FINAL
WATER MASTER PLAN

FOR

TOWN OF BAYFIELD

May 29, 2014

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ENGINEERING

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Table of Acronyms

ADD	Average Daily Demand
AF	Acre-Feet
ANSI	American National Standards Institute
CIP	Capital Improvement Program
CPDWR	Colorado Primary Drinking Water Regulations
CDPHE	Colorado Department of Public Health & Environment
CT	Contact Time
cfs	Cubic Feet Per Second
DBPP	Disinfectant Byproduct Precursors
D/DBP	Disinfectants / Disinfectant By-Products
EU	Equivalent Unit
ERT	Equivalent Residential Tap
GAC	Granulated Activated Carbon
GMF	Granular Media Filtration
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
gpm	Gallons per Minute
HAA	Haloacetic Acids
HRT	Hydraulic Retention Time
IGA	Intergovernmental Agreement
LAPLAWD	La Plata Archuleta Water District
LSI	Langelier Stability Index
LCR	Lead and Copper Rule
LT1ESWTR	Long-Term 1 Enhanced Surface Water Treatment Rule
LT2ESWTR	Long-Term 2 Enhanced Surface Water Treatment Rule
MCC	Motor Control Center
MCLG	Maximum Contaminant Level Goal
MCL	Maximum Contaminant Levels
MDD	Maximum Daily Demand
MPA	Microscopic Particulate Analysis
MRDL	Maximum Residual Disinfectant Levels
MG	Million Gallons
MGD	Million Gallons Per Day
MSDS	Material Safety Data Sheet
ND	Not Detected
NR	Not Requested

NSF	National Sanitation Foundation
NTU	Nephelometric Turbidity Unit
O&M	Operations and Maintenance
PHD	Peak Hourly Demand
PLC	Programmable Logic Controller
PWSID	Public Water System Identification
SCADA	Supervisory Control And Data Acquisition
TDH	Total Dynamic Head
THMs	Trihalomethanes
TTHM	Total Trihalomethane
TOC	Total Organic Carbon
UV	Ultraviolet
VFD	Variable Frequency Drive
WQCD	Water Quality Control Division
WTP	Water Treatment Plant

1.0 EXECUTIVE SUMMARY

This Master Plan has been prepared for the Town of Bayfield (Town), and is intended to assist in the assessment of existing water system supply, distribution and treatment components; development of a water model that can be used to assess both the existing system and a projected future build-out system; identification of water system upgrades required to meet the needs of future growth; and development of proposed system improvements for inclusion in a capital improvements plan. The report documents the available raw water supplies, identifies the needs at the treatment facilities, and the distribution system within the specified timeframe.

Ultimately, the usefulness of a Master Plan is tied to the reliability of planning decisions made using information contained in the Master Plan. It is strongly recommended that this Master Plan be updated every five (5) years and appropriate sections revised annually since the quality of the data and information within the Master Plan becomes progressively obsolete once the Master Plan has been published .

The various subsystems and components of the Bayfield water system are appropriately sized and in general will continue to provide for community needs over the planning period ending in 2035 (approximate 20-year period). Several items that are recommended for capacity-related upgrades or replacement to be addressed during the planning period are summarized below.

The most pressing area that presents a potential future capacity limitation is available municipal water rights. User demand is projected to overcome the currently available municipal rights within the next five years. Converting the remaining 1.869 cfs irrigation right on the Los Pinos Ditch is believed to be the best first step to increase municipal raw water supply rights.

The next set of potential future capacity-related items are not anticipated to become significant until somewhere near 2025; although the staging/prioritization of these items could shift significantly as additional information becomes available in future Master Plan updates. Between the years 2025 and 2028, treatment capacity, finished water pump capacity, and certain distribution pipe sections will all likely need to be re-assessed.

It is recommended to phase the upsizing of the sections of distribution piping identified below. This allows separate sections to be addressed as funding becomes available. It would likely be beneficial to replace the 6-inch asbestos cement (AC) piping along Mountain View (and crossing Hwy 160) before replacing the 10-inch sections near the WTP and storage tanks. The AC pipe section represents some of the oldest distribution piping still in service for the Town, and is the major transmission line to areas south of the highway. The rationale associated with the 10-inch sections identified for replacement is due to the fact that the model results indicate these sections will eventually become velocity limited and induce unwarranted head loss in the system.

The projected need for additional future treatment capacity and finished pump capacity near the later years of the approximate 20 year planning period would be best addressed simultaneously, since the treatment plant/finished pump capacity upgrade would be a capital expense that cannot be phased or funded over a distributed time frame.

2.0 INTRODUCTION AND OBJECTIVES

This Master Plan has been prepared for the Town and is intended to assist in the assessment of existing water system supply, distribution and treatment components; development of a water model that can be used to assess both the existing system and a projected future build-out system; identification of water system upgrades required to meet the needs of future growth; and development of proposed system improvements for inclusion in a capital improvements plan. In addition, this report provides reference information for existing water system components and their expected service life. Major water treatment and conveyance components were evaluated for current and future functionality. Recommendations regarding planned replacement or upgrades are presented in Section 8.0.

2.1 APPROACH

The methodology used to develop required information for this Master Plan included the following:

1) Review of the Town's historic and projected population data in order to estimate water usage projections for the Town's service area.
2) Development of a hydraulic model to simulate the existing conditions.
3) Development of a hydraulic model to simulate the system using the projected population in 2035, and identify improvements required to maintain adequate service pressures consistent with future growth projections.
4) Assessment of future water storage requirements for the distribution system.
5) Identification of proposed system improvements for use in a capital improvements plan.

Section 3.0 presents a summary of existing water supply and treatment facilities, and regulated treatment requirements. Section 4.0 discusses estimated future water supply requirements based on recorded population and water usage rates, and estimated future population growth and water use. Section 5.0 of this report evaluates the current and estimated future raw water supply requirements. Section 6.0 presents a discussion of the water distribution model. In Section 7.0 the various system components are evaluated, including raw and finished pumping systems, distribution system storage tanks, pumps, piping, and the treatment facility. System components recommended for inclusion in the capital improvements plan are presented in Section 8.0.

3.0 DESCRIPTION OF EXISTING WATER SYSTEM

3.1 SERVICE AREA

The Town's current water system service area expands beyond the corporated limits of the Town of Bayfield; however, for purposes of this Plan, the Town limited the scope to the existing water system which is primarily within the corporate limits of the Town. The majority of future growth is predicted to be north of Highway 160 and east of County Road 501. Additional concentrated growth is projected to occur along the southeast border of the existing service area. Appendix A presents maps depicting the Current and Future Service Area, Future Land Use, and allocation of Sanitary Sewer Equivalent Residential Taps (ERT's) within the current and predicted service areas. The ERT Maps have been prepared by Souder Miller & Associates (SMA) to provide reference of current and future user distributions for both the water and wastewater systems.

3.2 WATER SUPPLY FACILITIES

Described below are the Town of Bayfield's existing raw water infrastructure, including raw water intakes, conveyance to the Bayfield Water Treatment Plant, and water storage reservoir.

The Town of Bayfield has two options for conveying raw water from the Pine River to the existing raw water reservoir: (1) gravity flow through the Los Pinos Ditch, and (2) the Los Pinos Raw Water Pump Station.

3.2.1 Los Pinos Ditch

Raw water from the Pine River flows through the Los Pinos Ditch to the raw water reservoir adjacent to the existing Bayfield WTP. The Los Pinos Ditch intersects the Pine River roughly 1.5 miles north-northeast of the WTP. Flow diversion from the Ditch to the reservoir is controlled by a manually operated gate. A 9-inch Parshall flume provides for measurement of the flow diverted from the ditch. An ultrasonic level sensor is used to measure the flow rate, which is recorded at the WTP via the Supervisory Control and Data Acquisition system (SCADA). This diversion will be upgraded as part of a water treatment plant improvements project that is part of an Intergovernmental Agreement (IGA) between the Town and La Plata Archuleta Water District (LAPLAWD), scheduled to begin construction in 2014.

3.2.2 Los Pinos Pump Station

The existing Bayfield WTP can also be supplied with raw water from the Pine River via the Los Pinos Pump Station that draws water from the Pine River downriver of the Los Pinos Ditch diversion point. The Los Pinos Pump Station draws water directly from the Pine River via an infiltration pipe buried under the river bed. The Pump Station consists of two (2) vertical-turbine centrifugal pumps, which convey water to the raw water reservoir. At time of installation the pumps were rated to produce 600 gpm at the estimated Total Dynamic Head (TDH) of 125 feet. The Los Pinos Pump Station is typically in service only 2-3 weeks per year, but its use is driven by need for additional water supply. Two (2) new vertical turbine pumps are included in the water treatment plant improvements project referenced in Section 7.7.1.

3.2.3 Raw Water Reservoir

Adjacent to the existing Bayfield WTP is a raw water reservoir with a capacity of approximately 30 acre-feet (9.8 MG) and a surface area of roughly 2.1 acres. The average depth of the raw water reservoir is roughly 15 feet, with a maximum depth of approximately 20 feet. The reservoir was constructed in 1977.

The raw water reservoir provides short-term storage of raw water prior to treatment, and also allows some removal of particulate matter via sedimentation.

3.3 TREATMENT PLANT

The existing WTP receives untreated (raw) water via gravity flow from the raw water reservoir, and operates three 0.5 MGD package treatment units which include one Trident and two ActiFloc units.

The portion of the WTP that houses the Trident unit was originally constructed in 1977, and upgraded in 1995. It includes coagulant and polymer feed systems, clearwell, backwash and finished water vertical turbine pumps, motor control center (MCC), the laboratory, and chlorine storage/feed room. The ActiFloc expansion was constructed in 2003, and included a pre-fabricated metal building, clearwell, blowers, separate coagulant and polymer feed system, and master control panel.

