas Project water to its intake and infiltration gallery pursuant to the decree in Case No. 96CW17, the wastewater effluent from such water would be reusable and could be exchanged again, used for augmentation, or stored.

5.3.2 New Water Rights

As demand for water in the Arkansas River Basin increases, there will be an increased probability that heretofore reliable water rights will be called out. As previously discussed, a call placed on the river system senior to 04/30/1866 has the potential to reduce Buena Vista's legally available water rights to 2.0 CFS, or 1.29 MGD if only the Thompson Ditch is available. This is less than the current annual peak day demand. The town should obtain additional senior irrigation water rights in order to decrease the susceptibility to water shortages. These senior water rights could be located on either Cottonwood Creek or the Arkansas River.

Given the existing infrastructure, the known water quality, and known reliability of Cottonwood Creek water rights, the town's first priority should be to obtain senior irrigation rights on Cottonwood Creek. Once the water court proceedings were completed, the town would be able simply to start diverting water with little, if any, additional capital expense. The only potential disadvantage to obtaining Cottonwood Creek rights is that the town still would be limited to only one physical water source (Cottonwood Creek). It is preferable for a community to have more than one water source to protect against unforeseen problems (e.g. the devastating impacts of a large scale wildfire in the watershed, other contamination event, or a breakdown of current treatment facilities).

Another possible source for additional senior water rights is the Arkansas River. Senior Arkansas River water rights could be used in an augmentation plan to cover out-of-priority depletions to the Arkansas River without significant capital improvements. For example, such rights might be used to meet the town's obligations under its agreement with St. Charles Mesa (Case 83CW88). Additionally, if the appropriate infrastructure is installed, these senior Arkansas rights could be changed to municipal uses to serve as a second water source either for non-potable irrigation water or for potable water via a new well or surface diversion. Alternatively, the consumptive use associated with senior Arkansas River irrigation rights could be used to augment wells or out of priority diversion by a junior surface diversion. Due to the expense of surface water treatment it would be preferable for this second source of supply to be a well or wells. A key challenge, in this case, is to find well sites adjacent to the Arkansas River that provide sufficient quantity and quality.

As previously stated, the town's existing water rights are projected to be adequate to serve through the 20-year planning horizon of this master plan. If population projections should change, however, three trigger points have been developed to assist in timing new water rights acquisitions. These trigger points are as follows:

- Trigger Point #1 occurs when the current MDD is eighty percent of 3.88 CFS (amount currently available for diversion at town intake and infiltration gallery), or 3.1 CFS. Given the town's current water rights portfolio, its best option at Trigger Point #1 is to prove dry-up of the land irrigated by the Leesmeagh as outlined in Case 83CW88 and start diverting the Leesmeagh water at the town Intake and Infiltration Gallery. This will bring the amount of usable water rights to 4.98 CFS. This trigger point is expected to be reached

- Trigger Point #2 occurs when the current MDD is eighty percent of 4.98 CFS (amount available for diversion after Leesmeagh dry-up), or 4.0 CFS. Assuming that the town is limited to the existing water rights portfolio, the town's best option at Trigger Point #2 is to prove dry-up of the land irrigated by the Gorrel as outlined in Case 83CW88 and start diverting the Gorrel water at the town intake and infiltration gallery. This will bring the amount of usable water rights to 5.88 CFS.

- Trigger Point #3 occurs when the current MDD is eighty percent of 5.88 CFS (amount available for diversion after Leesmeagh and Gorrel dry-up), or 4.7 CFS. At this point, additional demand will exceed the existing water rights currently available to the town. Assuming that the town is able to obtain additional water rights prior to Trigger Point #3, then the town's best option at Trigger Point #3 is to transfer the point of diversion of said water rights to the town Intake and infiltration gallery or other applicable point of diversion, or use the consumptive use from senior irrigation rights to augment depletions from wells and/or junior surface diversions.

5.3.3 New Wells

A well that is tributary to the Arkansas River has some advantages over a well that is tributary to Cottonwood Creek. Out-of-priority well diversions on the Arkansas are more easily augmented than on Cottonwood Creek because of the availability of stored contract water (Fry-Ark) on the Arkansas River. Additionally, a well in the Arkansas River Alluvium would provide a redundant, second source of water in addition to Cottonwood Creek. A disadvantage to a well tributary to the Arkansas is the expense of connecting the well to the existing distribution system. Some preliminary research was performed to determine the feasibly of developing a municipal well which is tributary to the Arkansas River. The results of this research follow.

In the Upper Arkansas River Basin there are three principal aquifers: the Alluvial Fill Aquifer (Alluvial), the Glacial Till and Outwash Aquifer (Glacial), and the Basin Fill Aquifer (Basin). A USGS report titled *Hydrogeology and Quality of Ground Water in the Upper Arkansas River Basin from Buena Vista to Salida, Colorado, 2000-2003* gives general descriptions of these three aquifers. The Alluvial aquifer ranges in depth from 10 to 165 feet and has well yields ranging from 0.01 to 1500 GPM. The Glacial aquifer ranges in depth from 0 to 500 feet and has well yields ranging from 0.03 to 60 GPM. The Basin aquifer ranges in depth up to 5000 feet and has well yields ranging from 0.01 to 1500 GPM. The wide range of these numbers points to the heterogeneity of these aquifers and the subsequent difficulty with predicting well yields at a specific well location.

Due to the geography of Buena Vista there is limited area were a well tributary to the Arkansas could be developed. Well records for 24 wells located to the east of US-24 and to the North of Main St. were investigated. Figure 2 shows the locations of 19 of the 24 wells investigated (The remaining 5 wells did not have exact locations indicated on the permits and are not shown). These wells range in depth from 46 feet to 125 feet. The depth to water ranges from 30 feet to 85 feet. A review of the well logs indicates that most drillers encountered large boulders somewhere within the first 30 feet of drilling. Below this depth there was a wide variety of materials ranging from clay to large granite boulders to bed rock. Well production from these 25 wells is generally much less than would be required by a municipal well, on the range of 10-15 GPM. However, these data do not indicate that larger production wells in the area are impossible. Most of these wells were drilled for single family residences; there was no effort to maximize yield beyond these low flow rates. A review of the pumping test records points to the variability in yields for this area. One well was tested for 2 hours at a rate of 25 GPM and showed no drawdown. Another well was tested for 2 hours at a rate of 15 GPM and showed 125 feet of drawdown. Yet another well was tested for 2 hours at a rate of 60 GPM and showed a drawdown of 51 feet.

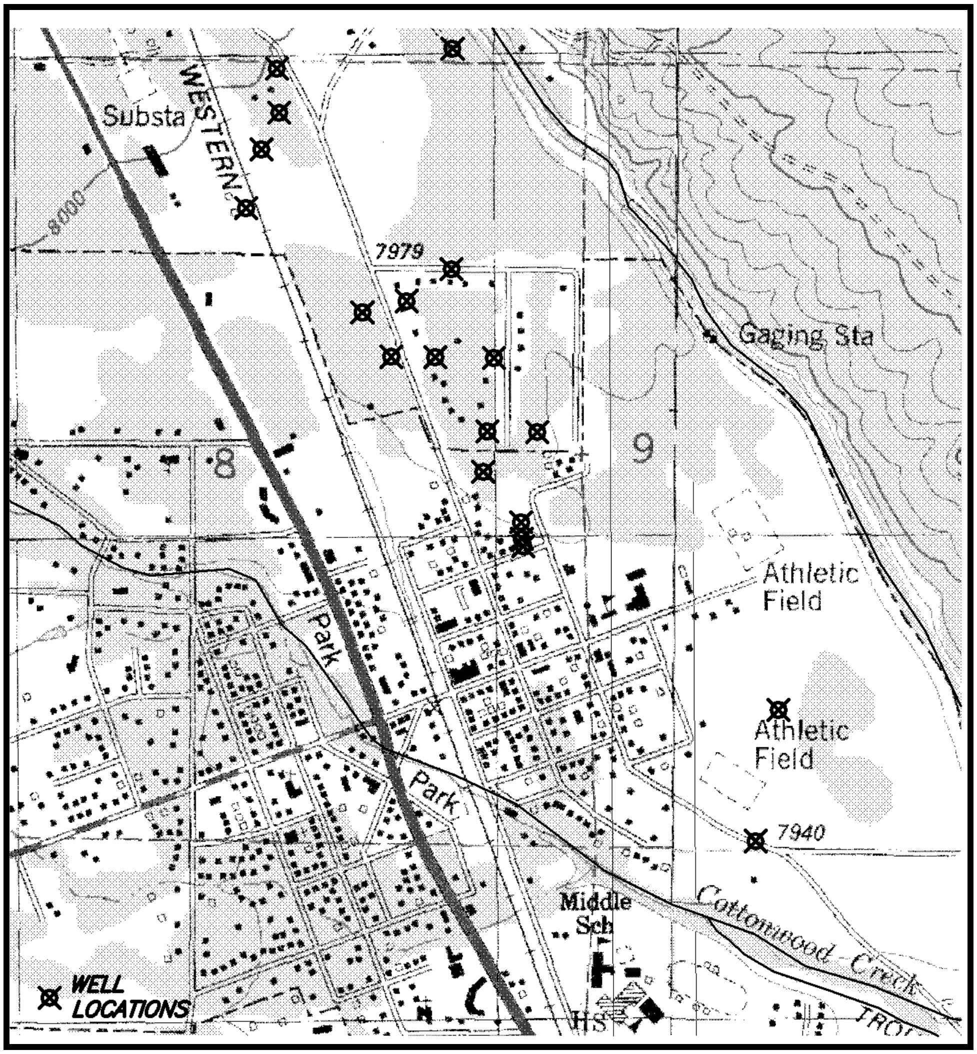

Figure 2 - Location of Well Logs Reviewed

Conversations with the town's water rights engineer (Wright Water Engineers) indicate that the Department of Wildlife drilled and tested two high capacity wells at the Department of Corrections fish hatchery facility south of town. A conversation with a local well driller indicated that these wells produced in excess of 500 GPM, but no other information has yet been found on this testing.

Given the apparent variability of aquifer properties it is difficult to say conclusively whether a particular proposed well will provide adequate yield. However, given the augmentation opportunities as well as the security of a second, redundant source, it is recommended that the town investigate the feasibility of constructing a well which is tributary to the Arkansas River.

The construction of a well which is tributary to Cottonwood Creek would not offer the advantages that an Arkansas well would. However, there may be operational advantages to constructing more wells which are tributary to Cottonwood Creek due to the reduced treatment requirements of groundwater. It should be noted that all wells will require an augmentation plan, and this will require both acquisition of an effective augmentation source and water court approval. For these reasons, it is recommended that if the town pursues additional wells in the Cottonwood Creek drainage, it should also obtain additional senior irrigation rights to augment the diversions from those wells.

5.3.4 Augmentation Storage

The final component to Buena Vista's future water supply is storage. The ability to store water allows a water right to be stored while it is in priority and released at a later time to satisfy demand when it is out of priority. The water can be released and diverted or it can be released to satisfy downstream users (e.g. exchange or augmentation). Clearly, the ability to store water on Cottonwood Creek would be beneficial to Buena Vista. Over the last 10 years, the town has been negotiating for storage on Cottonwood Creek.

Case No. 96CW17 was a joint application by Buena Vista, the Upper Arkansas Water Conservancy District and the Southeastern Colorado Water Conservancy District to allow exchange of Fry Ark Project water to Cottonwood and Rainbow Lakes and to the Buena Vista Intake and Infiltration Gallery. The decree allows the town to exchange a total of 75 acre-feet a year of Fry Ark water into storage in Cottonwood and Rainbow Lakes, and/or to its Intake and Infiltration Gallery. In addition, this decree allows Upper Arkansas Water Conservancy District to exchange up to 75 acre-feet per year of Fry-Ark water into storage in Cottonwood and Rainbow lakes. This decree does not give the town any actual storage agreements with Upper Arkansas Water Conservancy District for the use of Cottonwood and Rainbow Lakes, although the town has held discussion with the Upper Arkansas Water Conservancy District in an effort to obtain such storage.

Case No. 98CW38 is the town's right to exchange Fry-Ark Project water to Well No. 2, and to augment well depletions using Fry Ark Project water stored in Cottonwood and Rainbow Lakes, as well as to change a portion of the Buena Vista Water Works right to that well. It has been decreed, but the town has not been able to work out a storage agreement with Upper Ark. The town also purchases Fry Ark water annually and currently has over 1,500 acre feet.

Given the status of the above mentioned cases, it is recommended that the town investigate alternative storage opportunities on Cottonwood Creek. The development of a new storage facility will potentially require a significant investment of time and money, and for this reason may not be feasible. These costs could be somewhat reduced if an existing body of water were utilized as opposed to creating an entirely new facility. In particular, the potential for storage in Fox Lake, above Cottonwood Lake on Cottonwood Creek, should be investigated.

Alternatively, an intergovernmental agreement with the Upper Arkansas Water Conservancy District could be sought that will provide for Cottonwood Creek augmentation of the town's municipal wells.

The town has also contracted for 1000 AF of Fry-Ark Project water which is stored in Twin Lakes. This water is held in reserve by the town to be used to satisfy calls on the Arkansas as well as agreements with St. Charles Mesa. Due to the reliability of the town's water rights on Cottonwood Creek, the town has never been required to use this water. Even in the dry year of 2002 the town did not have to release any of this Fry-Ark water, although Town officials have indicated that they expected an Arkansas River call that would have required the release. The town's water rights engineer (Wright Water Engineers) has attempted to model the current agreement under the conditions that occurred in 1977, another significant drought year. It is estimated that if the agreement with St. Charles Mesa was in place in 1977 the town would have had to supply the 0.88 CFS of Cottonwood Irrigating water for a period of approximately two weeks. This would have required approximately 1.75 AF a day. Over the two week period this would equate to approximately 24.5 AF, well below the 1000 AF set aside for such purposes.

As previously mentioned, as more pressure is placed on the Arkansas River there is increased possibility that the town's rights will be called out or that the town will have to satisfy the St. Charles Mesa agreement outlined in Case 83CW88. Therefore, it is recommended that the town maintain this contracted water. If, in the future, the town is unable to secure additional water rights on either Cottonwood Creek or the Arkansas River, it may be in the town's best interest to purchase additional Fry-Ark water.