Operational control of the treatment plant is automated based on level in the clear well. When the low level set point is reached, flow through the treatment systems is enabled; when the high level set point is reached the treatment system is stopped. When the facility is running, any filtration units set in 'Auto' mode will actuate and process water at their respective flow set points. Chemical pumps and the disinfection systems are also centrally controlled by the Supervisory Control and Data Acquisition system (SCADA) and will start and stop as needed by the respective treatment units.

Filter backwash cycles are controlled based on head loss. If a filter exceeds its head loss set point, a backwash will be initiated automatically. Backwash waste flows to a pond outside the treatment facility before being pumped back to the raw water reservoir. The backwash ponds will be replaced with a backwash recovery system including a high-rate plate settling system, track-mounted sludge collection, and pumping station as part of the WTP improvements project described in Section 3.3.4.

3.3.1 Chemical Systems, Pre-Filtration

3.3.1.1 Coagulant Dosing

The System currently doses an Aluminum Chlorohydrate solution to achieve particle destabilization and coagulation. EC-309, supplied by Southern Water Consultants Inc., is used in all three (3) treatment units. [See Material Safety Data Sheets (MSDS) and National Sanitation Foundation (NSF) certification data in Appendix B.]

3.3.1.2 Polymer Dosing

Two different polymers are currently in use; each is purchased dry and hydrated onsite in batch tanks. Each polymer is a formulation of polyacrylamide and thus limited to dosing rates of less than 1 mg/L per NSF requirements. The ActiFloc units use Magnafloc LT22S, while the Trident unit is dosed with Magnafloc LT20. (See MSDS and NSF certification data in Appendix B.) The LT20 product is a nonionic medium molecular weight polymer; the LT22S product is a cationic high molecular weight polymer.

3.3.2 Filtration

3.3.2.1 Trident Unit

The existing Trident unit is rated for flow rates up to 350 gpm (0.5 MGD). The Trident units were originally designed and marketed by U.S. Filter; the technology is currently owned by WesTech. The Trident design utilizes an up-flow clarifier with buoyant media followed by a multimedia filter bed and is classified by Colorado Department of Public Health and Environment (CDPHE) as a direct filtration treatment process.

3.3.2.2 ActiFloc Units

The two ActiFloc units are each rated for flows up to 350 gpm (0.5 MGD). Kruger's ActiFloc design incorporates a sand-ballasted flocculation system, a cyclone sand/sediment separation process, high rate settling, and a multimedia filter bed. The ActiFloc process is classified as conventional treatment by CDPHE.

3.3.2.3 Microscopic Particulate Analysis

Microscopic particulate analysis (MPA) testing of the existing Bayfield WTP's raw and treated water indicate that existing packaged water treatment units achieve particulate and microorganism consistently above 2.0 log removal. See MPA's in Appendix.

3.3.3 Disinfection Systems

3.3.3.1 Chlorine Dosing

Chlorine is the primary disinfectant for the System. Chlorine gas is fed from 150 pound cylinders which are stored in a dedicated room. The existing chlorine room has a separate entrance from the remainder of the facility; it will be relocated and upgraded during the proposed 2014 expansion. The existing Capital Controls NXT Series gas dosing system was installed in 2010. The system is set up for flow pacing within the SCADA program.

3.3.3.2 Ultraviolet (UV) Disinfection System

A UV system is part of the proposed 2014 expansion; it will provide contact disinfection while chlorine will continue to provide the residual disinfection.

3.3.3.3 Clear Wells

The existing Bayfield WTP has two clearwells to provide contact time for chlorine disinfection prior to distribution. The primary clear well was originally constructed to serve the single Trident unit; it also holds the Trident backwash pump and finished water pumps used to convey flow into the distribution system. The second clear well was constructed to receive flow from the two ActiFloc units; it also uses a single backwash pump for the two filters. The two clearwells are connected hydraulically and water entering the second clearwell flows to the primary clearwell via gravity.

Baffling in the existing clearwells ranges from poor to average, resulting in baffling factors of 0.3 to 0.5. The contact time (CT) and Log Removal values achieved in the existing clearwells (with an assumed pH of 7.0) are shown below in Table 1.

TABLE 1 – CT VALUES IN EXISTING CLEARWELLS

Trident Clearwell			
Residual Chlorine Concentration	CT Achieved (mg*min/L)	Log *Giardia* Inactivation	Log Viruses Inactivation
0.6	6.5	0.1	2.2
1.0	10.9	0.1	3.7
2.0	21.7	0.3	7.5
ActiFloc Clearwell			

Residual Chlorine Concentration	CT Achieved (mg*min/L)	Log *Giardia* Inactivation	Log Viruses Inactivation
0.6	2.0	0.03	0.6
1.0	3.3	0.05	1.0
2.0	6.6	0.08	2.2

The WTP improvements project that will be constructed beginning in 2014 will contain a new clearwell sized to provide in more than 3-Log Giardia and 4-Log Virus inactivation credit for all three existing treatment units in addition to the new ActiFlo Filter treatment unit.

3.3.4 Existing Backwash Pond

The existing Bayfield WTP has a single backwash pond immediately to the north of the existing water treatment plant building. The backwash pond receives backwash water from the existing treatment processes and provides flow equalization of the waste process water.

Water within the existing backwash pond is pumped back to the raw water reservoir. It is assumed that some backwash water is lost to evaporation. If the backwash pond becomes overfilled, there is an emergency overflow to the Schroeder ditch, which is adjacent to the WTP. Periodically, when backwash sludge in the backwash pond has accumulated to maximum storage capacity, the Town of Bayfield physically removes, dewaters on-site, and beneficially reuses backwash solids as a soil amendment in landscaping applications. If no beneficial reuse opportunities exist, the Town retains the solids on site.

As part of the WTP Improvements Project, the existing backwash ponds will be replaced with a backwash recovery system consisting of a high-rate settling system, automated sludge collection system, and pumping station to convey settled backwash recovery water to the reservoir.

3.3.5 Supervisory Control and Data Acquisition (SCADA) System

The Town installed a SCADA (Supervisory Control and Data Acquisition) system in 2011. The SCADA system utilizes an Allen Bradley CompactLogix PLC, and the associated RSView software package. The existing SCADA system allows operations staff to remotely monitor plant performance for both the water and wastewater treatment facilities.

The SCADA system includes the following functionality:

- Electronic hand-off-auto operation for all actuated valves and pumps.
- User-adjustable settings for process control functions, including backwash, filter-to-waste, and flush, as appropriate for each treatment unit.
- User-adjustable settings for alarm limits.
- Turbidity, headloss, and timer control for automatic backwash initiation, as well as a manual backwash sequence initiation button.
- Recording individual filter turbidity data.
- Ability to expand in future by 5-10 extra input/output (I/O) points.

The SCADA system has an alarm callout process that is user-adjustable; it includes a callout list and adjustable limits for alarm parameters. Callout alarms include equipment failure alarms, finished turbidity, finished chlorine residual, low storage tank level, and loss of raw water flow.

3.3.6 Treated Water Quality Requirements

A number of rules have been developed by the U. S. Environmental Protection Agency (EPA), pursuant to the federal Safe Drinking Water Act (SDWA). The following section discusses the applicable treatment regulations identified in the following list:

- Long Term 1 Enhanced Surface Water Treatment Rule (LT1ESWTR)
- Long-Term 2 Enhanced Surface Water Treatment Rule (LT2ESWTR)
- Stage 1 and Stage 2 Disinfectant and Disinfection By-Products Rules
- Filter Backwash Recycling Rule
- Lead and Copper Rule
- Total Coliform Rule

These rules are discussed below.

Long Term 1 Enhanced Surface Water Treatment Rule

The Long Term 1 Enhanced Surface Water Treatment Rule (LT1) requires systems using surface water or Groundwater Under Direct Influence (GWUDI) to comply with strengthened filtration requirements. The following LT1 conditions apply:

- Systems serving fewer than 10,000 persons must achieve a 2-log removal of Cryptosporidium.
- Systems using alternative filtration systems (other than conventional, direct, slow sand or diatomaceous earth systems) must meet specific State-established combined effluent turbidity requirements.
- Systems must develop a disinfection profile unless observed DBP concentrations are less than 0.064 mg/L for TTHM and 0.048 mg/L for HAA.
- Systems significantly changing disinfection practices must develop a disinfection inactivation benchmark for the existing practice and consult with the State before implementing changes.
- Finished water reservoirs must be covered.
- Unfiltered systems must comply with updated watershed control requirements including addition of Cryptosporidium as a pathogen of concern.

Long Term 2 Enhanced Surface Water Treatment Rule

The Long Term 2 Enhanced Surface Water Treatment Rule (LT2) further addresses the acute risk of exposure to microbial pathogens found to naturally occur in the water supply. The LT2 rule supplements the requirements of the SWTR and the LT1 rule to strengthen the protection of drinking water from pathogens. Among its applicable requirements for systems serving fewer than 10,000 persons are:

- Systems must conduct initial source water monitoring for E. coli or Cryptosporidium.
- Subsequent Cryptosporidium monitoring is required if E. coli levels exceed 10/100 milliliters (mL) for lake and reservoir sources, or 50/100 mL for flowing stream sources.
- Based on monitoring data, water sources will be classified into specific bins identifying whether additional treatment and control methods are required to remove and inactivate Cryptosporidium, and the level of additional log removal and inactivation is required.

- Systems required to implement additional treatment and/or control techniques to meet additional log removal/inactivation requirements may utilize a "microbial toolbox" consisting of select treatment and control options.

During 2008 and 2009 the Town completed testing required for classification within the LT 2 Rule. This resulted in a Bin 1 classification, meaning the system has no additional removal/inactivation requirements in the treatment process and no additional monitoring requirements for Cryptosporidium.

Stage 1 and Stage 2 Disinfectant and Disinfection By-Products Rules

The Stage 1 and Stage 2 Disinfectant and Disinfection By-Products (DBP) Rules provide protection of public health through the regulation of disinfection by-products (DBP). Long-term risks (associated with chronic exposure) are managed by providing treatment to reduce the potential for forming DBPs.

Stage 1 was promulgated in December 1998 and supersedes the 1979 regulations regarding total trihalomethanes (TTHM). Maximum contaminant levels (MCL) for DBP concentrations were established for specific disinfection by-products including individual trihalomethanes (THM), haloacetic acids (HAA), chlorite and bromate. In addition, maximum residual disinfectant levels (MRDL) were established for chlorine, chlorine dioxide, and chloramines.

Stage 2 was promulgated in January 2006 and builds upon the requirements of the Stage 1 rule. The main requirements of the Stage 2 rule are increased monitoring of THM and HAA at specific locations within the distribution system. Compliance with the Stage 2 rule requires completion of a distribution system evaluation and establishment of standard sampling sites at representative locations throughout the system.

The Town completed the Initial Distribution System Evaluation (IDSE) during the same time frame as the LT2 testing, and the sampling sites for computing the required locational running annual averages (LRAA) have been selected. For systems serving fewer than 50,000 persons, both the LT2 and Stage 2 Rule monitoring requirements go into effect in October 2013.

Filter Backwash Recycling Rule

The Filter Backwash Recycling Rule (FBRR) requires monitoring and reporting of recycle practices to reduce the opportunity for affecting treatment process performance and to help prevent pathogens from passing through treatment processes and into the finished drinking water in concentrations that may increase the acute health risks to customers. The FBRR applies to all facilities that recycle water from filter backwash, thickener supernatant, or dewatering processes.

Systems recycling these fluid streams are required to retain the following information:

- List of recycle flows and the frequency they are returned.
- Average and maximum filter backwash flow rates, the average and maximum backwash duration.
- Typical filter run length and summary of how length is determined.
- Type of treatment provided for recycle flow.
- Data on equalization and/or treatment units; typical and maximum hydraulic loading rates; type of treatment chemicals used and average dose and frequency of use; and frequency at which solids are removed.

Lead and Copper Rule

The Lead and Copper Rule requires water suppliers to control corrosion in the distribution system and monitor lead and copper concentrations at customer taps. Treatment techniques are typically used to maintain finished water pH above levels that are considered to be aggressive, i.e. corrosive. Such techniques may include the addition of a basic chemical to finished water to encourage the formation of mineral scales that inhibit the leaching of lead and copper into the distribution system.

Total Coliform Rule

The Total Coliform Rule established a maximum contaminant level goal (MCLG) of zero for total coliform, which are representative of the bacteria population within the distribution system. Total coliform is used as a gross surrogate for evaluating overall treatment effectiveness. Requirements include regular monitoring of the distribution system.

3.4 DISTRIBUTION SYSTEM

The Town of Bayfield currently has approximately 15.5 miles of water distribution lines that supply drinking water to the Town's residents. The distribution system is organized into five geographic pressure zones. Most pressure zones are separated by pressure reducing valves that regulate appropriate downstream discharge pressure.

Operational control of the various facilities is based on tank levels in the distribution system. If the Highlands concrete tank reaches a low level set point, the booster pump adjacent to the Tamarak tanks will start and convey water until the Highlands concrete tank reaches its high level set point. If the Tamarack tanks reach their low level set point, the clear well pumps at the treatment facility will start and convey water until either the clear well reaches a low level set point or the steel tanks are filled to their high level set point.

3.4.1 Pipelines

The distribution piping consists of a range of pipe materials and sizes. Materials currently in use include asbestos reinforced concrete (AC), polyvinyl chloride (PVC), and steel; pipe sizes range from four (4) inch in some older sections, and up to ten (10) inch for some newer transmission lines. Six (6) and eight (8) inch are the predominant pipe diameters throughout the distribution system.

3.4.2 Storage Facilities

The Town of Bayfield currently has four (4) drinking water storage tanks providing a total capacity of 1.75 million gallons (MG), as shown in Table 2 below. All four tanks are located in the northeast portion of the Town.

Three of the tanks are located along Tamarack Drive and float at the same water elevation. The first 0.25 MG tank was built on this site in 1977; a second 0.25 MG welded steel tank was constructed in 1989. The interior and exterior surfaces of each of the welded steel tanks were recoated in 2011. The third tank is a 1 MG bolted steel tank with a glass liner that was constructed in 2007. The fourth storage tank is a partially buried 0.25 MG concrete tank that is located near the end of Dove Ranch Road, on the side of Highlands Hill. (See the Pressure Zone Map in Appendix C which identifies the two tank location sites.)

TABLE 2 – FINISHED WATER STORAGE CAPACITY

Finished Water Storage Tank	Total Capacity (MG)
Tamarack East Welded Steel Tank (constructed 1989)	0.25
Tamarack West Welded Steel Tank (constructed 1977)	0.25
Tamarack Bolted Steel Tank (constructed 2007)	1.0
Highlands Concrete Tank (constructed 2010)	0.25
Total Storage Capacity	1.75

3.4.3 Pump Station

There is a single pump station in the distribution system. It is adjacent to the Tamarack tank site and conveys finished water to the Highlands concrete tank as needed. The pump station was placed into service in 2007 and includes a single Berkeley centrifugal pump that was designed for a duty point of 400 gpm at a TDH of 260 feet.

3.4.4 Interconnections with Other Systems

The La Plata Archuleta Water District is the sole consecutive system. An Intergovernmental Agreement (IGA) has been signed between the Town and the LAPLAWD. This IGA includes the expansion of the existing water treatment facilities, interconnection between the two distribution systems, and sharing of distribution system facilities.

Under the IGA certain components are designated as Joint-Use-Facilities, to be shared by both LAPLAWD and the Town. These include the existing raw water pump station, existing raw water intake from the Los Pinos Ditch, existing raw storage reservoir, treatment facility, and areas of the distribution system (refer to the IGA for more detail – See Appendix). The impacts of this IGA on the existing system will be evaluated further in Sections 6.0 & 7.0 below.

4.0 WATER SUPPLY REQUIREMENTS

4.1 POPULATION PROJECTIONS AND GROWTH AREAS

4.1.1 20 Year Planning Period

The twenty (20) year planning horizon is from 2013 to 2032. Population and water usage estimates presented in this study cover the planning period and continue through 2035.

The historical population data presented in Section 3.0 is derived from U.S Census and Colorado State Demographers databases. The future population growth estimate is from a study completed by Souder Miller & Associates (SMA).

4.1.2 Comparison of Recorded versus Predicted Growth Rates

The Town of Bayfield was incorporated in 1910 with a population of 227 people. The 1970 census showed that the Town had grown by less than 100 people to a population of 320. By 1980 the population more than doubled to 724 persons. Since then the population growth has been consistent at approximately 3.2 percent per year. The last federal census was completed in 2010. Figure 1 (below) presents the recorded and estimated Town population between 1980 and 2035.

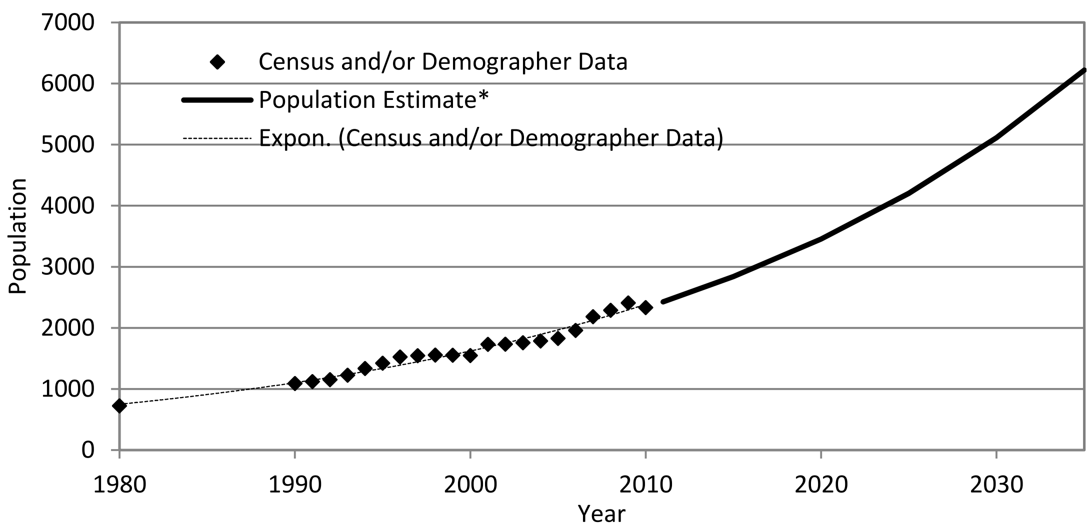


FIGURE 1 – TOWN OF BAYFIELD HISTORICAL AND PREDICTED FUTURE POPULATION
**Population estimate provided by Souder Miller & Associates.*

4.1.3 Specific Areas of Concentrated Growth

In recent years, growth has been largely concentrated on the North side of Highway 160 with some growth to the East of downtown Bayfield. The majority of future growth is projected to be North of Highway 160 and East of CR 501. The most concentrated areas of medium growth are projected to be immediately north of Dove Ranch Road, eventually migrating east of the existing end of Mesquite Street. Additional medium density growth is also anticipated to the Southeast of the current service area, adjacent to the existing Mesa Meadows and Clover Meadows subdivisions. For more detail, see the ERT Map (compiled by SMA) located in Appendix A.