6.0 WATER QUALITY & TREATMENT

6.1 SOURCE WATER

The Town of Buena Vista currently has three available sources of raw water. These are 1) the groundwater infiltration gallery which is recharged by water from North Cottonwood Creek, 2) Well No. 2 and 3) direct diversion of water from Cottonwood Creek.

The town's primary source of raw water is the infiltration gallery. The infiltration gallery consists of a diversion off North Cottonwood Creek metered by a parshall flume. The flow is spread over a meadow just to the west of the water treatment plant. The water percolates into the ground and is collected by an underground system of perforated pipes and delivered to the water treatment plant for disinfection. Water from the infiltration gallery is deemed not under the influence of surface water by CDPHE and therefore requires only disinfection prior to being sent to the distribution system. The infiltration gallery area is shown in Figure 1. Because it requires minimal treatment and no pumping, the infiltration gallery is the town's primary supply. The town's existing infiltration gallery supply is a valuable asset due to the following characteristics:

- It requires no filtration or other advanced treatment; it only requires chlorine disinfection, corrosion control using pH (and possibly, alkalinity) adjustment, and fluoride addition, if desired.
- It flows entirely by gravity, which saves energy and pump maintenance costs, increases its reliability since its availability is unaffected by power failure (given that the existing chemical building does have backup power).
- Its output can be controlled by the operators.
- It is fed from a reliable source of supply.

The secondary source of raw water is Well No. 2 which sits adjacent to the treatment facility and disinfection system. Well No. 2 is used to augment raw water delivery whenever demands outstrip the ability of the infiltration gallery to supply the system. This alluvial well is 100 feet deep and provides high quality water, which like the infiltration gallery, only requires chlorination for treatment.

The third source of water is a direct surface water intake off of Cottonwood Creek adjacent to the water treatment plant. This source has not been used in the past as the infiltration gallery and periodic augmentation from Well No. 2 have historically been able to supply all demand necessary.

A fourth source of water, Well No. 1, is located at the Rodeo Grounds and is available for use in that area.

6.2 SOURCE WATER PROTECTION

Economic and political pressures to change the land use of the infiltration gallery site may increase with increasing development of surrounding lands. The gallery's product water quality will likely be sensitive to on-site and nearby land uses. If the town desires to keep the resource as one requiring limited treatment only, it will be in its best interest to control activities in and around the infiltration gallery site.

6.3 EXISTING WATER TREATMENT PLANT

The Buena Vista WTP (BVWTP) is a direct filtration plant with pre-sedimentation that treats water from Cottonwood Creek. The plant has a nominal capacity of 1. 5 MGD based on a 4-GPM/sf filter loading rate with both filters running and about 10% daily production loss due to backwash water usage and time off-line for backwashing. Filter media was replaced in 1998 and has remained in the filter beds since. Figure 3 shows the WTP building.

Figure 3 - Existing Water Treatment Plant

The plant was constructed in 1974 and has been very well maintained; most of the major equipment items are original and still in good repair. Table 18 presents summary descriptions of the systems and processes in the plant. More detailed accounts can be found in the 2001 Comprehensive Performance Evaluation (CPE) report produced for CDPHE by Sear-Brown. In summary, the treatment facility consists of the following unit processes:

1. **Raw Water Intake off of Cottonwood Creek (Figure 4):** The existing raw water intake is in relatively poor shape and is corroded. The concrete structure is also deteriorated.

Figure 4 – Existing Raw Water Intake Screen

2. **1.0-Million Gallon Pre-sedimentation Pond (Figure 5):** The existing pre-sedimentation pond is lined with concrete that has heaved at joints and is in need of repair. A rock overflow wall was installed to decrease short circuiting. Prior to bringing the pre-sedimentation pond online it is recommended that sediment and plant material be removed and the pond be re-lined.

Figure 5 - Existing Pre-Sedimentation Pond

3. **Chemical Pre-Treatment and Rapid Mix:** The chemical pre-treatment system consists of the addition of polyaluminum chloride coagulant and cationic polymer. A single in-line vertical rapid mixer is located on the influent pipe to the filters.

4. **Flocculation (Figure 6):** There is one flocculation bay prior to each filter bay. These bays are 12-ft x 12-ft x 10.5-ft. and separated by horizontal wooden baffles into three compartments. Each bay has a single vertical shaft flocculator with 21-inch blades in the second compartment of each basin. The flocculator drives are variable speed but are typically set for 15 rpm.

Figure 6 – Existing Flocculator Vertical Shaft and Motor

5. **Filtration (Figure 7):** There are two (2) multi-media, gravity flow filters designed to operate in parallel with 144 square feet of surface area each. The existing media consists of 18-inches of anthracite, 12-inches of course and garnet sand and 15-inches of gravel. There are no provisions for filter to waste in the existing WTP. Currently, the filter media is covered with plastic to protect it while the plant is not being used.

Figure 7 - Existing Filter Bay

6. **Backwash Pumping and Handling (Figure 8):** There is one constant-speed vertical turbine pump pulling from the existing clearwell. Flow is controlled by valve modulation and is typically around 2,500 gpm. When in operation, the backwash duration was around 15-minutes. There are two lined ponds that hold spent backwash from each filter. A dual submersible pump station pumps backwash supernatant back to the pre-sedimentation pond.

Figure 8 - Existing Backwash Ponds and Pump Station

7. **Clearwell:** The existing treatment plant has a 33,000 gallon non-baffled clearwell. Due to the volume of water required for backwash there is little to no reliable chlorine contact time in the existing clearwell. The clearwell is also the source of supply for surface wash and plant water.

8. **Post-Chlorination (Figure 9):** Gas chlorine is delivered from 150-lb cylinders via a vacuum feed system in a separate building adjacent to the treatment plant. The typical chlorine residual leaving the plant is about 1.5 mg/L. There is no onsite chlorine contact provided. After addition of chlorine the water passes through a concrete discharge vault that serves to combine flows from the WTP, infiltration gallery and Well No. 2. Chlorine contact for the system is provided in the transmission main to the distribution system.

Figure 9 - Chlorine Building

Table 18 - Existing Water Treatment Plant Process Descriptions

Process	Notes
Raw water supply	• Cottonwood Creek
Raw water intake	• Riverbank concrete diversion structure • Coated steel intake screen - 3” to 4” openings, manual-clean • Manually-operated gate valve for 24” intake pipe
Pre-sedimentation	• Lined, rectangular 1-MG presedimentation pond with rock weir wall near midpoint, overflow back to creek, backwash recycle return flow at midpoint, and perforated submerged effluent collection pipe
Pre-treatment chemical addition	• Polyaluminum chloride (Nalco 8157) coagulant: 15 to 30 mg/L (typ. dose) • Cationic polymer (Nalco 8102): 1 to 1.5 mg/L (typ. dose) • Streaming current monitor aids dosage control decisions
Rapid mixing	• Single in-line vertical mixer with impeller in filter influent pipe
Flocculation	• Single 12’x12’x10.5’-deep basin upstream of each filter • 11,300 gallons per filter (19.6 min. at 576 GPM) • Horizontal wooden plank baffles defining three compartments • Single vertical shaft flocculator with four 21” blades set in the middle of the second compartment of each basin. • Variable speed flocculator drive set to 15 rpm
Filtration	• Two (2) multi-media, gravity-flow filters in parallel • 144 sf of surface area per filter (4.0 GPM/sf at 576 GPM) • Media: 18” anthracite, 12” coarse and garnet sand, 15” gravel • Leopold underdrains • Surface wash arm at 2” above media • Filter operation: constant-level with automatic effluent rate of flow control; filters operated only one at a time historically • Backwashing initiated manually, typically based on headloss, although sometimes on turbidity breakthrough • Filter-to-waste not incorporated
Backwash pumping	• 1 constant-speed vertical turbine pump pulling from clearwell • Flow controlled by valve modulation • 2,500 GPM (17 GPM/sf) is typical rate • Backwash duration is 15 minutes
Backwash handling	• Lined pond with two cells, one for spent filter backwash from each filter • Dual submersible pump stations to recycle supernatant volume back to presed. pond
Clearwell	• Water volume: 33,000 gallons, maximum • No baffles (A/T ratio likely = 0.1) • No reliable contact time due to very low level after backwash • Source of supply also for surface wash and plant water
Post-chlorination	• Gas chlorine delivered from 150-lb cylinders via a vacuum feed system at a separate chemical building provides residual of about 1.5 mg/L leaving the plant site
Post-treatment blending	• Buried concrete “discharge vault” serves to combine flows from the WTP, the infiltration gallery, and Well #2.
Primary disinfection contacting	• No on-site contacting • Distribution system facilities, including 18” pipeline and 0.27-MG tank (for Ivy League zone only), provide contact time.

Given the good condition of the plant and equipment, recent upgrades to controls and backwash handling systems, improved pre-treatment chemical selection, and filter media replacement, the plant should be able to provide significant additional service life. Furthermore, based on raw water quality and pre-sedimentation pond performance, the plant's direct filtration process appears to be an adequate technology selection for the application - one that should be able to meet regulations for years to come.

At a minimum, when no recharge to the gallery is available (winter or call situation) and only one filter train is operational at the WTP, the town's current water production capacity is about 1.4 MGD. Given that peak day demand does not occur in the winter and the likelihood is very low of gallery recharge not being possible and one filter not being operational, this capacity should not be used to determine the need for additional production.

Because of the operational expense of running the WTP and the current production surplus (the infiltration gallery and Well #2 alone have been able to meet peak day demands recently), the town has not run the WTP regularly over the last several years. The WTP is only exercised periodically to keep it operable. However, it is reasonable to assume that during peak demand season a situation could be encountered where either a filter at the WTP is off-line due to an operational problem or the gallery is unable to be recharged for some reason. In this situation, the system production capacity could be limited to 2.11 to 2.15 MGD. Table 19 shows theoretical production capabilities in these scenarios.

Table 19 - Existing WTP Production Scenarios

Source	No Gallery Recharge		With Gallery Recharge	
	1 Filter (GPM)	2 Filters (GPM)	1 Filter (GPM)	2 Filters (GPM)
Treatment Plant (Cottonwood Crk.)	520	1,040	520	1,040
Infiltration Gallery (Groundwater/ recharged ditch water)	300[1]	300[1]	800[1]	800[1]
Well #2 (Alluvial groundwater)	150	150	150	150
Total (GPM)	**970**	**1,490**	**1,470**	**1,990**
Total (MGD)	**1.4**	**2.15**	**2.11**	**2.86**

1. Gallery output range reported as 200 to 850 GPM with 200-500 GPM as the range without recharge. Here, a representative low-end value is used for the "no recharge" condition and a representative high-end value for the "with recharge" condition.

For a reliable water supply, the town should plan to have a firm system-wide capacity that is adequate to meet its annual peak day water demands.

In the future, as long as the infiltration gallery water does not require filtration, the town will likely be best served to continue using its supplies in the same order it currently does; however, production levels from the WTP will need to increase as peak demands increase over time. This places an ever-increasing importance on firming-up the town's ability to process Cottonwood Creek water through the plant.

6.4 EXISTING WATER TREATMENT PLANT PROCESS IMPROVEMENTS

There are a number of areas for potential future improvement to the BVWTP which will increase treated water quality and reduce operation and maintenance costs.

6.4.1 Chlorine Contact

Currently there is no dedicated chlorine contact tank provided at the treatment plant. A dedicated on-site chlorine contact vessel for plant finished water would allow straightforward primary disinfection compliance demonstration and de-couple treatment and distribution, thus providing increased flexibility to modify and operate the distribution system to focus on water supply and hydraulics considerations. While distribution system operations are not currently hindered by this since adequate detention occurs in the transmission main prior to the first user, the potential for future constraints to arise exists. Also, increasing water demands and production rates will eventually require that new contact volume be added to meet primary disinfection requirements.

6.4.2 Chlorine Monitoring

Whether or not dedicated on-site chlorine contacting is provided, the town should provide the instrumentation needed to continuously monitor chlorine residual (for CT determinations) before the BVWTP is returned to service. With the existing system configuration, this means on-line chlorine residual analyzers and recording capabilities at Westmoor pump station. An on-line analyzer is already installed on the Ivy League Booster Pump Station.

6.4.3 Filter Backwash and Filter to Waste

A second back-up backwash pump should be added to increase plant reliability; this will be especially important as the town's growing demands force increased reliance on WTP production.

Also, expanded clearwell capacity should be considered to allow for back-to-back filter backwashing; similar to the addition of the second backwash supply pump, this firms up the plant's production reliability. A clearwell expansion project could be coupled with a dedicated on-site chlorine contacting project for construction efficiency.

Filter-to-waste capability should be considered to (a) allow a filter to come back on-line after backwashing without requiring an extended resting period, and (b) to provide another tool to meet ever-increasingly-stringent filtered water turbidity regulations. A more detailed analysis of plant hydraulics and operational patterns is needed to determine the economic and technical feasibility of providing filter-to-waste.

6.4.4 Flow Metering

To facilitate treatment optimization and system efficiency efforts, the town should consider installing a new backwash supply flow meter. These improvements would allow the town to more confidently and accurately perform water accounting calculations. The backwash supply meter would also allow straightforward adjustment of backwash supply rates that should be made in response to seasonal water temperature differences.

6.4.5 Treatment Building HVAC

The treatment building has no significant ventilation system. The building houses significant electrical and control equipment and contains open process basins. For the comfort and health of operations staff and to increase the service life of electrical equipment, the town should install an engineered ventilation system if it intends to extend the life of the plant significantly. The plant has a single gas-fired unit heater, which appears to be suffering from corrosion due to the lack of ventilation in the facility. The town should consider budgeting for a replacement and/or second heater.

6.4.6 Flocculation Basins

Even though baffles were added to the flocculation basins, water enters and leaves the 10.5' deep flocculation basins near the top of the basin, which likely promotes short-circuiting. A possible low-cost improvement to the basins would be to extend the influent water pipes downward to within a couple of feet of the floor. This would force water flow to more completely utilize the full vertical dimension of the flocculators. This is most important during spring runoff when water is colder and typically benefits from increased flocculation time.

6.4.7 Raw Water Screening

The raw water intake structure is one of the few areas of the plant that does not appear to be in good repair.