4.1.4 Land Use

The predominant zoning type in the Town is residential. Less than 10 percent of incorporated areas are designated as commercial. Future growth is projected to continue the existing pattern of residential zoning with sufficient commercial development to provide for the major needs of local residents. For visual reference see the Bayfield Future Land Use Map (developed by RPI Consulting), located in Appendix A.

4.2 HISTORICAL WATER USE

4.2.1 Annual Water Production

The average water demand varies seasonally. During winter months when there is no irrigation, water demand has averaged approximately 0.28 MGD for the past 5 years. Summer flows show a significant increase in water demand, which peaks in June and July with an average demand of 0.6 MGD. Average daily demand (ADD) is 160 gpcd. Figure 2 shows the averaged ADD per month for the period January 2007 through July 2012.

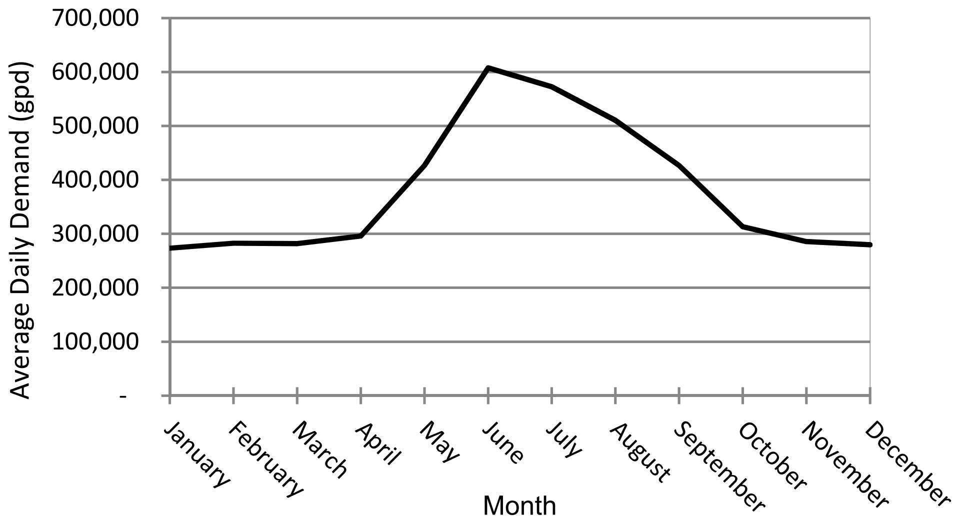


FIGURE 2 – AVERAGE DAILY WATER DEMAND PER MONTH (JAN 2007 – JULY 2012)

During the same time period discussed above the Maximum Day Demand (MDD) has ranged from 0.82 MGD to 0.94 MGD; the average MDD for this six year period has been 0.88 MGD. In relation to population levels, the MDD demand has shown a decreasing trend during this period. In 2007 the MDD represented a demand of 402 gpcd, while in 2012 the MDD usage was 353 gpcd. This decrease is likely associated with the implementation of water restrictions. The MDD data is presented below in Figure 3.

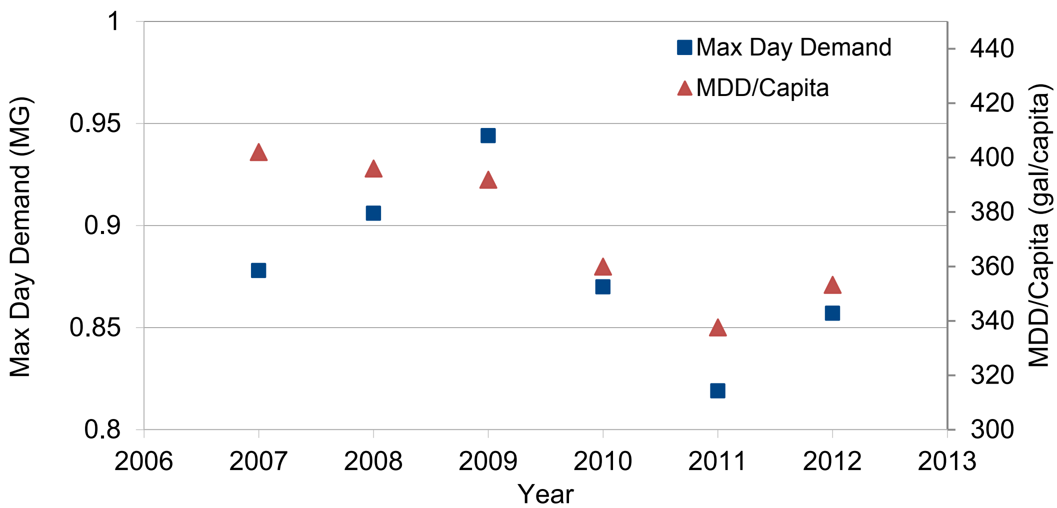


FIGURE 3 – ANNUAL MAXIMUM DAY DEMAND

4.2.2 Historical Connections

Between January 2011 and August 2012 the number of billed accounts fluctuated between 838 and 914 accounts. During this period the average number of billed accounts in relation to population has been 2.77, which compares reasonably well to a national average of 2.5 (population estimates are presented in Figure 1, in Section 3.1.2). Table 3 presents the calculated present and future population and projected number of billed accounts.

TABLE 3 – POPULATION AND BILLED ACCOUNTS PREDICTION

Description	Existing Service Area (2012)	Future Service Area (2035)
Population	2,529	6,219
Billed Accounts	913	2,245

4.2.3 Current Population and Per Capita Water Usage

The most recent federal census was conducted in 2010, and reported a population of 2,333 persons living in the Town of Bayfield. Based on an extended population estimate prepared by SMA, the population was projected to be 2,529 by end of 2012.

Per capita water usage has been fairly consistent over the past five years. The overall annual average usage is 160 gpcd. The annual usage pattern is strongly influenced by irrigation; the summer months (May through September) show a typical average 1.8 times higher than during winter months (October through April). The winter average is 121 gpcd, while the summer months show an average of 216 gpcd. Average daily demand per capita is shown below in Figure 4.

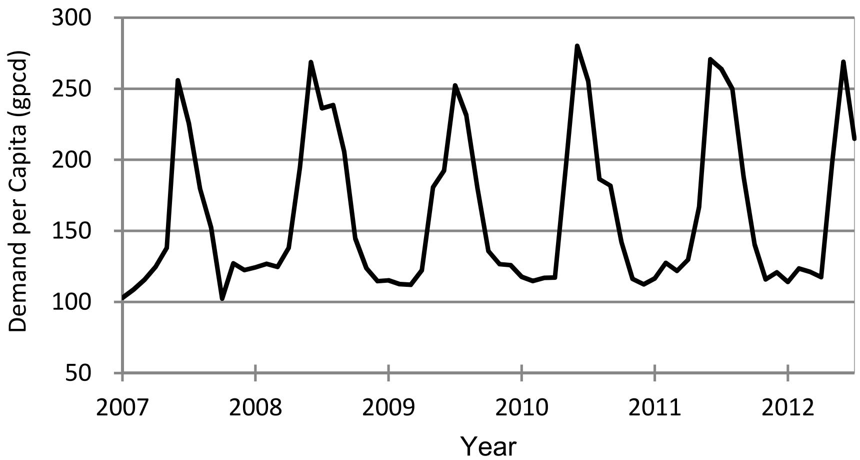


FIGURE 4 – AVERAGE DAILY DEMAND PER CAPITA

4.2.4 Water Losses

Distribution systems typically have some water loss; in general, losses of approximately 15 percent or less are considered acceptable. Comparing production and billing records allows calculation of the water loss in a distribution system (assuming there are no unmetered users). The Bayfield distribution system has exhibited an apparent average loss of 37 percent between January 2011 and July 2012. This level of loss warrants some investigation; it could be largely a paper error, although the most likely answer is some combination of real and paper losses. Town staff is aware of this issue and they are working to address known leaks and correct any possible paper errors.

4.3 WATER DEMAND PEAKING FACTORS

Peaking factors provide a means to estimate future usage demands based on projected population growth. Average demand per capita is projected to be fairly consistent with time, allowing estimation of future water use as a linear relation to population growth. Average daily demand (ADD) can then be used to calculate maximum daily demand (MDD) and peak hour demand (PHD) by application of peaking factors. These factors are multipliers used to adjust ADD and provide estimates of MDD or PHD flow rates. The peaking factors are best developed for each system based on recorded water usage rates; for this study, peaking factors have been developed using records spanning a five year period.

The overall ADD for the years 2007 through 2012 has been 0.38 MGD. The average MDD for 2007 through 2012 has been 0.88 MGD. During the past five years, MDD has been an average of 2.3 times the ADD. This peaking factor was at a high of 2.7 in 2007, and a low of 2.0 in 2011. Published literature indicates a MDD/ADD ratio between 1.5 and 3.0; Bayfield's average of 2.3 is in the middle of the predicted range. MDD water consumption is typically used to determine capacity requirements in the treatment facility.

PHD is not as easily measured as ADD or MDD. An accurate measure of PHD would require detailed measurements taken throughout the distribution system. An approximation of PHD can be made using the hydraulic model of the system; based on modeled data, the current PHD is estimated to be approximately 1.4 MGD. The estimated PHD is 3.7 times the average ADD. Published literature indicates the ratio of PHD/ADD ranging from 2.5 to 5.0. PHD consumption rate is commonly used to determine minimum sizing of distribution system components. Table 4 (below) summarizes the peaking factors determined for the Bayfield water system.