At a minimum, the existing corroded and damaged bar screen should be replaced. The town could look into the installation of a fully submerged slotted stainless steel intake screening with compressed air burst cleaning mechanism. However, these systems are also subject to failure and are significantly more expensive. Such a system may suffer fewer issues with surface ice, however. Given the WTP's large raw water pond, major improvements to the raw water intake do not appear to be a very high priority.

6.4.8 Fluoride Feed System

Fluoride feed system: the town historically fed fluoride to the potable water from a hydroflourosilicic acid system located in the chemical building. There are advocates of fluoridation in the community, but due to concerns over storing incompatible chemicals (caustic and acid) in the same building without real containment, the fluoride system was removed. If the town is committed to fluoride addition, a new system could be installed in a dedicated new room added to the chemical facility. This appears to be the most appropriate solution, as compared to engineering the caustic and fluoride systems to have adequate containment, given the space constraints in the existing chemical building.

The town should consider the following treatment facility rehabilitations at a minimum prior to restarting the WTP:

- Install a new surface water intake structure
- Rehabilitate and line the existing pre-sedimentation basin

- Install two (2) clarifiers with domed covers
- Install a new polymer feed system
- Install a new caustic feed system
- Install a new static mixing system
- Install piping and valves to allow filter-to-waste
- Install an emergency generator
- Add additional space to the treatment building to house chemical feed systems

A cost estimate for rehabilitating the existing water treatment plant with these improvements is shown in Table 20. This rehabilitation will bring the plant into modern treatment standards. Which would be recommended should it be desired to operate to 2026. The plant is not expected to be needed until the year 2026, however, it is likely that, by that time, a membrane plant and other advanced treatment techniques may be required. This is discussed further in this section.

Table 20 - WTP Rehabilitation Cost Estimate

EXISTING WATER TREATMENT PLANT IMPROVEMENTS AND RE-COMMISIONING				ENGINEER'S ESTIMATE	
ITEM	DESCRIPTION	QTY	UNIT	UNIT PRICE	SUBTOTAL
1	SURFACE WATER INTAKE STRUCTURE	1	LS	$ 15,000	$ 15,000
2	PRE-SEDIMENTATION BASIN REHAB: SYNTHETIC LINER, PIPE BOOTS, VENTS, ETC.	51,240	SF	$ 2	$ 92,232
3	PRE-SEDIMENTATION BASIN REHAB: REMOVE EXISTING CONCRETE SIDEWALLS	1	LS	$ 35,000	$ 35,000
4	CLARIFIER: CLARIFIER AND EQUIPMENT	2	EA	$ 340,000	$ 680,000
5	CLARIFIER: DOME COVER	2	EA	$ 25,000	$ 50,000
6	POLYMER FEED SYSTEM[1]	1	LS	$ 85,000	$ 85,000
7	CAUSTIC FEED SYSTEM[1]	1	LS	$ 60,000	$ 60,000
8	STATIC MIXING SYSTEM	1	LS	$ 70,500	$ 70,500
9	FILTER TO WASTE SYSTEM[1]	1	LS	$ 40,000	$ 100,000
10	EMERGENCY GENERATOR	1	LS	$ 65,000	$ 65,000
11	MISC. YARD PIPING AND VALVES	1	LS	$ 50,000	$ 50,000
12	RECOMISSION EXISTING TREATMENT PLANT / NEW SCADA SYSTEM	1	LS	$ 120,000	$ 120,000
13	TREATMENT BUILDING ADDITION	2,500	SF	$ 95	$ 237,500
			SUBTOTAL		**$1,660,232**
1	MOBILIZATION/DEMOBILIZATION/SITE RESTORATION (10%)	1	LS	$ 166,023	$ 166,023
2	CONSTRUCTION SURVEY (1.5%)	1	LS	$ 24,903	$ 24,903
			SUBTOTAL		**$190,927**
			SUBTOTAL CONSTRUCTION COST		**$1,660,232**
			ADD 20% CONTINGENCY		**$332,046**
			SUBTOTAL CONSTRUCTION COST + CONTINGENCY		**$1,992,278**
			ENGINEERING - PERMITTING AND DESIGN		**$175,000**
			CONSTRUCTION MANAGEMENT SERVICES (8% OF CONSTRUCTION COST)		**$132,819**
			TOTAL ESTIMATED PROJECT COST		**$2,300,097**

6.5 HEALTH-BASED REGULATIONS AND WATER QUALITY ISSUES

The primary purpose of a water treatment plant is to clarify (remove particles) and disinfect the water supply. This section summarizes key particle removal regulations, existing WTP performance, and planning implications in this area.

The latest turbidity limits of the Long-term 1 Enhanced Surface Water Treatment Rule (LT1 Rule), which became effective for the town in January 2005, are:

i. LT1 Rule combined filter effluent (CFE) turbidity Maximum Contaminant Levels (MCLs) are:

    1. ≤ 0.3 Nephelometric Turbidity Units (ntu) - 95% of 4-hour readings

    2. ≤ 1.0 ntu - 100% of all continuous readings

ii. LT1 Rule individual filter effluent (IFE) turbidity MCLs are:

    1. ≤ 1.0 ntu - any two consecutive 15-minute readings

    2. ≤ 0.5 ntu - any two consecutive 15-minute readings taken 15+ minutes after a backwash.

    3. Since the BVWTP has only two filters, the town can monitor and comply with the IFE turbidity limits at the CFE location (the LT1 Rule does not require monitoring of individual filter effluent turbidity for WTPs that have only one or two filters)

o **<u>Implications</u>:**

- The LT1 Rule is a new regulation for the town (it was not in effect when the plant was last regularly used). Though the plant can consistently produce less than 0.1-ntu filtered water during periods of good raw water quality, the 0.3-ntu CFE MCL will require the town to carefully monitor and control the filtration process during higher raw water turbidity periods (spring runoff) during the first few years of resumed operation to re-establish a solid operational understanding of plant performance under this new regulation.

- The need for filter-to-waste capability may be greater than it was prior to the LT1 Rule due to the limits on peak turbidity levels on the 15-minute monitoring basis.

- One of the biggest benefits of membrane filtration, a potential future treatment process for the town's WTP, is the ability to consistently meet these turbidity MCLs, virtually independent of influent water quality conditions or plant hydraulic loading rates.

The Surface Water Treatment Rule (SWTR) and the LT1 Rule require that surface water treatment plants achieve specified reductions in the levels of three different microorganisms/organism classes. The total disinfection levels must be met through physical removal and/or inactivation of the microorganisms. The required levels of disinfection (with the typical breakdown of needed log removal and inactivation credits for a direct filtration plant) are:

i. *Giardia*: 3-log total disinfection (2.0 - removal; 1.0 - inactivation)

ii. Viruses: 4-log total disinfection (2.0 - removal; 2.0 - inactivation)

iii. *Cryptosporidium (Crypto.)*: 2-log total disinfection (2.0 removal)

CDPHE assesses achievement of the above-noted log removals, in part, via annually required Microscopic Particulate Analysis (MPA) removal results. When the BVWTP operated regularly prior to 1998, it rarely was able to demonstrate through annual MPA removal measurements that it met the required pathogen log removal requirements. However, the 2001 Comprehensive Performance Evaluation (CPE) performed at the BVWTP after the most recent set of plant improvements was completed indicated that the plant's processes were suitably designed and well-operated, such that the town is generally able to maintain excellent filtered water quality.

- **Implications:**
  - Annual MPA removal testing will likely continue to indicate low MPA log-removal values at the BVWTP due to low influent MPA levels (given analytical detection limits, it is difficult to demonstrate high log removal of MPA counts when influent values are very low). Optimization of the BVWTP filtration process and maintenance of low filtered water turbidity levels will continue to be critical for demonstrating compliance with the SWTR and LT1 Rule.

The Long-term 2 Enhanced Surface Water Treatment Rule (LT2 Rule) was finalized in January 2006 and tightens requirements for Cryptosporidium disinfection.

i. Total *Cryptosporidium* disinfection credit requirements are a function of source water *Cryptosporidium* concentrations (greater *Crypto*. disinfection is required for waters with higher *Crypto*. levels) as determined by rule-required source water sampling.

ii. Source water sampling will be required at the BVWTP with the following compliance deadlines:

   1. A sampling plan must be submitted to CDPHE for systems serving less than 10,000 people by July 1, 2008.

   2. One year of biweekly *E. Co*li sampling starting by October 1, 2008 is the minimum for systems serving < 10,000 people unless the *E. Coli* level exceeds 50 per 100mL, in which case 1 to 2 years of *Cryptosporidium* monitoring is required starting no later than April 1, 2010.

The BVWTP design includes the recycle of spent filter backwash water to the head of the plant, specifically, to the middle of the pre-sedimentation pond. Because of this recycle feature, federal and state requirements regulating the practice of recycle apply to the BVWTP. Section 7.4 of CDPHE's Primary Drinking Water Regulations (as amended January 19, 2005) contains the requirements of USEPA's Filter Backwash Recycle Rule (FBRR) as implemented in Colorado. These recycle regulations require the town to report (to CDPHE) and maintain on-site records regarding the recycle of spent filter backwash water at the WTP.

- **Implications:** Key recycle provisions applicable to the BVWTP (for complete details, see Section 7.4 of Colorado *Primary Drinking Water Regulations*) include:
  - Notification to CDPHE by 12/8/2003 that backwash recycle is practiced, along with submission of a plant schematic highlighting selected items.
  - Reporting of various plant flow rates, including recycle flows.

- Collection and maintenance at the plant of records to be available for CDPHE staff to review (during sanitary surveys, primarily) regarding: recycle notification paperwork, list of recycle flows/frequencies, backwash flows and durations, filter run lengths, recycle flow treatment processes, data on physical characteristics of spent backwash handling/recycling facilities.

6.5.1 Pathogen Inactivation for All Sources

Section 10.6 of Colorado's Primary Drinking Water Regulations, per the federal SWTR, requires that chemical disinfection performance be tracked/verified daily by calculation of concentration-time (CT) ratio for Giardia and virus during the peak hourly flow period.

- **Implications:** This is a critical regulatory, recordkeeping, and process understanding issue that needs to be addressed as soon as the BVWTP is returned to service. Furthermore, while CDPHE has not historically required submission of CT values in monthly reports or checked during sanitary surveys that systems are tracking CT and meeting CT requirements, CDPHE staff has indicated that this is likely to change in the near future.

For surface water sources treated by a direct filtration process, the SWTR requires that 1-log of Giardia inactivation credit and 2.0-logs of virus inactivation credit be achieved (with free chlorine, in the case of the BVWTP) prior to the first customer. Achievement of the required inactivation levels is based on a CT (disinfectant Concentration times contact Time) calculation approach. CDPHE also has a minimum 30-minute free chlorine contact time design requirement that applies to both surface water and groundwater sources.

- **Implications:**

    - The town needs to provide the greater of 1.0-log of Giardia inactivation and 30 minutes of contact time prior to the first customer for all the water leaving its surface water treatment plant. Of these two requirements, the 1-log Giardia inactivation requirement will typically control. The CT required to achieve the 1-log Giardia inactivation is dependent upon the pH, temperature, and chlorine residual at the point in the system representing the end of the contact time period. For the current BV water system configuration, the location of this controlling compliance point changes depending upon how the system is operated on a given day. This is because the town currently has to rely upon the 18"-diameter transmission line (for all water) and its 0.27-MG tank (for water going to the Ivy League system) for chlorine contact time. For example, under the current limiting condition scenario of maximum day demand (1.5 MGD), this controlling point will be at the Ivy League BPS if that station is running at capacity (two pumps) since this decreases the time through the 0.27-MG tank; or, this point will be at the Westmoor BPS if the Ivy League station is running at less than capacity. The Ivy League station running at capacity on peak day defines the least amount of contact time available prior to the first customer as compared to any other operating scenario. In this scenario, when the BVWTP is running, the town needs to maintain a minimum chlorine residual of 1.1 to 1.2 mg/L at the Ivy League BPS (given the assumptions of the analysis) for regulatory compliance with the SWTR.

- Because the true point-of-entry (POE) to the system, given disinfection compliance determinations, is at the Ivy League BPS or the Westmoor BPS, depending upon system operating conditions, the town will be required to monitor and record chlorine residuals at these locations whenever it is operating its surface water treatment plant. The town should install continuous chlorine analyzers at these system locations.

- This Master Plan should evaluate the potential for the town to de-couple its treatment and distribution infrastructure by providing all the needed disinfection credit for its surface supply before the water mixes with the groundwater supplies at the plant site, or by providing at least all of the Giardia and Crypto. credit needed upstream of the mixing point (since virus inactivation with chlorine requires minimal CT). Mixing with the groundwater supplies prior to chlorine contacting increases the overall amount of contact volume needed. Providing dedicated chlorine contacting will simplify compliance demonstration and provide a distribution system that can be operated and expanded with ease and flexibility in the future.

- The above-noted suggestion to separate disinfection contacting from distribution becomes even more sensible as demands and production rates increase. A CT analysis of a 3.4-MGD buildout demand scenario (with all water production coming from the three supply sources on the existing WTP site) indicates that the town would need to provide a minimum chlorine residual of 2.7 mg/L at the Westmoor BPS on peak day to achieve compliance unless infrastructure improvements to increase effective contact volume were made (or much future additional production capacity was introduced to the system at locations separate from the current plant site). Such a chlorine level would likely cause customer taste/odor complaints and raise Disinfection By-Products (DBP) concentrations. It should be noted that even if the 2.7-mg/L chlorine residual were provided, the contact time would be less than 30 minutes (currently a CDPHE requirement for all waters); therefore, unless CDPHE relaxes the 30-minute requirement when it moves to enforce a CT-based disinfection approach, additional contact volume will be needed at the plant no matter what chlorine residual is carried whenever flows leaving the plant site become greater than about 2.35 MGD (1,630 GPM).

As noted above, CDPHE has a minimum 30-minute free chlorine contact time design requirement that applies to groundwater as well. For groundwater sources, systems can apply to CDPHE, and in some cases receive, a waiver for the 30-minute contact time requirement for if the supply is determined to be at no significant risk of surface contamination. Given the physical characteristics of the town's infiltration gallery source, it is unlikely that such a waiver would be granted.

- <u>**Implications:**</u>

  - If the town's WTP is running and total flows are less than 2.35 MGD, the 1-log Giardia inactivation requirement for surface water will control and the 30-minute contact time requirement that applies to the town's groundwater is not critical (assuming the WTP and groundwater streams are combined as they currently are).