TABLE 4 – SUMMARY OF PEAKING FACTORS

Description	Multiplier
Average Daily Demand (ADD)	1.0
Maximum Daily Demand (MDD)	2.3
Peak Hour Demand (PHD)	3.7

4.4 PROJECTED FUTURE WATER DEMAND

Estimates of future population growth used in making the evaluations included in this report were prepared by Souder Miller & Associates (SMA); data representing past and current population are based on U.S. census and State demographer records. Utilizing the available population data and growth projection (presented above in Figure 1) along with the long term average per capita demand of 160 gpd, allows for the calculation of projected future water demands. This leads to a projected average demand of 0.45 MGD in 2015, increasing to 0.67 MGD by 2025. The historic and anticipated future average daily demand is presented below in Figure 5.

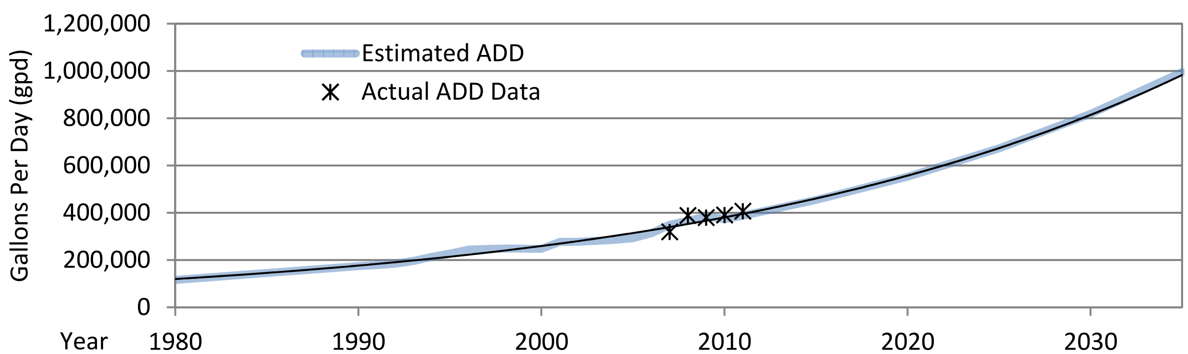
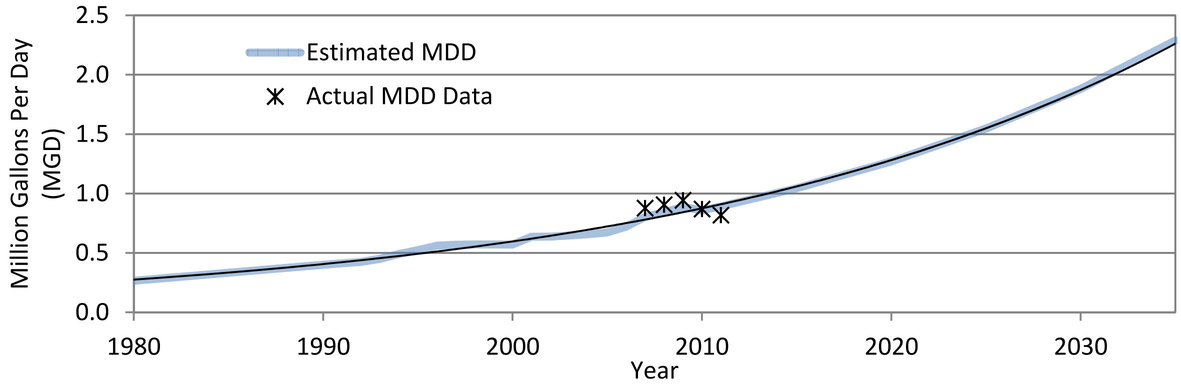


FIGURE 5 – ESTIMATED AVERAGE DAILY WATER DEMAND

Utilizing the above estimate of ADD usage, in conjunction with the average peaking factor of 2.3, allows prediction of future MDD usage. Treatment plants are typically designed to meet the maximum day usage, so a MDD is useful for estimating the remaining lifespan of the existing facility. Figure 6 presents the Estimated MDD usage. Based on this data, the existing water plant is projected to be capable of meeting MDD usage through 2020; MDD will likely exceed existing plant capacity at some time between 2020 and 2025.

FIGURE 6 – ESTIMATED MAXIMUM DAILY WATER DEMAND



To aid in the evaluation of distribution system components, an estimate based on the PHD peaking factor of 3.7 is presented below, in Figure 7. In order to avoid low pressures and the associated risk of pipe failures, the distribution system must be capable of meeting the peak hour usage rates. Table 5 (below) presents the current and future projected water demands.

FIGURE 7 – ESTIMATED PEAK HOUR DEMAND

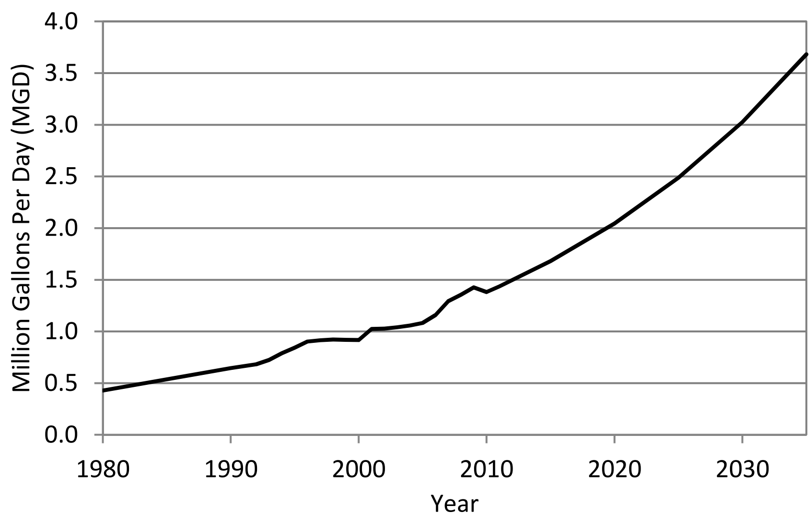
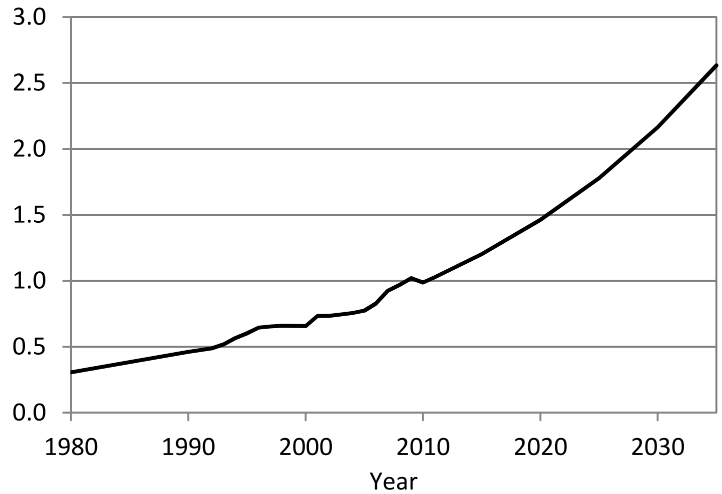


TABLE 5 – PROJECTED WATER DISTRIBUTION DEMAND

Description	Existing Service Area (2012)	Future Service Area (2035)
Average Daily Demand (ADD, 160 gpcd)	0.40 MGD	1.00 MGD
Winter Average Daily Demand (121 gpcd)	0.31 MGD	0.75 MGD
Summer Average Daily Demand (216 gpcd)	0.55 MGD	1.34 MGD
Maximum Daily Demand (MDD, 2.3·ADD)	0.93 MGD	2.29 MGD
Peak Hour Demand (PHD, 3.7·ADD)	1.50 MGD	3.68 MGD

Raw water demand is projected to be 15 percent higher than distribution demand due to filter backwash requirements, and other minor uses within the treatment facility; an estimate of MDD raw water demand is presented in Figure 8 below.

FIGURE 8 – ESTIMATED RAW WATER DEMAND



5.0 EXISTING RAW WATER SUPPLY

All current water sources arrive at the Treatment Facility via the Pine River. The Raw Water Reservoir is filled using either the Los Pinos Ditch or the Los Pinos Pump Station. Additional raw water volume is also stored in Vallecito Lake, and may be called upon if the Town's direct flow rights are curtailed. Information pertaining to water rights is based upon a report completed in 2003 by Wright Water Engineers, the text of which is presented in Appendix D.

5.1 SURFACE WATER SOURCES

The Town of Bayfield owns two (2) separate surface water rights, each derived from the Pine River. The Town also holds storage capacity within Vallecito Reservoir. Both surface water rights are conveyed primarily through irrigation ditches that run adjacent to the Town water treatment plant; each ditch derives flow from the Pine River. The Los Pinos Ditch provides the bulk of the Town's rights, while the Schroeder Ditch conveys the remainder of the Town's rights.

The Town also maintains the Los Pinos Pump Station to convey water directly from the Pine River to the WTP's raw water reservoir. The Pump Station is permitted as an optional means to convey the Town's municipal permitted water rights.

5.2 WATER RIGHTS FOR SURFACE WATER SOURCES

The municipal rights owned by the Town may be conveyed by means of the respective ditches or using the Los Pinos Pump Station. The Pump Station is permitted to convey the full 1.8 cfs municipal right at any time that higher priority rights are not taking precedence.

The Town holds a Priority 4 Right to 2.869 cfs of water from the Los Pinos Ditch; 1.0 cfs of this right has been transferred to municipal use, the remaining 1.869 cfs is limited to irrigation uses. The Los Pinos Irrigating Ditch Company imposes an approximately 15-20 percent water loss on all rights, to make up for losses within the ditch itself. Due to this loss, the Town's 1.0 cfs municipal Right to Los Pinos Ditch water is reduced to not more than 0.85 cfs. This right can only be exercised via the Los Pinos Pump Station when its water cannot reasonably be carried through the Ditch, such as when the Ditch is being cleaned or repaired, when the Town has a legitimate need to pump water through the Pump Station for maintenance, for water quality, when the ditch is subjected to freezing such that it cannot carry the Town's water, and for other legitimate reasons related to the treatment of the water or other similar purposes, or as the Ditch Company otherwise agrees in writing.