- If the WTP is NOT running or it is running and the total flow to the system is more than 2.35 MGD, the 30-minute contact time becomes the controlling requirement for primary disinfection. A contact time analysis indicates that if the well is running at 150 GPM and the gallery at 850 GPM, there would be 50 minutes of effective contact time available through to the discharges of both the Ivy League and Westmoor pump stations. So, the 30-minute requirement is easily met with existing infrastructure when only the groundwater supplies are running.

- If the town moves to provide dedicated on-site disinfection contacting for the WTP finished water, it would still need to provide contact time in the distribution piping for the groundwater supplies. Also, continuous chlorine residual analysis would be needed in the distribution system for the groundwater supplies to demonstrate that a residual greater than 0.2 mg/L is being achieved continuously at the effective POE for these supplies.

The USEPA's Groundwater Rule tightened disinfection requirements for groundwater. The rule focuses on increasing disinfection requirements for groundwater supplies with hydrogeologic conditions that are sensitive to surface contamination or supplies in which coliforms have been detected. The proposed rule requires any groundwater that currently does not provide 4-log virus inactivation for primary disinfection to perform hydrogeologic assessments and sampling to determine whether or not the supplies are in a sensitive environment. If they are found to be at higher risk of contamination, 4-log virus inactivation may be required.

- **<u>Implications:</u>**

  - The town's infiltration gallery groundwater source has been classified under the Groundwater Rule as not being hydrogeologically sensitive to surface contamination and therefore not under the influence of surface water.

  - The town will still need to continuously monitor chlorine residual to demonstrate achievement of adequate CT. For this reason, the town may wish to provide dedicated 4-log virus chlorine contacting on the WTP site for the combined groundwater supplies.

The recently-promulgated LT2 Rule may require additional Crypto. disinfection credit for the surface water supply. Requirements will depend upon the results of required source water sampling, as previously discussed.

- **<u>Implications:</u>**

  - If source water pathogen levels are low enough the town may not have to provide any additional Crypto. disinfection.

  - If source water Crypto. levels are significant enough to trigger additional Crypto. disinfection, but not high enough to warrant the installation of expensive advanced treatment technologies such as UV disinfection or membrane filtration, the town may be able to achieve compliance through a combination of a variety of methods such as maintaining low (<0.15-ntu) individual and/or combined filter effluent turbidity levels, implementing an acceptable source water protection program, installing coagulation upstream of pre-sedimentation, drawing water from the river through riverbank wells or infiltration galleries, and/or demonstrating sufficient Crypto. removal/disinfection through the treatment process through a special study.

- If the additional level of Crypto. disinfection required is high enough (> 1.5 logs), the town will need to consider installation of membrane filtration or UV disinfection.

The LT1 Rule requires that all surface water systems serving less than 10,000 customers develop a disinfection profile and file it on-site for availability to CDPHE staff during sanitary surveys. The purpose is to document the level of disinfection provided, so that if in the future a system needs to change its disinfection practice in order to achieve compliance with a disinfection by-product regulation, it will not do so by significantly compromising the historical level of disinfection provided. This requirement is a key to USEPA's push to ensure both adequate disinfection and disinfection by-product control. A waiver from the disinfection profiling requirement is available to systems with no individual Trihalomethane (TTHM) and Haloacidic Acids (HAA5) measurements greater than 0.064 mg/L and 0.048 mg/L, respectively, at the representative maximum distribution system location during the month of warmest water temperature since January 1, 1998.

- **Implications:**
  - The town has not had the opportunity to develop a disinfection profile because its WTP has not operated since the LT1 Rule became effective.
  - If a profile is required, staff should develop a disinfection profile over a 12-month calendar period once the town resumes regular WTP operations. Data need to be collected and calculations made on a weekly basis during the 12 months. For weeks when the plant is not run, no profile can be calculated. If there are months when the plant is not run, the town should plan to do profiling in those months during a future year when the plant is run in that month. Note that a disinfection profile involves CT determinations, which require chlorine, pH, and temperature data from the end of the primary disinfection segment, which is in the distribution system for the town. This means that the town will need to implement monitoring improvements or move to on-site disinfection contacting before it can develop a profile.

6.5.2 Disinfection By-Products

The Stage 1 Disinfectants/Disinfection Byproducts Rule (Stage 1 Rule) became effective for the town in January of 2004. The town has had no problem complying with the MCLs of the Stage 1 Rule, but has not yet had to demonstrate compliance with the total organic carbon (TOC) removal, "enhanced coagulation," treatment technique requirements at its WTP yet. Furthermore, the surface water plant likely has filtered water TOC levels that are higher than those observed in the infiltration gallery effluent water. In June 2006, TOC sampling indicated that the infiltration gallery influent and effluent TOC values were 1.5 and 0.8 mg/L, respectively. A TOC removal percentage of 47% may not be easily achieved through the direct filtration treatment plant at an influent TOC level of 1.5 mg/L. Therefore, historical DBP levels measured in the distribution system may not be a good indicator for future compliance with the Stage 1 Rule once the BVWTP is back on-line. This is because higher TOC levels at the point of chlorination produce higher DBP levels.

- **Implications:**
  - The town should monitor TOC levels across the infiltration gallery and in filtered creek water to develop an estimate of how DBP levels may be affected, and whether a significant impact on Stage 1 DBP Rule compliance status is anticipated, once the BVWTP is running again.

USEPA finalized the Stage 2 Disinfectants/Disinfection Byproducts Rule (Stage 2 Rule) in January 2006. The Stage 2 Rule tightens regulations for DBPs by making compliance based on the running annual average (RAA) value of the location with the highest RAA value (this is called a "locational" running annual average (LRAA) compliance basis). The MCL values remain as follows:

- Total Trihalomethanes (TTHMs) MCL: 80 ppb
- Sum of 5 Haloacetic Acids (HAA5) MCL: 60 ppb
- These MCLs are the same values as set by the Stage 1 Rule - it is the compliance methodology that has changed.

- **Implications:**
  - The town will need to comply at its WTP with the enhanced coagulation provisions of the Stage 1 DBP Rule. This means demonstration of TOC removal or compliance using alternative compliance criteria (most likely the 2.0-mg/L treated water TOC criterion) on a running annual average basis using monthly values. This will require monthly analysis of raw and finished water TOC and raw water alkalinity. It is not anticipated that the town will have any problem complying with TOC removal requirements so long as it is coagulating, flocculating, and filtering sufficiently to meet filtered water turbidity requirements of the LT1 Rule.
  - Because of its small population served, the town's water system has only been required under the Stage 1 D/DBP Rule to sample at a single site. However, for systems that choose to sample at a single location, the Stage 1 Rule requires that it be the representative maximum residence time site. The town has been sampling at a site representative of its maximum residence time. The Stage 2 Rule bases compliance with MCLs on the location(s) in the system with the highest DBP levels. While the town's compiled historical Stage 1 Rule compliance data indicate that maximum DBP levels have been low enough to result in future Stage 2 Rule compliance, the DBP data thus far have been representative of a system operating condition that does not include the BVWTP. The BVWTP will likely increase DBP formation in the system (as discussed in the previous section). When the WTP is used more regularly in the future, distribution system DBP levels may rise. This will be especially true if the plant is used during peak runoff times or after summer storms because total organic carbon (TOC) levels (DBP-precursors) are typically at their peak values during these times. For this reason, and due to concurrent high raw water turbidity levels, the town may wish to operate the system in the future

with the WTP off-line, or running at a reduced flow rate, during peak runoff and summer storm periods. Furthermore, the town should carefully track its DBP levels when the plant is brought back on-line to ensure that Stage 1 or Stage 2 DBP Rule compliance will not become a problem.

- The town will has performed an initial distribution system evaluation (IDSE) to locate sites with maximum TTHM and HAA5 levels in the distribution system. Sites with higher levels than those historically sampled may be identified (though it is unlikely that significantly higher levels stand to be found). Furthermore, the geographical spread of the town's distribution system will increase as the town grows, and so the controlling DBP location is likely to move further out with the maximum concentrations, especially for TTHMs, increasing as this occurs.

- For the above-noted reasons it will be important that any treatment plant modifications or new treatment plant designs incorporate the ability to remove organic carbon from the water prior to chlorination. More specifically, should the town decide to use membrane filtration, it should include provisions for chemical coagulation, mixing, and detention time upstream of a membrane filtration system.

As noted above, the Stage 2 Rule requires water systems to perform an initial distribution system evaluation (IDSE) to identify distribution sampling sites with peak DBP concentrations. These sampling sites subsequently are used for Stage 2 Rule compliance monitoring.

6.5.3 Other Health-based Rules / Parameters

MCL or other violations for current and foreseeable future health-based water quality regulations are unlikely for the town (however, the town does need to ensure that it complies with all monitoring, reporting, and record keeping provisions). This is primarily true because of source water quality (i.e. regulated or potentially future regulated chemicals or chemical classes have not been detected in the town's groundwater sources or Cottonwood Creek). This includes existing rules for:

i. Organic chemicals

   1. Volatile organic chemicals (VOCs)

   2. Synthetic organic chemicals (SOCs)

ii. Inorganic chemicals (IOCs)

   1. Including arsenic, nitrate, nitrite, fluoride, and others

iii. Radionuclides

Endocrine-disrupting (ED) chemicals, such as pesticides and compounds found in pharmaceuticals and personal care products, have become an emerging focus of water quality in the United States and Europe. This is an area of potential future regulation. The most affected water systems likely will be those whose source waters receive significant wastewater discharges. Recent research has found that in such receiving waters, concentrations of EDs can be "elevated" and detrimental effects on fish health have been observed. EDs will continue to be a concern and a target for future regulations.

- **Implications:**
  - The town should be cognizant that future wastewater discharges and pesticide applications in the area of its infiltration gallery supply could contribute to an ED problem. Land-use-based protections will be the key to maintaining the long-term quality of the infiltration gallery supply.

The town has to comply with the provisions of the Lead and Copper Rule (LCR). It has had a corrosion control plan in place for a number of years to achieve LCR compliance. The plan involves pH adjustment of infiltration gallery water to a finished water target value of about 7.8.

- **Implications:**
  - The town will need to continue with corrosion control measures for the infiltration gallery supply.
  - town staff has expressed desire for a soda ash system to increase the alkalinity of the supplies to stabilize the pH adjustment process.

6.5.4 Aesthetic-based Standards and Water Quality Issues

- **Iron Removal**
  - Low levels of iron exist in Cottonwood Creek water. CDPHE has adopted the federal secondary standard for iron of 0.30 mg/L to protect against colored/red water in the distribution system. Though the town does not use pre-oxidation at its WTP, and therefore no dissolved iron is removed, the levels present in finished water do not appear to be high enough to have caused colored water in the distribution system.

6.6 FUTURE WATER CAPACITY & TREATMENT

The town will need to take the necessary steps to meet projected potable water demand pressures. As noted, this means reducing per capita water consumption through conservation, bringing additional firm production capacity on-line, and/or reducing non-potable demand pressures on the potable system.

The additional firm production capacity could be provided by:

- firming existing supplies through
  - improved treatment plant production reliability (i.e. being able to reliably run two filters continuously), and
  - improved infiltration gallery operational reliability (which is primarily a water rights and source water protection issue), and/or

- constructing new production capacity through
  - a treatment plant expansion, and/or
  - adding new wells

Other than widespread water conservation measures, the most feasible ways to reduce future peak day demands on the town's potable system are:

- developing dedicated non-potable irrigation water supply systems for existing large open spaces (for example the downtown ball fields), and/or
- requiring new developments to provide secondary raw water irrigation systems

There are several factors, whose outcomes currently are uncertain, which will significantly influence how the town's water system should evolve to best meet the future's production and water quality requirements. These factors make it difficult to establish a detailed long-term infrastructure plan for the potable system at this time. A discussion of these factors is presented below:

Because of natural filtering provided by subsurface aquifer materials, groundwater can be of higher quality than surface water. This is not always, but is often the case. The higher quality often results in decreased treatment requirements for water produced from a well than water diverted from a surface source. These decreased treatment requirements typically lead to reduced initial capital costs for developing the resource and decreased on-going operational and maintenance costs for utilizing it. For these reasons, it serves water providers well to consider first the feasibility of developing additional well capacity when expanding overall system capacity.

Because the infiltration gallery and Well #2 have long been an adequate supply for the town, not much effort has been expended investigating the possibility of additional groundwater production facilities. While many small domestic wells have been drilled within the town, there is no public record of large capacity production wells having been drilled or attempted to be drilled. Therefore, at this time there is considerable uncertainty regarding the likelihood and extent to which the town will be able to develop wells in the future to meet potable system demands.

This Master Plan recommends that the town undertake the required studies and physical investigations to evaluate the feasibility of developing additional groundwater supplies in both the Arkansas River and Cottonwood Creek basins. The recommended well evaluations must include considerations of land ownership, ease of tie-in to the existing system, well yield/aquifer characteristics, water quality, cost of water, and especially, water rights. Water rights considerations alone could result in the best possibility for a new municipal well being in the Arkansas River basin with a junior water right that is augmented with contract water so as to be able to be run as a peaking supply only.

Depending on the results of required future source water sampling, the town may need to provide additional Cryptospordium disinfection for Cottonwood Creek water than is currently provided at its WTP. The WTP's treatment process is otherwise able to satisfactorily meet all current regulations given the quality of the raw water (with the noted exception that off-site transmission pipelines and tanks are needed for adequate chlorine disinfection credit to be achieved). Furthermore, providing significant additional Cryptosporidium disinfection credit (i.e. greater than 0.5 to 1.0 logs more) will require expensive new upgrades to the WTP. For these reasons, the results of the above-noted source water sampling are critical to determining future surface water treatment needs for Buena Vista. Therefore, this Master Plan recommends that the town execute the required sampling as soon as practicable (i.e. in advance of regulatory deadlines) to develop a clear understanding of treatment infrastructure requirements.

Both surface water and groundwater have been targets of increasing regulatory focus. Treatment requirements for both have been becoming more stringent and are increasingly being based on site-specific water quality and/or physical characteristics of the supply. The nature of this focus makes it conceivable that the town's infiltration gallery could be regulated much more stringently in the future. This is primarily based on its physical characteristics and observed behavior. If the infiltration gallery water requires additional treatment, this affects the most efficient way to treat directly-diverted surface water and the infiltration gallery water at the plant site. However, to-date there are no upcoming, proposed or final regulations that will require anything more than chlorination of the supply. Since filtration or other advanced treatment is not likely to be required for the infiltration gallery within the next 10 years (at least), it is only the long-term uncertainty in requirements that may impact the planning process. This Master Plan will assume that additional filtration or advanced treatment will not be required for the infiltration gallery over the planning horizon; however, this Master Plan recommends that any planned improvements to the plant site be conceived with the possibility that infiltration gallery flows may eventually require more advanced treatment.

This section presents recommended production/treatment-related projects given a reasonable "worst-case" planning scenario. The goal is to develop a basis for conservative estimated infrastructure costs to serve as a starting point for rate and fee setting processes. Such a scenario for the purposes herein is defined as:

- Water rights constraints, inadequate physical supply, and poor water quality prohibit the town from being able to develop any additional groundwater supplies for the potable system in the future.
- The town is able to develop a non-potable well tributary to the Arkansas River for watering current and future ball fields downtown. This reduces potable system water demand by approximately 1.5 million gallons in the peak month, or 0.05 MGD on the peak day.
- Source water sampling on Cottonwood Creek requires that the town achieve an additional amount of Cryptosporidium disinfection that essentially requires treatment with either membrane filtration or ultraviolet light (UV) disinfection.
- Water rights considerations require discontinuation/curtailing of infiltration gallery recharge practices, which limit the supply's reliable yield to about 300 GPM.
- No additional regulations or deteriorations in effluent water quality that require any more than 4-log virus inactivation by chlorine per a potential final Groundwater Rule for the gallery supply.
- The town is able to transfer sufficient water rights to Well #2 to cover a 150-GPM peak instantaneous diversion.

In this "worst-case" production/treatment scenario, the town would need to provide 2.1 MGD of firm capacity in a Cottonwood Creek surface water treatment plant at "buildout." This value is calculated as follows:

Projected "buildout" peak day potable system demand:	2.80 MGD
Potable system demand off-set due to ball field well(s):	- 0.05 MGD
Adjusted "buildout" peak day potable system demand:	2.75 MGD
Well #2 peak day contribution:	- 0.22 MGD
Infiltration gallery peak day contribution:	- 0.43 MGD
Remaining Demand to be met with WTP:	**2.10 MGD**

It is recommended, however, that if treatment is required a 3.0 MGD treatment plant be planned for to ensure a conservative approach. For the purposes of this planning exercise, it is assumed that the BVWTP's existing filters can be retrofitted with a submerged membrane filtration system in order to raise surface water production capacity to 2.1 MGD. This membrane filtration-based process improvement would allow the town to meet the Cryptosporidium requirements and the future demands primarily within the existing plant footprint. This essentially represents a doubling of the plant's filtration capacity. This magnitude of capacity increase is within typical capacity increases observed when outfitting existing granular mixed-media filter basin with submerged membrane equipment.

However, due to the additional space required for ancillary equipment (filtrate pumps, backwash air blowers, backwash supply tank, etc.) and membrane cleaning (and waste neutralization) chemical storage and feed systems and a membrane chemical cleaning waste neutralization basin, an additional building located adjacent to the main filter building would likely be needed. While it appears that the existing 18" filter building influent line is large enough to carry the needed 2.2-MGD raw water flow rate (2.1 MGD with 5% waste) to the flocculation basins, detailed hydraulic calculations through the plant will need to be made. Furthermore, membrane systems require a fine screen upstream to remove potentially membrane fiber-damaging debris. The headloss associated with this type of screen may necessitate raw water pumping. It is assumed herein that this will be needed though screening systems with lower headloss may ultimately be identified that could be placed at the head of the flocculation basins so that a pump station can be avoided.

Given that the existing flocculation and filter basins in each process train at the plant are connected by two pipes, it may be straightforward to split each of the two existing filter basins into two cells in order to have four independent membrane filter cells. Each cell could be outfitted with about 1 MGD of membrane filtration capacity, so as to achieve an overall firm capacity of 3 MGD with one filter cell out of service. Table 21 presents estimated capital costs to retrofit the existing BVWTP with 3 MGD of firm submerged membrane filtration capacity.

Table 21 - 3.0 MGD WTP Membrane Retrofit Cost Estimate

3-MGD MEMBRANE FILTER RETROFIT				ENGINEER'S ESTIMATE	
ITEM	DESCRIPTION	QTY	UNIT	UNIT PRICE	SUBTOTAL
1	SURFACE WATER INTAKE STRUCTURE	1	LS	$ 15,000	$ 15,000
2	PRE-SEDIMENTATION BASIN REHAB: SYNTHETIC LINER, PIPE BOOTS, VENTS, ETC.	51,240	SF	$ 2	$ 92,232
3	PRE-SEDIMENTATION BASIN REHAB: REMOVE EXISTING CONCRETE SIDEWALLS	1	LS	$ 35,000	$ 35,000
4	PRE-SETTLED WATER PUMP STATION	1	LS	$ 500,000	$ 500,000
5	3 MGD MEMBRANE FILTRATION PROCESS RETROFIT	1	LS	$ 3,300,000	$ 3,300,000
5	POLYMER FEED SYSTEM[1]	1	LS	$ 85,000	$ 85,000
6	ACID CIP SYSTEM[1]	1	LS	$ 50,000	$ 50,000
7	CAUSTIC FEED SYSTEM[1]	1	LS	$ 60,000	$ 60,000
8	STATIC MIXING SYSTEM	1	LS	$ 70,500	$ 70,500
9	FILTER TO WASTE SYSTEM[1]	1	LS	$ 40,000	$ 40,000
			SUBTOTAL		$4,247,732
1	MOBILIZATION/DEMOBILIZATION/SITE RESTORATION (10%)	1	LS	$ 424,773	$ 424,773
2	CONSTRUCTION SURVEY (1.5%)	1	LS	$ 63,716	$ 63,716
			SUBTOTAL		$488,489
	SUBTOTAL CONSTRUCTION COST				$4,247,732
	ADD 20% CONTINGENCY				$849,546
	SUBTOTAL CONSTRUCTION COST + CONTINGENCY				$5,097,278
	ENGINEERING - PERMITTING AND DESIGN				$175,000
	CONSTRUCTION MANAGEMENT SERVICES				$339,819
	TOTAL ESTIMATED PROJECT COST				$5,612,097

Though the BVWTP is in good repair, it is over 30 years old and treatment process technologies have advanced during that time. The alternative to rehabilitating the existing treatment plant and the possibility of major renovations is to complete construction of a new ultra-filtration water treatment plant that will be capable of serving the Town of Buena Vista for 20-years or more. The estimated cost for a new 3-million gallon per day ultra-filtration plant is shown in Table 22.

Table 22 – New 3.0 MGD Ultra-Filtration Cost Estimate

ULTRA-FILTRATION WATER TREATMENT PLANT (3 MGD)				ENGINEER'S ESTIMATE	
ITEM	DESCRIPTION	QTY	UNIT	UNIT PRICE	SUBTOTAL
1	SURFACE WATER INTAKE STRUCTURE	1	LS	$ 15,000	$ 15,000
2	PRE-SEDIMENTATION BASIN REHAB: SYNTHETIC LINER, PIPE BOOTS, VENTS, ETC.	51,240	SF	$ 2	$ 92,232
3	PRE-SEDIMENTATION BASIN REHAB: REMOVE EXISTING CONCRETE SIDEWALLS	1	LS	$ 35,000	$ 35,000
4	3 MGD ULTRA-FILTRATION TREATMENT SKID	1	LS	$ 3,500,000	$ 3,500,000
5	POLYMER FEED SYSTEM[1]	1	LS	$ 85,000	$ 85,000
6	ACID CIP SYSTEM[1]	1	LS	$ 50,000	$ 50,000
7	CAUSTIC FEED SYSTEM[1]	1	LS	$ 60,000	$ 60,000
8	STATIC MIXING SYSTEM	1	LS	$ 70,500	$ 70,500
9	FILTER TO WASTE SYSTEM[1]	1	LS	$ 40,000	$ 40,000
10	EMERGENCY GENERATOR	1	LS	$ 65,000	$ 65,000
11	MISC. YARD PIPING AND VALVES	1	LS	$ 50,000	$ 50,000
12	TREATMENT BUILDING	6,000	SF	$ 95	$ 570,000
			SUBTOTAL		$4,632,732
1	MOBILIZATION/DEMOBILIZATION/SITE RESTORATION (10%)	1	LS	$ 463,273	$ 463,273
2	CONSTRUCTION SURVEY (1.5%)	1	LS	$ 69,491	$ 69,491
			SUBTOTAL		$532,764
	SUBTOTAL CONSTRUCTION COST				$4,632,732
	ADD 20% CONTINGENCY				$926,546
	SUBTOTAL CONSTRUCTION COST + CONTINGENCY				$5,559,278
	ENGINEERING - PERMITTING AND DESIGN				$175,000
	CONSTRUCTION MANAGEMENT SERVICES				$370,619
	TOTAL ESTIMATED PROJECT COST				$6,104,897

6.7 SECTION SUMMARY

1. Several key unknowns prevent establishment of a firm capital improvements program for production and treatment infrastructure. To develop a clearer planning picture, the town should complete the following as soon as possible:

    a. Source water sampling at the water plant for E. Coli/ Cryptosporidium to determine the impact of the Long-term Surface Water Treatment Rule on future treatment needs.

    b. Well feasibility study - Arkansas and Cottonwood Creek basins.

2. Current max production requirement of about 1.4 MGD. Currently met using just the Infiltration Gallery and Well #2.

3. The nominal production capacity of the WTP is 1.5 MGD.

4. It is currently estimated that the town could face $5M (or more) in capital costs to upgrade and expand its production/treatment facilities over the next 15 years, or so, to meet growing water demands and increasingly stringent water quality standards. This will significantly impact both tap fees and water use rates.

5. Before the town's WTP is returned to service, the town should install the instrumentation and recording devices at two booster pump stations required to track the level of chlorine disinfection being achieved for adequate demonstration of regulatory compliance.

6. The town should rectify unaccounted water discrepancies this year and begin collecting data in which it has confidence. This will allow verification of water demand assumptions that serve as a basis of this plan.

7. There are a handful of recommended near-term upgrades to the treatment plant site to improve reliability, operations, and environmental conditions that this plan identifies. Most should be implemented as soon as possible.

7.0 WATER DISTRIBUTION SYSTEM

7.1 DISTRIBUTION PRESSURE ZONES

The Town of Buena Vista distribution system is comprised of two main gravity feed zones and one constant pressure zone. The two gravity zones are the Lower Zone and the Upper Zone. The constant pressure zone is fed from the Ivy League Pump Station and is therefore referred to as the Ivy League Zone. Water is pumped from the Lower Zone to the Upper Zone through the Westmoor Booster Pump Station.

There are three water storage tanks in the system. These are the 1.5 M.G. Lower Zone Tank, the 0.75 M.G. Upper Zone Tank and the 0.27 M.G. Ivy League Tank. The WTP gravity feeds the Lower Zone and Ivy League Tanks. These tanks gravity feed the Lower Zone. PRV stations located at Yale Street and Sangre De Cristo Lane, West Main and Sangre De Cristo Lane and Meadow Lane separate the Upper and Lower Zones, and allow water from the Upper Zone to feed back into the lower zone at a reduced pressure.

The Ivy League zone is pressurized by the Ivy League Booster Pump Station which feeds water from the 0.27 million gallon Ivy League Tank to the Ivy League distribution system. The Ivy League Tank is fed off of either the infiltration gallery by gravity or by Well No. 2. The pumping capacity for all pumps in the system is shown in Table 23.

Table 23 - Existing Pumping Capacity

PUMP	ELEV. (FT)	WELL DEPTH (FT)	DISCHARGE (GPM)	T.D.H. (FT)
Well 1 (at Rodeo Grounds)	8,400	~100	-	-
Well 2 (at WTP)	8,400	~100	150	-
Ivy League #1	8,400	N/A	350	100
Ivy League #2	8,420	N/A	350	100
Westmoor #1	8,255	N/A	60	90
Westmoor #2	8,285	N/A	120	90
Westmoor #3	7,950	N/A	350	90

Piping with the system ranges from 4" cast iron to 18" ductile iron. Installation dates also vary greatly with some lines installed back in the 1950's. According to town staff, there is no knowledge of internal scaling or exterior corrosion problems anywhere in the system.

7.2 DISTRIBUTION SYSTEM MODELING

An updated water model was not intended to be a part of this master plan. This section is intended to reiterate the findings of the original water model. The Buena Vista water system was originally modeled using Haestad Methods WaterCAD v6.5 in 2006. System layout and configuration was generated using digital "as-built" drawings, hard copy mapping and town staff. Demands were established using production records, billing records and building department information. Pumping capacities, tank levels, PRV settings, and standard operating conditions were established using District records and staff knowledge. All elevations came from town base mapping.

The hydraulic model was created using digital maps of the town water system. Elevations for appurtenances were ascertained from a hard copy of a system overview map prepared by Wright Water Engineers (WWE) dated 9/12/97. Topographic data was also taken from this WWE map. Information on system connectivity, pump capacities, appurtenance condition, number of service taps, and water usage was obtained from phone conversations with town staff.

Discussions with town staff disclosed that the finished water is essentially neutral and as such there is little to no deterioration of interior surfaces of system piping, not even in the old cast iron lines that were installed over 40 years ago.

7.3 DISTRIBUTION SYSTEM MODEL CALIBRATION & VALIDATION

Water model accuracy is increased by calibration with field results. This was accomplished by using recently completed fire hydrant flow testing that was completed as part of the ISO Evaluation. Model results were correlated by manipulating modeling parameters, such as roughness and minor losses.

Model calibration is typically considered complete when model results are within 10% of field results. In this case, the best results were obtained by adjusting roughness factors to their highest levels (c=150 for ductile iron pipe and c=144 for cast iron pipe) with no minor losses. Under this "best-possible-case", the model results were typically still lower than the field results. This was deemed acceptable for two reasons. First, the physical parameters could not be adjusted any higher. Second, even at the highest possible values, the model results were still generally lower than field results; conservative model results more appropriate for planning purposes. Potential developers should confirm actual field conditions prior to engineering and plan review. Results of the calibration results are presented in Table 24.