An additional Priority 12 Right to 1.737 cfs is conveyed by the Schroeder Ditch; 0.785 cfs of this right has been transferred to municipal use, the remaining 0.952 cfs is limited to irrigation uses. There is no existing means to convey water from the Schroeder Ditch to the raw water reservoir (the Schroeder Ditch Company uses 15% for water loss which includes evaporation and ground saturation); the municipal rights on the Schroeder Ditch currently can only be exercised via the Los Pinos Pump Station.

Town has a third party contract with PRID for 90 acre feet (30 AF – Leased and 60 AF –Standby) of storage water annually. The 30 AF is available each year if the town's water rights fall out of priority. The 60 AF is also available annually; however, once standby water is converted to leased water, the water is leased for the duration of the contract. A copy of the Third Party Contract for Lease of Prin River Project Water is in Appendix I. Raw Water Quality

The water quality of the Pine River is considered good, and with treatment can supply drinking water that meets drinking water Maximum Contaminant Levels (MCLs) and other drinking water standards established by the U.S. Environmental Protection Agency and enforced by the Colorado Department of Public Health & Environment (CDPHE). Table 6 (below) summarizes the typical raw water quality parameters.

In November 2004, CDPHE published a Source Water Assessment Report for the Town of Bayfield's drinking water supply. (Available online at: http://emaps.dphe.state.co.us/website/SWAP_Summary/Counties/La_Plata/134030-Bayfield_Town_of_SW.pdf) The report concluded that the susceptibility of the Town's drinking water source was in the range of moderate to moderately high.

In addition to the testing of physical and chemical constituents of the raw water, the Town of Bayfield has also performed yearly Microbiological Particulate Analysis (MPA) testing of the raw water quality. MPA results for the period 2007 through 2012 indicate results consistent with a typical good quality surface water supply source.

TABLE 6 – TYPICAL RAW WATER QUALITY

Parameter	Typical Range of Analytical Results
Total Organic Carbon (TOC) (mg/L)	1.25 to 1.5
pH	7.0 - 7.75
Alkalinity (mg/L as $CaCO_3$)	50 to 250
Turbidity (NTU)	0.5 - 100
Specific Ultraviolet Absorption (SUVA) (L/mg-m)	2.0 - 3.0
Temperature (˚C)	4-12
Total iron (mg/L)	0 - 0.3
Total manganese (mg/L)	0 - 0.05
Hardness (mg/L as $CaCO_3$)	100 - 200

6.0 HYDRAULIC MODEL

Computer based hydraulic models can be a great tool to aid in analysis and design of distribution systems. Every major component in the distribution system, from pumps and pipes, to valves and tanks, can be represented in a model. Pumps can be modeled based on actual output capacity curves, pipes and valves assigned head-loss factors and set to be open or closed, and tanks can be set to accurately reflect volume equalization and pressure maintenance effects. A properly calibrated model is an excellent resource for evaluating existing system conditions and planning for future improvements.

The hydraulic modeling analysis has been completed using WaterGEMS V8i distribution modeling software. This software allows for the development of static and dynamic models of the existing system that can be used to evaluate existing conditions and potential system improvements driven by projected future growth.

6.1 HYDRAULIC MODEL CONDITIONS

The physical components integrated in the hydraulic model are based on a recent survey completed by Pinnacle Surveying for the Town that included all major components of the distribution system. Storage tanks have been entered in the model to reflect the real world elevations and level controls. Similarly, the pumping capacity at the treatment facility and the distribution booster pumps are modeled to reflect the capacity of their physical counterparts.

User demands within the model are based on current and predicted future water usage. The distribution of these demands is based on a distributed allocation of ERT's completed by SMA; this provided both current and future allocations of equivalent unit demands throughout the current and future service areas (see ERT Map in Appendix A). Hourly variations in user demand are based on a diurnal curve developed from measured sewer flows in the Bayfield collection system; this diurnal curve allows for evaluation of predicted peak hourly demands and the associated peak velocities within the distribution system.

6.2 HYDRAULIC MODEL CALIBRATION

Initial calibration is a phased process, one that can lead to discovery of unknown and/or forgotten restrictions and connections within the system. Calibration of the model involves comparing measured pressures in the distribution system to representative locations within the computer model. Any model is only as good as the data it is built from and the calibration achieved based on that data. A properly calibrated model is not a finished product, but a work in progress; a work that must be kept up to date as changes are made in the physical system that is being modeled.

Calibration is based on the Town's hydrant testing records. The Town Public Works Department conducts the hydrant flow tests and submits the data to the local fire protection district for their records. The testing records provide three data points for each hydrant, two of which are useful for calibrating the computer model; each data point is a pressure reading. The first is static pressure, which is the main pressure recorded with hydrant closed. The other two points are recorded with the hydrant opened to provide maximum flow. Residual pressure is that which remains in the pipe while the hydrant is flowing, while pilot pressure is that recorded outside the hydrant using a standard pilot gauge. Static and residual pressures can be used for calibrating the computer model.

A minimum standard for calibration was determined through literature review (Walski, Journal for Water Resource Planning & Management, ASCE, vol. 109, issue 4). Average and maximum deviation between modeled and measured data is desired to be within 5 and 15 feet respectively; this is a reasonable expectation for calibration when using a good quality data set. Lower quality data could be cause for a reduced expectation of calibration accuracy. The computer model has been calibrated in comparison to the Town's hydrant flow testing records using a six year average (2007 through 2012) of available data to create a representative set for calibration purposes. The Town maintains approximately 150 fire hydrants throughout the distribution system; each was evaluated for consistency of measurements over the 6 year averaging period. Thirty (30) hydrants (distributed throughout the system) showing the best consistency were selected to use for calibrating the model. Measured hydrant data is presented along-side the calibration results in Appendix E.

The first step in model calibration is to achieve agreement under a static scenario. The static pressures measured during hydrant testing are ideal for this; it is reasonable to assume that all normal user demands were in place at the time of measurements, making this easy to approximate in a computerized model. Measured data shows a maximum standard deviation of 10 psi within the calibration data set. Calibration efforts were able to achieve modeled results within one (1) standard deviation (or less) of the measured pressures at all hydrants used for the calibration. The static calibration using all 30 hydrants achieved a differential of 3.6 (average) and 14.4 (maximum) feet in comparison to measured data. In comparison to the goal of 5 and 15 (average and maximum) this calibration is considered good.

The second phase of calibration utilizes the residual pressure measured during hydrant flow testing. This requires 'opening' a hydrant in the model to see what pressure remains (the residual) when the hydrant is providing its maximum available fire flow. Note that the hydrant flow rate itself is secondary in this analysis; it is the residual pressure that is being evaluated. This is a much more demanding calibration step, as velocity and friction losses become a significant factor of the system. The measured data shows a maximum standard deviation of nearly 17 psi within the 30 hydrants used for calibration. The calibrated model achieved agreement within 1 standard deviation for all but 1 hydrant. As calibrated, the residual pressure using all 30 hydrants shows 4.2 and 13.2 feet for average and maximum differentials. This calibration result is also considered good. (Refer to Appendix E for more detail on the calibration data.)

7.0 WATER SYSTEM ANALYSIS

7.1 GENERAL

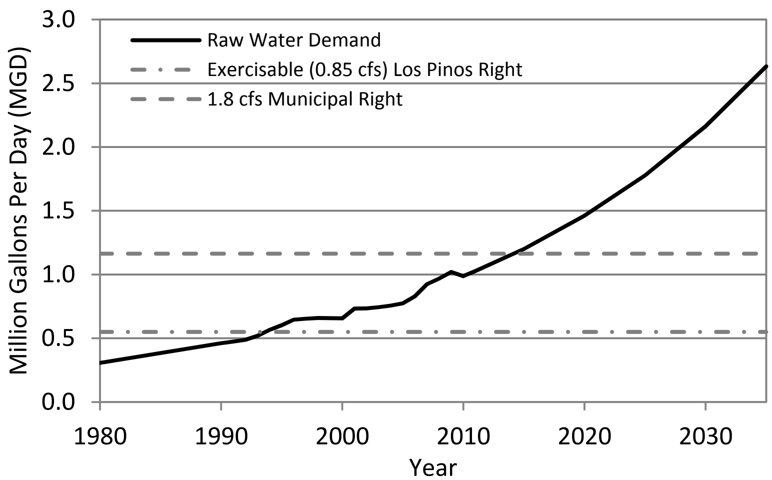
A broad analysis of the ability of existing systems to meet current and future demands can be completed by using the information in Sections 2.0 through 5.0, above. The existing systems were described in Section 2.0. Projected population growth and associated increases in water demand are presented in Section 3.0, while Section 4.0 provided a synopsis of current and future water supply requirements. Finally, Section 5.0 details the development and calibration accuracy achieved within the computerized hydraulic model. All analyses presented below are the result of the cumulative understanding developed through the previous sections of the report.

The IGA between the Town and LAPLAWD has established a list of Joint Use Facilities and further defined an allotment of 0.75 MGD of treatment capacity to LAPLAWD. Using the 0.75 MGD as a ratio of the total proposed treatment capacity of 2.5 MGD for the WTP Improvements Project indicates a 30/70 split (LAPLAWD/Town) in allotment of Joint Use Facility capacity. While the 30/70 split is not specifically defined in the IGA, it is used in the following evaluations to provide a baseline of capacity available to the Town. This ratio is only applied to evaluation of the IGA defined Joint Use Facilities.