Table 24 - Fire Flow Calibration

FH Location	Field Test Results			Before Calibration		After Calibration	
	Flow	Static	Resid.	Static	Resid.	Static	Resid.
CR317 & Antero	1850	78	56	79	49	78	61
Harvard & Marquette	2060	74	60	73	38	74	54
Oak (btwn. Gunn. & San Juan)	2550	62	38	62	0.94	62	30
Gold (btwn. Belden & Court)	1910	66	44	64	-1.5	66	40
Court & Main	2280	64	48	64	44	64	54
Crossman & Pleasant	2190	54	40	56	22	54	35
McDonald & Thompson	1770	50	24	47	-9	50	16
Gunnison & W. Main	2020	64	56	62	51	64	60
Susan (btwn. Connie & Main)	2460	54	40	56	27	54	36
Shaman & Windwalker	1940	78	52	73	42	78	36

Town staff also recorded field settings for the two PRV's. Table 25 compares field results with predicted model results. While accuracy is desired for both the inlet and outlet pressures, outlet pressures are more critical as it governs system performance. Some discrepancy between field and model is expected due to inherent inaccuracy in field gauges and readings.

Table 25 - PRV Calibration

PRV Location	Field		Model	
	Upstream (psi)	Down (psi)	Upstream (psi)	Down (psi)
Yale	82	44	78	42
Main St. (CR306)	88	44	86	46

7.4 RESULTS OF MODELING

Steady State analyses are instantaneous evaluations for a given set of conditions, much like a snapshot of the system in operation. Steady State analyses were used to evaluate system performance, such as pressure and line velocities, as well as to evaluate fire flows at each node in each zone for the entire Buena Vista Water System. The model was run utilizing normal operating conditions and PRV settings to simulate "real world" conditions. This analysis establishes maximum flow rates that can be delivered to all nodes in the model without causing unsatisfactory performance to other pipes and nodes. Steady State analysis results can be found in the Appendix G, and individual results are presented in the specific zone discussions of this report. A map of the current water system and service boundary is included in Appendix H.

The system was evaluated by creating multiple scenarios of system conditions. Pump run status (on/off), tank level, and fire flow calculations were then manipulated to simulate worst-case conditions. These multiple scenarios were then used to both validate the model with known conditions and to determine system deficiencies. The results presented herein are the culmination of these multiple scenarios along with pertinent individual scenario results. Supporting modeling results are presented in the appendices of this report.

The following performance criteria are commonly accepted industry standards that were used to evaluate model results. Fire flow and system storage criteria are discussed in subsequent sections of this report.

- Velocity: < 5 fps at Maximum Daily Demand

  < 10 fps at Peak Hour Flow

- Pressures: > 40-psi at main at tap locations

  < 190-psi within service areas

- 225-psi max. @ pump discharge

- Line Size: 6 inch (minimum)

7.4.1 Lower Zone Evaluation

The Lower Zone is gravity fed from two storage tanks located immediately downstream of the town Water Treatment Plant. These tanks have individual capacities of 1.5 million gallons and 0.27 million gallons; the combined storage total for the Lower Zone is 1.77 Million gallons (MG). These tanks have a hydraulic grade line (HGL) of 8,104 feet. For planning purposes only, this HGL can generally serve elevations below 8,004 feet with minimum service of 40-psi.

All water supplied to the town flows through the Lower Zone tanks. In turn, water is pressurized by the Ivy League Pump Station for local service and by the Westmoor Pump Station for delivery to the Upper Zone. This means that the town Water Treatment Plant must deliver the entire MDD of the town to Lower Zone Tanks.

Per the ISO study, the highest current flow requirement in the Lower Zone is 3,500 GPM. However, it should be kept in mind that the system was design around older fire flow requirements that were much lower, say 500 to 1,500 GPM. For the purposes of this study, the system was evaluated against the ISO requirements (see Appendix A).

Fire flows of over 1,500 GPM are available to the majority of town, with all areas meeting the prescribed ISO requirements. There are a few notable low fire flow (500 to 1,000 GPM) locations that that are all associated with 4″ and 6″ mains. The low fire flow areas are:

- end of Ponderosa Place
- ends of Cottonwood Avenue and Centennial Plaza (south of Cottonwood Creek)
- end of California Street
- north end of HWY 24
- all of Arkansas Avenue south of the railroad
- south end of Colorado Ave by Main Street
- east end of Cedar Avenue
- end of McDonald Avenue
- area around Public Works on Gregg Drive

No other existing service concerns were identified in the Lower Zone. No excessive line velocities were indicated. No excessive high or low pressures were identified in the existing system. Service pressures generally range from 40 to 80-psi in this zone under current MDD conditions.

A future conditions analysis was conducted to evaluate the Lower Zone under foreseeable buildout conditions at reasonable densities. The buildout condition includes all foreseeable properties, both inside and outside of the current boundary. The evaluation indicates that the existing system is generally sized appropriately and that only additional looping is required to maintain service pressures and fire flow requirements; upsizing existing mains is not needed. Identified future system loops in the Lower Zone include (see map in Appendix I):

- Future PRV in Gregg Dr. for emergency download from Upper Zone to Lower Zone for future emergency service and Colorado Center.
- 8″ loop line from Railroad Street to Brady Road to serve future Collegiate Heights.
- 8″ loop through Crossman Addition with connection to existing 6″ line on east side of HWY 24 to serve future Crossman Addition.

- 12” extension off existing 12” line in HWY 24 to serve future Colorado Center, with possible loop connection to Gregg Drive also.

7.4.2 Upper Zone Evaluation

The Upper Zone is a gravity pressure zone that is fed from the Lower Zone through the Westmoor Pump Station. The Upper Zone has an HGL of approximately 8198 feet. For planning purposes only, this HGL can generally serve elevations below 8091 feet at minimum service of 40-psi. The Upper and Lower Zones are roughly divided along Sangre de Cristo Avenue; the Upper Zone lies to the west and Lower Zone lies to the east.

The Westmoor Pump Station pumps water from the Lower Zone to the Upper Zone. It is located on the southwest corner of County Road 306 and Robert Drive. The pump station is equipped with three pumps of 60, 120 and 350 GPM capacity. The 350 GPM pump is VFD controlled, with the pump speed manually set to deliver the desired flow. The two smaller pumps are constant speed pumps. Suction pressure at the station is typically around 24-psi. Discharge pressure is typically around 62-psi.

The Upper Zone also connects to the Lower Zone through Pressure Reducing Valves (PRVs). There are three existing PRVs, one on CR 306 (a.k.a. West Main Street), one on Yale Avenue and one on Meadow Lane that connects Meadow Ridge Filing No. 4 and Meadow Ridge Filing No. 3. All PRV’s from the Upper Zone to the Lower zone are for emergency service only (set to open only for low system pressures during fire flows).

The Upper Zone is currently all residential development. This means that the fire flow requirement is currently 1,000 GPM for 2-hours. However, for future planning purposes, it is recommended that the system be planned to deliver 3,500 GPM to allow for diversification in type and size of structures.

No existing service concerns were identified in the Upper Zone. Fire flows of over 1000 GPM are available at all current service locations in the Upper Zone. No excessive line velocities were indicated. No excessive high or low pressures were identified. Service pressures generally range from 55 to 90-psi in this zone under current MDD conditions

A future conditions analysis was conducted to evaluate the Upper Zone under foreseeable buildout conditions at reasonable densities. The buildout condition includes all foreseeable properties, both inside and outside of the current district boundary. The evaluation indicates that the existing system is generally sized appropriately and that only additional looping is required to maintain service pressures and fire flow requirements; upsizing existing mains is not needed. Identified future system loops in the Upper Zone include (see map in Appendix I):

- 8” loop through the converted Ivy League subdivision (i.e. gravity off Upper Zone Tanks) with connection back to the future 10” main serving potential future development to the north and west.

- Future PRV in Gregg Dr. for emergency download from Upper Zone to Lower Zone for future emergency service and Colorado Center.

- 8” loop from Gregg Drive to Rodeo for future service to residential area south of Gregg Drive.

- 10" Upper Zone loop from CR 306, around CR337, Gregg Drive and Rodeo Rd. to a connection to the existing system at Connie Drive. This will allow service to all future areas in the Upper Zone in and around the Sunset Vista and Rodeo grounds.
- 8" loop from Rodeo to Gregg Drive to serve future Sunset Vista expansions.
- 10" extension from Sangre de Cristo to Windwalker to complete the 10" loop through the Upper Zone.
- 10" main off of the south end of Brady looped to Antero Circle.

7.4.3 Ivy League

The town has one small pressure zone known as the Ivy League subdivision. This zone is pressurized by the Ivy League pump station, which is located adjacent to the 0.27 MG tank. The Ivy League Pump Station was rebuilt in 2005 with two new 350 GPM pumps powered by Variable Frequency Drives (VFD's). The suction pressure is direct from the 0.27 MG tank, so it can vary from 5 to 32 feet, but is typically above 20 feet. Discharge pressure is maintained at 48-psi. The Ivy League currently has no known service issues other than fire flow limitations due to pump capacity.

In the future, the town would like to convert this zone to a gravity system off of the Upper Zone Tank. This would also allow the pump station to then be used to fill the Upper Zone Tank and thereby provide redundancy (and capacity if needed) to the Westmoor BPS. This appears to be a viable conversion that will require minimal yard piping and extension of the Upper Zone piping to the pump station. The gravity system should provide about 45-psi at the pump station, with slightly higher pressures in the subdivision as elevations decrease. It has been preliminarily determined that a 12" line should be extended from CR 306 with connection to the discharge side of the existing pump station.

7.5 FUTURE SERVICE AREAS

There are no known future service areas at this time. As additional service areas become a possibility, the logistics of providing service to these areas will need to be explored.

7.6 SECTION SUMMARY

1. A water model of the town water system has been developed and calibrated using fire hydrant flow tests. The calibrated model is a good tool for system planning and generally produces conservative results. Potential developers should confirm actual field conditions prior to engineering and formal plan submission.
2. The water model does not indicate any significant concerns (i.e. pressure, fire flow, pipe velocity, etc.) in the existing distribution system.
3. Future conditions analyses indicate that the existing system can be adapted for the additional demands. The required modifications will be additional system looping at the expense of the developer.
4. No upgrades to existing pipes appear necessary to maintain current service conditions.

5. The town should start building a "pipe replacement fund" to replace aging infrastructure (there are some pipes in the system that are over 50 years old).

6. The town should standardize their system with minimum 8″ pipe. This means that all 4″ and 6″ pipe should be upsized to 8″ when they are replaced, and all new developments should only use 8″ or greater.

7. Preliminary investigations indicate that it is feasible to convert the Ivy League subdivision to gravity service off the Upper Zone tank. The town will need to perform a more detailed analysis during preliminary engineering to confirm service conditions given the chosen routing and flows requirements.

8. The majority of the Future Improvements herein recommended should be funded by development.

7.7 RECOMMENDED DISTRIBUTION SYSTEM IMPROVEMENTS

7.7.1 Integrate the Ivy League Zone and the Upper Zone

With the current distribution system mapping provided by the town and elevation data taken from the USGS, RGA believes that the Ivy League Zone can be completely served by connection to the Upper Zone thereby more reliably allowing the Ivy League Subdivision to operate on gravity from the Upper Zone Tank rather than have all pressure supplied by pumps, and have more reliable fire flow. Converting the Ivy League over to the Upper Zone will provide additional storage for the Ivy League Zone and redundant storage for the Upper Zone.

To complete this conversion a 12-inch diameter line would be run from the existing upper zone line in County Road 306 down Tee Road to connect into the Ivy League system. The Ivy League booster pump station could then be converted to pump from the existing Ivy League tank to the upper zone tank. A cost estimate for the 12-inch diameter line in Tee Road is shown in Table 26.

Table 26 – Ivy League Conversion to Upper Zone

IVY LEAGUE CONVERSION TO UPPER ZONE				ENGINEER'S ESTIMATE	
ITEM	DESCRIPTION	QTY	UNIT	UNIT PRICE	SUBTOTAL
1	12-INCH DIAMETER DUCTILE IRON PIPE	600	LF	$ 135	$ 81,000
	SUBTOTAL - CONSTRUCTION COST				**$81,000**
1	MOBILIZATION/DEMOBILIZATION/SITE RESTORATION/EROSION (12%)	1	LS	$ 9,720	$ 9,720
2	CONSTRUCTION SURVEY (2.5%)	1	LS	$ 2,025	$ 2,025
3	TRAFFIC CONTROL (5%)	1	LS	$ 4,050	$ 4,050
	SUBTOTAL				**$15,795**
	SUBTOTAL CONSTRUCTION COST				**$81,000**
	ADD 20% CONTINGENCY				**$16,200**
	SUBTOTAL CONSTRUCTION COST + CONTINGENCY				**$97,200**
	ENGINEERING - DESIGN				**$13,000**
	CONSTRUCTION MANAGEMENT SERVICES (8% OF CONSTRUCTION COST)				**$6,480**
	TOTAL ESTIMATED PROJECT COST				**$116,680**

Upon completion of integrating the Ivy League Zone into the Lower Zone, the Ivy League Booster Pump station can serve the Upper Zone. Through preliminary calculations RGA has found that this pump station can serve the upper zone with no modification.

It is recommended that the town begin an annual water main replacement or new construction program to eliminate old and undersized mains and loop dead end mains and to construct waterlines in streets where none currently exist. Table 27 shows a cost estimate to replace approximately 500 linear feet of pipe.

Table 27 - Annual Pipe Replacement Cost Estimate

ANNUAL WATER MAIN REPLACEMENT PROGRAM				ENGINEER'S ESTIMATE	
ITEM	DESCRIPTION	QTY	UNIT	UNIT PRICE	SUBTOTAL
1	8-INCH DIAMETER DUCTILE IRON PIPE	500	LF	$ 110	$ 55,000
	SUBTOTAL - CONSTRUCTION COST				$55,000
1	MOBILIZATION/DEMOBILIZATION/SITE RESTORATION/EROSION (12%)	1	LS	$ 6,600	$ 6,600
2	CONSTRUCTION SURVEY (2.5%)	1	LS	$ 1,375	$ 1,375
3	TRAFFIC CONTROL (5%)	1	LS	$ 2,750	$ 2,750
	SUBTOTAL				$10,725
	SUBTOTAL CONSTRUCTION COST				$55,000
	ADD 20% CONTINGENCY				$11,000
	SUBTOTAL CONSTRUCTION COST + CONTINGENCY				$66,000
	ENGINEERING - DESIGN				$15,000
	CONSTRUCTION MANAGEMENT SERVICES (8% OF CONSTRUCTION COST)				$4,400
	TOTAL ESTIMATED PROJECT COST				$85,400

8.0 WATER STORAGE

8.1 SYSTEM STORAGE REQUIREMENTS

Required water storage volumes were determined for each zone. The required volume is comprised of two parts: Fire flow, and operational/equalization storage. Each zone should have sufficient quantities of both of these components.

Zone transfers through PRV's were not considered in this evaluation due to the inherent reliability concerns associated with routine maintenance and proper adjustment. In reality, some additional level of fire volume is available from download from the Upper Zone to the Lower Zone. However, downloading and the related system affects are not quantified in this report.

There are several engineering design and planning philosophies in sizing storage tanks. One of the more widely used is to size the tank for the maximum day demand plus the required fire flow.

8.1.1 Fire Volume

The prevailing Uniform Fire Code (UFC) in use by the town dictates fire flow requirements. However, the UFC also allows the local fire authority to adjust fire flow requirements to fit the particular situation and system. Therefore, the local fire chief should be consulted to establish the actual fire requirement for every new subdivision. In some cases the fire chief may require interior sprinkler systems to reduce the fire volume requirement. If the fire chief is unavailable, then the town Engineer shall set fire flow requirements based on his interpretation of the prevailing UFC.

Per discussion with town staff, the largest known fire flow requirement in town is presently 3,500 GPM for 3 hours. This equates to a maximum emergency storage need of 630,000 gallons. This is a high fire flow requirement associated with high-risk structures such as hotels and industrial facilities. This requirement should sufficiently cover any future developments in the town. The town should also require fire sprinkler systems on all new high-risk construction, as well as all renovations, so that fire storage requirements are minimized.

8.1.2 Maximum Day Demand

Maximum Day Demand is the total usage in the maximum month divided by the number of days in that month.

8.2 STORAGE ANALYSIS

There are two storage zones that serve the town, the Upper Zone and the Lower Zone. The Lower Zone has two tanks that contain approximately 1,770,000 gallons in aggregate. The Upper Zone has one tank with a capacity of 750,000 gallons. All tanks are in relatively good condition and are well maintained. The Upper Zone tank is shown in Figure 10.

Figure 10 - Upper Zone Tank

The Lower Zone tanks currently supply both the town and the Ivy League subdivision. These tanks serve approximately 1295 SFE (1,255 SFE in 2012 + 40 SFE for the Ivy League). In addition, the town has committed to serve an additional 89 SFE in the proposed Crossman Addition and another 380 SFE in the proposed South Main Addition. This equates to a present commitment of 1764 SFE to the Lower Zone. The projected buildout requirement in the Lower Zone is 1863 SFE. Analysis of the Lower Zone indicates that the existing 1.77 MG of storage can support well over the full buildout of the Lower Zone. This means that the Lower Zone does not need additional storage.

The Upper Zone currently serves 363 SFE. The town is also committed to serve an additional 101 SFE. This equate to a total current service commitment of 466 SFE. The projected buildout requirement for the Upper Zone is 1,780 SFE.

Analysis of the Upper Zone indicates that the existing 0.75 MG of storage can theoretically support a population of less than 300 SFE. This means that planning and design of a new tank for the upper zone should begin immediately.

8.3 PROPOSED STORAGE

As previously stated, additional storage should be added to the upper zone immediately to support existing and future growth. It is recommended that an additional 0.75 million gallon tank be added to the upper zone. A cost estimate for the construction of a 0.75 million gallon tank is shown in Table 28.

Table 28 - 0.75 M.G. Storage Tank Cost Estimate

750,000 GALLON STORAGE TANK - GLASS FUSED TO STEEL				ENGINEER'S ESTIMATE	
ITEM	DESCRIPTION	QTY	UNIT	UNIT PRICE	SUBTOTAL
1	62 FT DIAMETER X 33 FT HIGH STORAGE TANK (0.75 MG)	1	LS	$ 500,000	$ 500,000
2	CONCRETE FOUNDATION	1	LS	$ 100,000	$ 100,000
3	COMMON EXCAVATION	1	LS	$ 65,000	$ 65,000
4	MISC. YARD PIPING	1	LS	$ 25,000	$ 25,000
	SUBTOTAL				$690,000
1	MOBILIZATION/DEMOBILIZATION/SITE RESTORATION/EROSION (12%)	1	LS	$ 82,800	$ 82,800
2	CONSTRUCTION SURVEY (2.5%)	1	LS	$ 17,250	$ 17,250
	SUBTOTAL				$100,050
	SUBTOTAL CONSTRUCTION COST				$690,000
	ADD 20% CONTINGENCY				$138,000
	SUBTOTAL CONSTRUCTION COST + CONTINGENCY				$828,000
	ENGINEERING - PERMITTING AND DESIGN				$65,000
	CONSTRUCTION MANAGEMENT SERVICES (8% OF CONSTRUCTION COST)				$55,200
	TOTAL ESTIMATED PROJECT COST				$948,200

9.0 SECONDARY WATER SYSTEM

9.1 RATIONALE, FEASIBILITY AND ALTERNATIVES

Note that this section is left largely unchanged from the original 2006 report. Most municipalities supply treated water for both domestic use (potable) and lawn irrigation (non-potable) through a single distribution system. Of the total flow, studies have shown that non-potable demands typically account for roughly 50% of the summer water demand in Western states. Treating and delivering potable water for non-potable uses thus accounts for a large portion of the initial and annual costs associated with water system.

Buena Vista's water system is currently used to treat and deliver both potable and non-potable water. As previously noted, this is not uncommon in the Western states, but the financial impact of this is significant, since roughly 50% of the process sizing and delivery mechanism are for outside irrigation. As water quality regulations become increasingly more stringent, the financial impact will continue to rise.

The town could save treatment costs and production capacity by attempting to reduce irrigation with potable water through improved conservation programs, encouragement of private secondary irrigation systems, and the use of City-owned/operated non-potable irrigation systems in new development areas. A recent study of 2002 drought water conservation programs in eight of Colorado's Front Range communities (Aurora, Boulder, Denver, Fort Collins, Lafayette, Louisville, Thornton, and Westminster) showed that mandatory restrictions of lawn watering to twice weekly reduced water consumption by 30% (limiting to once weekly yielded 50% savings).

While separate potable and non-potable systems are the ideal condition, at this point it is probably not reasonable for the Town of Buena Vista to convert to a dual system because of the costs involved. However, there are a few opportunities for the town to reduce current and future system demands.

Current demands can potentially be reduced by converting the town ball fields and parks to non-potable systems. This could be achieved by drilling wells at the fields/parks that are dedicated solely to irrigation purposes. Converting the town ball fields alone could save the town 1,000,000 gallons a month. The town should also encourage conservation through increasing-block fee structures, promoting xeriscaping, and mandatory watering restrictions such as odd-even address watering schedules.

As for future demands, the town should look to implement policies that require developers of new subdivisions to fund and construct raw water system infrastructure for supply to their developments. This is particularly applicable to future annexations. While such systems often have additional pumping, storage, and piping infrastructure to maintain, the repair timeframes are not as critical, and infrastructure does not have to be designed to resist significant freeze conditions or to meet sanitary and CDPHE standards. These non-potables systems can be either dedicated to the town, contract operated by the town, or maintained by the local Home Owners Association. If the system is to be dedicated to the town, system operation and maintenance should be funded by tap and service fees that are separate from the potable system.

10.0 WATERSHED PROTECTION

10.1 BACKGROUND

The town adopted into Code the formation of a Watershed Protection District (WSPD) in 2000. The WSPD was established to protect the primary water supply source for the town, Cottonwood Creek. The WSPD is given the authority to permit any development or land use within the WSPD boundaries. Items and activities that are defined in the code as having potential water quality impacts that require a permit may include, but are not limited to, sewage disposal systems; drilling; timber harvest; excavating, grading, filling, and blasting; spraying fertilizers, herbicides, or pesticides; handling or storing toxic materials; using, storing, or transporting flammable or explosive materials; tampering with the town waterworks in any way; or any activity presenting a risk to the town's water supply.

10.2 WSPD ADMINISTRATION

To date, the WSPD permit process has been exercised primarily with single-family development within the District boundary and the use of individual septic systems. There has been significant confusion and confrontation between permit applicants, Chaffee County, and the town during the permit process, primarily due to the amount of subjectivity required for the town to exercise in reviewing permits. A more standardized, risk-based approach to WSPD management and permit review is needed to reduce conflicts; however, because the infiltration gallery is such an important and potentially sensitive element in the Buena Vista water supply system, the authority of the District to protect the supply must be maintained. As the number and type of permit applications expands, it will be increasingly important for the town to have an effective and efficient means to review permits and protect its resource.

This Water Resources Master Plan recommends that the town pursue the following changes in the administration of the WSPD:

- Modify the WSPD boundary whenever any changes are made within a 5 mile radius of the WTP intake that warrant such a change (i.e. when supply points are added or changed, such as with a new well, so that areas outside current boundary become eligible for inclusion).

- Increase public awareness of the WSPD. The town should install signs along major roads at the boundaries of the WSPD with a note regarding permit requirements. The town should run a direct-mail campaign to residents in the WSPD on an annual or semi-annual basis that reviews requirements and depicts the boundaries of the WSPD. Periodic newspaper ads should also be considered. The town should also ensure that new property owners are aware of WSPD requirement during or immediately after property transfers.

- Require regular maintenance inspections of ISDS systems with the frequency dependent on type of system (advanced systems to be inspected more frequently). Require that maintenance reports be filed with the town.

- Revise the WSPD code to more clearly define management zones based on proximity and/or contamination risk to the water supply; within each zone define specific sewage handling requirements.

- Set clear requirements on what must be submitted to apply for a permit from the town, depending on the specific management zone where a use is proposed.

Table 29 presents proposed WSPD management zone categories defined by relative risk of water supply contamination along with proposed requirements for wastewater handling in the zones. Proposed zone definitions were developed with consideration of the following sources, which establish guidance and requirements for setbacks of contaminant sources to water sources and water supply infrastructure:

- *CDPHE Guidelines on Individual Sewage Disposal Systems*
- *Colorado DWR Rules and Regulations for Water Well Construction*
- *CDPHE Design Criteria for Potable Water Systems*

Appendix J presents a map showing the WSPD boundary, key water system intake points, and surface water supplies within the WSPD boundary that are tributary to the town's intake, and the proposed Zone Category I area around the town's infiltration gallery. Management zone categories to lands within the WSPD would be assigned based on Table 29 and the surface waters identified in WSPD Boundary map (in Appendix J) along with other surface waters and/or riparian areas that the town designates as requiring protection. The town should refine the definition of the zone boundaries and requirements, as appropriate, as additional watershed data become available and an improved understanding of groundwater fate and transport, especially near the town's infiltration gallery, is developed.

Table 29 - Management Zone Categories

WSPD Zone Category	Proposed Sewage Disposal Method Required	Zone Definition
I	No construction, ISDSs, or sewer lines allowed	Within 25' (horiz.) of all surface waters or riparian areas, or within 500' upgradient or 100' in any direction of the Town's Gorrel Meadow infiltration gallery or within 100' of any municipal potable production well
II	No ISDSs; all development must be sewered	Between 25' and 100' (horiz.) of surface waters or riparian areas
III	Advanced ISDSs required	Between 100' and 500' (horiz.) of surface waters or riparian areas; or where depth to groundwater is less than 20 feet
IV	Engineered ISDSs required	All other areas within the WSPD

10.3 WATERSHED STAKEHOLDERS GROUP

There are activities and events in Cottonwood Creek's watershed that could impact the town's source water quality and quantity, which cannot be addressed by its WSPD. Furthermore, a recent Source Water Assessment by CDPHE identified potential sources of water contamination in the Cottonwood Creek watershed. Effective watershed management hinges upon collaboration and communication between concerned water users. The town should spearhead the development of a watershed group for the Cottonwood Creek watershed. Possible goals of the stakeholder group would be to collaborate on:

- developing consensus watershed priorities and water quality goals
- implementing a water quality/quantity monitoring program

- identifying and implementing voluntary best management practices to maintain or improve water quality
- identifying and monitoring potential threats to water quality (for example, logging, fires, septic systems, and road construction)
- educating local public and visitors about water resource importance
- being a central advocate for watershed protection as key issues arise

Potentially interested parties for the envisioned watershed group are:

- Property owners and water users in the watershed (ranches, campgrounds, others)
- Town of Buena Vista
- Chaffee County
- San Isabel National Forest
- Recreational or environmental groups (such as Trout Unlimited)
- Other groups, districts

10.4 WATERSHED MONITORING

10.4.1 Water Quality Monitoring

The town currently performs raw water quality monitoring at its water production facilities for parameters required by CDPHE and those needed to run the production/treatment processes. However, this Water Resources Master Plan recommends that the town expand its water quality monitoring in and along Cottonwood Creek in order to establish baseline stream quality/health and track long- and short-term changes in stream quality. The goal is to have an advanced warning of changes occurring within the watershed that may produce significant negative impacts on water quality at the town's intake over time. Monitoring could also be performed in target locations along Cottonwood Creek to spatially pinpoint sources of contamination. The following types of monitoring locations should be considered:

- Near potential contamination sources (major ISDS's, etc.)
- Near major creek confluences
- Key groundwater locations
- At the town water supply intake

Through water quality monitoring, the town could establish baseline water quality and trigger points for the various parameters that would spur additional investigations or other activities to identify and rectify problems within the watershed. Example water quality parameters and sampling frequencies that should be considered are:

- Temperature, turbidity, conductivity, pH (weekly to monthly)
- Total dissolved solids, total suspended solids, total organic carbon, alkalinity, hardness (monthly to quarterly)
- total coliforms, fecal coliforms, E. Coli (monthly to quarterly)

10.4.2 Source Water Forecasting

Currently, the only continuous monitoring of water flows and movement in the Cottonwood Creek watershed is at the USGS gaging station just above Cottonwood Creek. In order to improve the understanding of water movement within the watershed, this Water Resources Master Plan recommends that the town develop and implement a program to collect data on water quantity and movement within the watershed. These data will be useful in improving the town's understanding of the impacts of watershed activities on water quality as well as its understanding of water supply reliability. This information would support future water supply planning efforts, watershed management decision-making processes, and regular water utility operational decisions. The following monitoring related to water quantity and movement should be considered.