7.2 SURFACE WATER SUPPLY EVALUATION

A minimum quantity for raw water supply is typically based on MDD water usage plus losses associated with filter backwash and other treatment plant uses; these additional losses are projected to be about 15 percent of total demand. Figure 9 (below) depicts the predicted raw water demands in relation to existing municipal water rights.

FIGURE 9 – COMPARISON OF PROJECTED RAW WATER DEMAND AND EXISTING WATER RIGHTS



As shown in Figure 9, the Town's currently held municipal water rights are projected to be sufficient to supply predicted MDD usage rates through 2015. The Town needs to have additional, municipally allocated, water rights within the next few years.

If the full 2.869 cfs right on the Los Pinos Ditch were converted to municipal use, that would provide a raw water supply of almost 1.85 MGD (not reduced by the Ditch Company imposed 15 to 20% loss). This quantity is projected to be sufficient to meet MDD usage rates through year 2020.

Converting the remainder of the currently held Town water rights from irrigation to municipal use would provide an additional 2.821 cfs (1.82 MGD) (NOTE: This does not account for the loss during the transition from irrigation to municipal or imposed water loss). A portion of the additional total would still be Priority 12, and subject to calls from senior rights holders. For non-drought conditions while the full allotted rights are in priority, the combination of the existing municipal and converted irrigation raw water supply is projected to be sufficient to provide for Town needs through approximately 2028. Except in times of drought, the Town has sufficient water rights to meet current average demand. If the existing priority 12 municipal right goes out of priority, the facility is limited to a raw water supply of 1 cfs (0.65 MGD). High user demand is typically experienced at the same time water rights priorities become an issue, and the average MDD for the previous 6 years has been 0.88 MGD; to meet the MDD demand (and maintain reservoir level) requires a minimum of 1.4 cfs in active water rights. Storage in Vallecito Reservoir can be called upon to make up potential raw water supply shortfalls; however converting the remaining priority 4 rights could help reduce dependence on reservoir storage; although if the water falls out of priority, the Town will need to augment its supply with Vallecito storage water. The Town is projected to require additional raw water rights in the next fifteen years, before year 2028.

7.2.1 Surface Water Intake Flume

The intake flume (between the Los Pinos ditch and raw water reservoir) is well sized for measuring raw water flow throughout the future of the facility; a standard 9-inch Parshall flume is capable of measuring flows of more than 8 cfs. Parshall flumes are considered accurate to between 3 and 5 percent of flow rate, unless limited by submergence. Submergence occurs when the outfall from the flume is restricted; as can be the case at the Los Pinos Ditch intake structure. As currently installed, a 10-inch diameter pipe carries flow between the flume and the raw water reservoir, leading to flume submergence at peak flow rates.

As part of the WTP Improvements project for construction in 2014 under the current IGA between the Town and LAPLAWD, a new intake channel and measurement flume is to be installed. This will eliminate the existing submergence issues and provide effective flow measurement for the current and future flow ranges.

7.3 TREATMENT CAPACITY ANALYSIS

The treatment facility must be capable of meeting the MDD usage rates. Currently the rated treatment capacity is 1.5 MGD. Figure 6 (above) depicts the predicted MDD usage through 2035. MDD demand is projected to surpass existing treatment capacity sometime near to 2025. Near the end of the 20 year planning period, in 2032, the MDD demand is predicted to reach approximately 2 MGD.

The WTP Improvements Project under the IGA will add an additional 0.25 MGD of treatment capacity to the Town’s allocation, bringing the Town’s capacity up to 1.75 MGD. This additional capacity should provide for Town water supply needs through approximately 2028; it is recommended that the Town plan to increase available capacity before this time. It should be noted that the WTP Improvements project under the IGA has been designed for a future additional1.0 MGD of treatment capacity through the installation of a second ActiFlo pretreatment and media filter unit in a future capital plant expansion project, bringing the total treatment capacity to 3.5 MGD.

7.4 DISINFECTION CAPACITY ANALYSIS

The existing clearwells are undersized for disinfection purposes, based upon a calculated total of 0.65 log inactivation credit achieved in the clearwells (see Table 4 above). This leaves a considerable burden on the removal credit achieved through filtration; in order to achieve 3.0 log credits the filters must achieve 2.4 log credits through removal. Ease of achieving this credit is dependent on the raw water quality: the cleaner the raw water the more difficult it is to attain the removal requirement. Improving baffling would improve disinfection characteristics, but the key limiting factor for the existing clearwells is their small size.

The IGA related expansion project will provide a new clear well to serve the entire facility. The new clear well, in conjunction with a UV disinfection system, has been sized to provide for disinfection needs up to the future planned flow capacity of 3.5 MGD.

7.5 PRESSURE ZONE ANALYSIS

The existing distribution system is separated into five pressure zones, divided by tanks and pressure reducing valves. Two of the zones are along Dove Ranch Road, in the most northern section of the system. The North Central and South Central zones (the largest two zones) are separated by a pair of PRVs. The fifth pressure zone encompasses the downtown area. A map and a schematic depiction of the pressure zones are presented in Appendix C.

The Upper Dove Ranch pressure zone is supplied water from the 0.25 MG Highlands concrete storage tank. The hydraulic grade within this zone varies from 7,394 to 7,153 feet. A PRV along Dove Ranch Road divides the Upper zone from the Lower Dove Ranch pressure zone; within the Lower zone, hydraulic grade varies from 7,292 to 7,017 feet. Currently the Highlands concrete tank supplies the Upper and Lower Dove Ranch zones exclusively.

The North Central pressure zone contains the remaining three (steel) Tamarack storage tanks; under normal operation the steel tanks are hydraulically connected and provide a combined capacity of 1.5 MG. The WTP is also located in the North Central zone, providing water as needed to maintain level in the steel tanks. A pump station located adjacent to the steel tanks provides water to fill the Highlands concrete storage tank. Hydraulic grade in the North Central zone varies from 7,181 to 6,973 feet. Within this pressure zone, houses have been built too close in elevation to the storage tanks, resulting in low service pressure (<40 psi) for customers within approximately a one-fifth (1/5) mile radius of the tanks. Development of a resolution to this issue will require additional analysis to develop an engineered solution, and is beyond the scope of this report.

The South Central pressure zone is separated from the North Central zone by two PRVs. One of the PRVs is located along Mountain View Drive (near the Middle School), while the other is just north of highway 160 near the Shell convenience store; despite the geographical separation, based upon the survey data these two valves are within 3 feet of elevation to one another. The South Central zone hydraulic grade is between 7,292 and 7,017 feet.

The Downtown pressure zone is separated from the South Central zone by a PRV on the edge of Foxfire Road. The Town recently added a PRV along Bayfield Parkway at North Street that will provide beneficial redundancy to the Downtown zone water supply. Hydraulic grade within the Downtown pressure zone varies from 7,073 to 6,850.

7.6 STORAGE CAPACITY ANALYSIS

The Town currently has a total storage capacity of 1.75 MG. A typical design factor for sizing storage requirements (not including fire flow needs) is to plan for at least 25 percent of the MDD as equalization storage capacity. Based on the 25 percent of MDD criteria, the existing storage capacity is projected to be sufficient throughout the twenty year planning period.

In the Town's 2005 Comprehensive Plan a goal of developing and maintaining a two (2) day storage capacity was set forth. Completion of the 1 MG steel and 0.25 MG concrete tanks (in 2007 and 2008 respectively) has achieved that goal. Two (2) days storage for MDD usage rates requires a minimum of 1.76 MG.

7.7 PUMPING CAPACITY ANALYSIS

7.7.1 Raw Water Pumping

The raw water (Los Pinos) pump station, located adjacent to the Pine River, was originally designed to provide a flow rate of 600 gpm at 125 feet; however the pumps are now only capable of about half the original rating. Due to age of the installation, original pump curves have proven difficult to locate; Figure 10 below presents a calculated system curve. Based on existing municipal rights, this station is permitted to withdraw up to 800 gpm (1.15 MGD or 1.8 cfs).

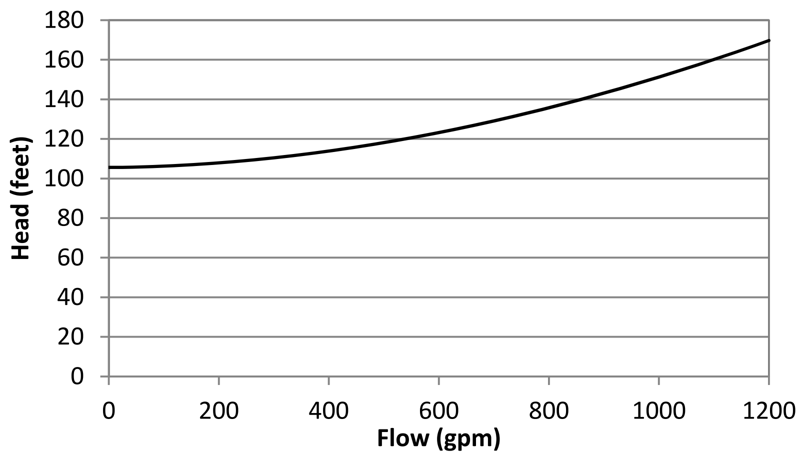


FIGURE 10 – RAW WATER PUMP STATION SYSTEM CURVES

The Los Pinos Pump Station will be reworked as part of the IGA related WTP Improvements project. Both existing pumps will be replaced and the air valve stations along the associated pipeline will be rebuilt. The upgraded Pump Station design capacity is 1.5 MGD.

7.7.2 Finished Water Pumping

There are three (3) finished water pumps in service at the treatment plant. Two of the pumps are rated to provide 350 gpm at 250 feet TDH; they have been in service since 1997. The third pump was designed to provide 700 gpm at 275 feet TDH; this pump has been in service since 2003. The existing treatment plant has a design capacity of 1.5 MGD, which is in excess of the capability of the finished water pumps. Figure 11 below presents the system and pump curves for the existing finished water system.