1. *Additional stream flow gaging* - regular monitoring of flows upstream of the existing USGS gage to better track physical water supplies available to the town and improve prediction of water shortages; one location could be the bridge just downstream of the Cottonwood Hot Springs, a historic gaging station site. Other sites would include just upstream of major creek confluences on the branch creeks.

2. *Snowpack monitoring* - winter/spring measurements of snowpack depths at several key locations within the watershed, combined with additional stream gaging would improve the town's ability to forecast water availability for the peak water use periods later in the year. This information could be used to implement water conservation or water restriction programs, as needed.

3. *Groundwater table monitoring* - seasonal measurements of groundwater elevations would provide the town a better understanding of groundwater flow directions under various conditions, and the potential impacts on the town's water sources; this would allow optimizing watershed control programs.

11.0 WATER SYSTEM RULES, REGULATIONS AND FEES

This section discusses recommendations for changes to the town's Municipal Code that impact the water system. Adequate fees including service fees and tap fees are necessary to pay for on going operation and maintenance, fund depreciation, and fund ongoing expansion and upgrades to the town's water system. Some of the discussion in this section will be directed at general policy issues for fees and revenue sources. Our recommendations at this time will provide a broad overview of rates, tap fees and regulations.

West Slope mountain based municipal water systems have developed trends and industry standards for water system accounting. Most municipal and special district water systems allocate monthly service fees to pay for ongoing water fund expenses for annual administration, operations, and maintenance. Service fees should pay for a percentage of depreciation that is maintained in a sinking fund account to pay for the capital replacement of water infrastructure as it wears out and needs replacement. Most service fees cannot support full funding of depreciation.

Service fees should also pay for capital projects that serve the existing users and that are required to meet new regulatory requirements. An example would be the upgrade to a water treatment plant because CDPHE drinking water regulations have changed and become stricter. Because all users benefit from these improvements and not just future new growth, all customers should pay for these types of improvements.

Tap fees should be accrued to pay for the investment the community has in new wholesale infrastructure. Wholesale infrastructure includes water rights, water supply, treatment, storage, transmission mains etc. Wholesale infrastructure is used by the entire town, as opposed to retail facilities, which are water distribution lines and service lines that serve local areas and individual customers. Retail facilities are usually installed by the development community and then dedicated to the town. Tap fees should not only pay for the existing wholesale infrastructure but also future wholesale infrastructure that may be necessary to serve anticipated growth. Tap fees should also be used to pay for any bonded indebtedness for past water system improvements.

11.1 WATER RATES

To determine if the town's current water rates are sufficient to cover the proposed improvements, future needs and provide a reasonable capital replacement fund, a computer spreadsheet model developed by RGA was used to identify water users, the growth in the number of these users over the next five (5) years, the historic amount of water used, and future water usage projections. This spreadsheet was used to examine current water costs and to project future water costs in order to identify and suggest several rate structures that could be implemented to recover the costs of operating the system. Finally, the model was used to determine what amount of surplus funds would be developed by each of the suggested rate structures. Printouts of the model data and results are attached to this plan.

To determine historic water usage, rather than use the entire database of the town's customers, one in ten of customer's data was selected from the database, and the 2012 water usage for these customers was tabulated on a monthly basis. The analyses were all done using this statistical base, then extrapolated to the entire user base. It is necessary to collate the water usage on a monthly basis to later determine how much revenue can be anticipated from surcharges over any established base amount and charges for each month.

In order to establish the validity of any rate structure, it is necessary to examine any water structure with consideration for the growth of the town.

From town records, costs for 2012 were obtained for the water system. Costs were separated into two (2) main categories: variable and fixed. Variable costs were considered to be repairs and maintenance, utilities, treatment, operations and engineering, while fixed costs were considered to include management, water rights, director fees, insurance, legal, audit, interest, administrative, and credit enhancement. Fixed costs are considered to be those costs that are not dependent on the number of customers in the town.

Typically, water rates are structured into a base fee with tiered attendant surcharge costs, usually a charge for each 1,000 gallons used over the base charge. The base fee is theoretically designed to cover the fixed costs, and the surcharges to cover the variable costs. That way, when water usage varies from month to month, the fixed costs are always covered, and the users who use more water than the base amount will pay more, in proportion to their usage.

Sometimes, the base fee is structured to cover fixed costs with no amount of water included in that amount, but usually the base fee allows some fixed amount of water to be used. Thus, a base fee of say, $30 per month with no water included and a surcharge of $1.50 per 1,000 gallons means that a user would be charged $30 per month, even if he did not use any water, then pay $1.50 for every 1,000 gallons used from 0 to the amount actually used. This structure is usually favored by people who are low water users, especially if, for example, they are away on vacation for extended periods of time and do not use any water in any given month.

On the other hand, a base fee of the same $30 per month, which includes using 4,000 gallons, with a surcharge of $1.50 per 1,000 gallons would mean that all users using up to 4,000 gallons per month would still pay the same base fee of $30, whether or not they used 1,000 gallons per month or 4,000.

SCENARIO I

The first scenario modeled assumed 1.2% annual growth with the town's existing rate structure. The model also included constructing the capital projects required by SFE demand (additional storage) and the projects recommended in the previous sections. This scenario is shown in Table 30.

Table 30 - Rate Study - Scenario I

	Rate Structure		
Base Fee	$29.00	for first	5,000
Tier 1	$0.00	up to	5,000
Tier 2	$0.00	up to	5,000
Tier 3	$0.00	up to	5,000
Tier 4	$2.38	for all over	5,000

Table 31 shows the yearly SFE increase at 1.2%. Note that rate increase percentages were not included in this model.

Table 31 - Rate Study – Scenario I SFE Increase

Year	Yearly SFE Increase	Rate Increase	% SFE Increase
2014	19	0%	1.20%
2015	20	0%	1.20%
2016	20	0%	1.20%
2017	20	0%	1.20%
2018	20	0%	1.20%
	Out of Town Multiplier	1.60	
	3/4" Tap Fee	$ 6,000.00	

Table 32 shows the year-to-year accumulated net revenue based on the population increase of 1.2%, and completing the capital projects required at the current rates. As shown, the current rates are not adequate to support the recommended capital improvement projects.

Table 32 - Rate Study - Scenario I Cash Accumulation

Year	Metered Revenue	Connection Revenue (Tap Fee)	Total Revenue	Expenses	Accumulated Net Revenue
2014	$ 1,000,350.86	$ 97,272.73	$ 1,097,623.58	$ 932,257.00	$165,366.58
2015	$ 1,010,067.78	$ 98,245.45	$ 1,108,313.24	$ 1,415,337.00	($141,657.18)
2016	$ 1,019,881.88	$ 99,227.91	$ 1,119,109.79	$ 942,257.00	$35,195.61
2017	$ 1,029,794.12	$ 100,220.19	$ 1,130,014.31	$ 942,257.00	$222,952.92
2018	$ 1,039,805.48	$ 101,222.39	$ 1,141,027.87	$ 942,257.00	$421,723.79

SCENARIO II

At the Board of Trustees workshop on May 13, 2014, it was ultimately decided, after many iterations of the model, that the population growth rate be established at 1% per year, that there would be three tiers of rates, and that the existing base rate and the second tier be increased by 5%. This result shows a small deficit in 2015, but allows for a five year accumulated revenue of $420,443.79. These values are shown in Table 33, Table 34 and Table 35.

Table 33 - Rate Study - Scenario II SFE Increase

	Rate Structure		
Base Fee	$30.45	for first	5,000
Tier 1	$2.97	up to	20,000
Tier 2	$0.00	up to	20,000
Tier 3	$0.00	up to	20,000
Tier 4	$4.00	for all over	20,000

Table 34 - Rate Study - Scenario II

Year	Yearly SFE Increase	Rate Increase	% SFE Increase
2014	16	0%	1.00%
2015	16	0%	1.00%
2016	17	0%	1.00%
2017	17	0%	1.00%
2018	17	0%	1.00%
	Out of Town Multiplier	1.60	
	3/4" Tap Fee	$ 6,000.00	

Table 35 - Rate Study - Scenario II Cash Accumulation

Year	Metered Revenue	Connection Revenue (Tap Fee)	Total Revenue	Expenses	Accumulated Net Revenue
2014	$ 1,000,350.86	$ 97,272.73	$ 1,097,623.58	$ 932,257.00	$165,366.58
2015	$ 1,010,067.78	$ 98,245.45	$ 1,108,313.24	$ 1,416,617.00	($142,937.18)
2016	$ 1,019,881.88	$ 99,227.91	$ 1,119,109.79	$ 942,257.00	$33,915.61
2017	$ 1,029,794.12	$ 100,220.19	$ 1,130,014.31	$ 942,257.00	$221,672.92
2018	$ 1,039,805.48	$ 101,222.39	$ 1,141,027.87	$ 942,257.00	$420,443.79

11.2 TAP FEES

Currently the town's "tap fees", or as described in Section 13 of the towns Municipal Code, are called System Investment and Development Fee (SIDF). These fees are charged when a new user taps onto the town's main and offset the increased costs caused by that new tap and demand.

Tap fees for most municipalities and districts are now based upon a detailed SFE schedule. An SFE is an acronym for Single Family Equivalent and is a unit of measurement of the water use of a typical single family home.

The current town SIDF fee is shown as follows:

- 5/8" Meter $4,000
- 3/4" Meter $6,000
- 1" Meter $10,200
- 1-1/2" Meter $20,219
- 2" Meter $32,459
- 3" Meter $64,859
- 4" Meter $121,558
- 6" Meter $253,315

The tap fees adopted by the town are consistent with small growing mountain communities that do require significant capital improvements to the water system.

11.3 WATER RIGHT DEDICATION FEES

Most municipalities require a water right dedication or cash-in-lieu of fee for new development. A water right dedication ordinance requires a new developer to dedicate to the town sufficient senior water rights to offset the consumptive use from the potable water system. If a development does not have sufficient water rights to dedicate, the town can require cash-in-lieu fees. We recommend working with the town and the town's water attorney to insure the cash in lieu requirement is sufficient to offset the costs the town occurs to obtain sufficient legal water rights to serve the new use. A similar discussion should be held to determine the water rights dedication fees for new developments outside of the town limits. Typically, the cash-in-lieu payment for water rights is based upon the towns ability to go out on the free market and obtain adequate water rights, either through a purchase of direct flow irrigation rights, storage rights,. In addition to the purchase cost, changing these water rights to municipal use and changing the location of diversion to the town diversion facilities will require preparation of augmentation plans and water court proceedings. These expenses should also be included in the cash-in-lieu of requirement.

Section 13-62(5)c. of the Buena Vista town Code outlines the town's authority to establish, at the town's option, a fee in-lieu of water rights dedication. The town's water rights engineer, Wright Water Engineers, performed a cash-in-lieu fee analysis to help establish the appropriate fee in-lieu of water rights. This analysis was used by the town as the basis for establishing their cash-in-lieu fee for extraterritorial service extensions. Table 36 outlines the current fee structure as adopted in Resolution No. 18 (Series 2006).

Table 36 - Fee for Cash In-Lieu of Water Rights

1	2	3	4
Classification	**SFE**	**Acre Feet per Year**	**Cash in Lieu Fee (col. (3) x Unit Cost of Water Right)**
Single Family Unit *In-House Water Use Only*	1	0.3	$6,000
Duplex or Attached *In-House Water Use Only*	0.8	0.24	$4,800
Multi-Family *In-House Water Use Only*	0.6	0.18	$3,600
Irrigation Water *Irrigation Water per 1000 square feet irrigated area*	-	0.06	$1,200

With increasing competition for the available water rights in the Arkansas River Basin the costs of purchasing water rights for municipal uses will increase. These increases in cost can be substantial. The price of Colorado-Big Thompson (CBT) units provides an example of the scale of these cost increases. Many Front Range communities require the purchase of CBT units. The cost of CBT units increased as much as 20% per year over the last ten years. Section 13-62(5)c. of the Buena Vista town Code outlines the town's authority to set the amount of the in-lieu fee reasonably necessary to purchase water rights of sufficient quantity and seniority to provide ample water to satisfy the demands of the development or property to be served. Because of the volatility in water rights costs, we strongly recommend that the town regularly monitor local and regional water rights cost and adjust the In-Lieu Fees accordingly. It is also recommended that the town develop and approve, through ordinance, a commercial cash-in-lieu water right dedication fee.

11.4 MISCELLANEOUS FEES

Another user fee the town should consider adopting and charging is standby fees. Standby fees are associated with second homes or homes that are not occupied when water is not used for longer periods. Standby fees can also be used for structures that are under construction. Usually both fees are nominal and are1/3 to 1/2 of the minimum customer charge.

Many municipalities and districts also impose fees for routine events that water department employees are required to perform, such as inspection fees for water connections, turning on and off water, meter inspections, line locates, etc. These fees should represent the actual cost to the department to perform this work.

11.5 MISCELLANEOUS RULES AND REGULATIONS

This section comments on a few of the town's water rules and regulations found in Chapter 13 Municipal Utilities section. Overall, this section seemed to be very thorough and include provisions similar to other small mountain communities. A few areas, however, should be considered:

- Complete a watershed protection plan developed in conjunction with CDPHE and CRWA.
- A definitive policy on the requirement of looping water distribution lines in new developments
- A definitive policy on the requirement to extend mains to the far property lines for the next adjoining development which will occur.
- Technical discussion on the need for conductivity straps
- The use of class 52 Ductile Iron Pipe
- More definitive bedding specifications
- Requirement for bacteria sample testing for new construction

APPENDIX A
ISO Fire Flow Data

APPENDIX B
Water Demand Calculations

APPENDIX C
Yearly Production Data

APPENDIX D
Storage
Calculations

APPENDIX E
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APPENDIX F
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APPENDIX G
Modeling Results

Current Conditions – 2006 CAL

Buildout Conditions – 2006 CAL Future

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APPENDIX H
Current Water System Map

APPENDIX I
Future Water System Map

APPENDIX J
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APPENDIX K
Rate Study Model

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