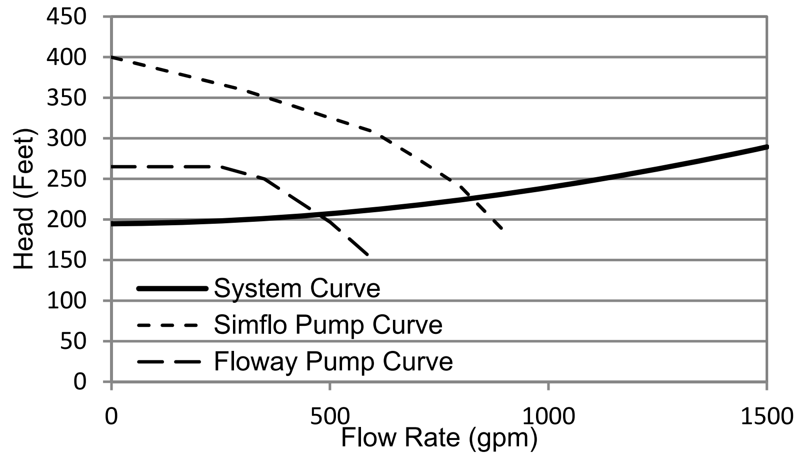


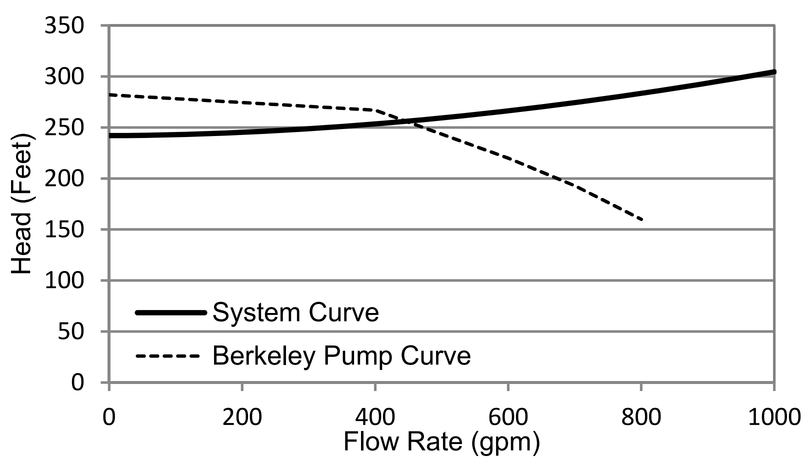
FIGURE 11 – FINISHED WATER SYSTEM AND PUMP CURVES

The finished water pumps will be replaced as part of the IGA related WTP Improvements project. The new finished water pumps will be rated to convey the full 2.5 MGD capacity of the expanded facility. The Town's 70 percent allotment of this capacity is 1.75 MGD; Town MDD demands are projected to exceed this capacity at some point between 2025 and 2030. Pumping capacity should be increased at the same time as treatment capacity, at some time prior to 2028.

7.7.3 Distribution Booster Pumping

The Highlands Booster Pump Station provides water to the Highlands concrete tank; this water supply currently serves only the taps along Dove Ranch Road. Existing daily demand on this system is approximately 0.025 MGD. Assuming no changes to existing inter-connections, future demand is projected to reach 0.24 MGD, and the existing pump is capable of meeting the currently predicted future demands. System and pump curves for the booster pump station are presented in Figure 12 below.

FIGURE 12 – BOOSTER PUMP STATION SYSTEM AND PUMP CURVES



7.8 DISTRIBUTION SYSTEM ANALYSIS

Distribution piping is currently of adequate diameter for water conveyance during peak hour demands. There are areas of the system that should be scheduled for replacement due to age of the piping and associated components. Notable among those areas is the 6-inch diameter AC piping along Mountain View Drive; this is a major conduit between the northern and southern areas of the distribution system.

As future build-out occurs, a few areas will become head loss limited due to high velocity. One of the first sections estimated to become velocity limited is along Lakeside Drive, as water leaves the treatment plant in the existing 10-inch diameter line. This section is estimated to need to be replaced with a larger diameter pipe sometime around 2025 to keep velocity under 4 ft/sec. The 10-inch diameter piping that forms the fill line for the Tamarack steel tanks is estimated to be impacted around the same time as the section along Lakeside Drive (approximately in year 2025).

7.9 FIRE FLOW CAPACITY ANALYSIS

The Town has adopted the International Fire Code (IFC) which defines minimum flow rates and pressures based on building size and classification. There is a general minimum of 1,000 gpm for one hour, for residential structures less than 3,600 sq. ft. All modeled hydrants were evaluated using this residential minimum.

The analysis of fire flow capacity was completed using the computerized distribution model. Within the model, 80 percent of hydrants are capable of meeting the IFC minimum standard. The lowest fire flow, as modeled, was at hydrant #48, which showed a flow of over 800 gpm. In general, the hydrants that failed to meet the IFC minimum were on small diameter mains in older parts of town and/or close in elevation to the storage tanks. All hydrants were able to provide flow for more than 2 hours, well in excess of the 1 hour minimum. A pumper type fire engine may be sufficient to provide the needed head to extract up to 1,000 gpm at all hydrants.

7.10 WATER LOSS EVALUATION

The calculated rate of loss within the distribution system is reported to be approximately 10 percent. Comparing billing records to water production records creates a different picture, with an apparent loss of approximately 37 percent. The Town is aware of this issue and working to correct the discrepancy between metered usage versus production.

7.11 WATER QUALITY COMPLIANCE STRATEGY

The combined efforts of management and operations staff at the Bayfield WTP have resulted in an excellent track record of producing water that meets regulated water quality standards. The only water quality related violations that have been reported during the past six years have been related to missed sampling deadlines; a system for tracking required sampling dates should be implemented if not already in place. There will always be new challenges and changing regulations, and as such the compliance strategy will necessarily remain a work in progress.

While the WTP has consistently met the legal compliance standards, there are seasonal struggles with taste and odor that can impact public perception of water quality. The seasonal taste and odor characteristics could be reduced with the addition of a system for pre-oxidizing the raw water before coagulation and filtration. Pre-oxidation can also reduce the formation of disinfection by-products as well as reducing taste and odor issues.

As part of the IGA related WTP Improvements project, a chlorine dioxide generator will be added to the facility. Pre-oxidizing the raw water with chlorine dioxide should reduce the seasonal taste and odor issues and also reduce disinfection by-product formation.

8.0 ITEMS FOR INCLUSION IN THE CAPITAL IMPROVEMENTS PLAN

8.1 RECOMMENDED IMPROVEMENTS

Based on the current list of capital projects from the Town and using information concerning treatment and collection systems, water usage, water supply sources, and using the water modeling methodology described in Section 4.0, a number of proposed system improvements have been identified in the following sections. A map depicting locations of the major improvements, outlined below, is presented in Appendix F.

8.1.1 Water Supply

The currently allocated municipal, direct-flow water rights are predicted to be insufficient to meet MDD usage rates within the next five years. It is recommended to begin the process of converting the Town's remaining agricultural irrigation rights to municipal usage. The Town should also develop a clear policy for ensuring adequate water rights are conveyed to the Town as property develops, especially at time of annexation. The Town may want to consider requiring development to provide municipal water rights rather than irrigation rights and/or pay for the cost of converting irrigation rights to municipal. There are properties within the Town's service area that have no irrigation water rights. The Town has in some instances accepted cash in-lieu of conveyance of water rights, but likely at insufficient value. The Town should determine an appropriate figure to accurately assess cash in-lieu of water right fees.

8.1.2 Treatment Capacity

Rated capacity of the treatment facility needs to be capable of meeting MDD usage rates. Design and construction of a capacity upgrade should be completed before 2028. To meet projected demands out to 2045 the new facility should be rated for, or be planned for upgrades to reach, a production rate of approximately 3.5 MGD to serve both the Town of Bayfield and LAPLAWD.

8.1.3 Pumps

All potential issues related to existing pumps or pumping systems will be at least partially addressed as part of the IGA related expansion project.

The Los Pinos (raw water) Pump Station will be expanded to a capacity of 1.5 MGD. This equates to a total capacity of 1041.6 gpm, of which 30 percent (312 gpm) is 'allocated' to LAPLAWD with the remaining 70 percent (729 gpm) 'allocated' to the Town. Future required capacity of this pump station is driven by available water rights and their allowable diversion points.

The Finished water pumps are projected to require a sizing increase at about the same time as treatment capacity. Replacement or addition of finished water pumps is recommended to be coordinated with the future treatment capacity upgrades.

8.1.4 Piping

Several areas of piping are projected to become head-loss limited due to high velocity as the service area grows. These impacts are not expected to be significant until later in the twenty year planning period.

The piping leaving the treatment plant (before branches in the system allow for splitting the velocity) is currently 10-inch diameter; this pipe section is about 1100 feet long. Another area that will be impacted is the piping leading to the steel storage tanks off of Tamarack Drive; this section of 10-inch piping is approximately 300 feet long. Each of these sections of 10-inch piping provides the only connection at the respective area of the system, and each will become limiting at the same flow rates. System demand is projected to make these areas problematic sometime around year 2025, and they should be scheduled for replacement before then. A dedicated line to filling the water storage tanks should be part of the future evaluation process; the alignment replacement could be completed incrementally within the existing alignment or follow a different route to provide redundancy should the existing system require repair or replacement.

An additional pipe section that should be considered for replacement is the 6-inch AC piping that runs along Mountain View and crosses Hwy 160. This section is a major connector within the northern and southern areas of the distribution system. While the computer model does not show it becoming velocity limited, a failure within this section could be disastrous to the overall distribution system. No time frame is attached to replacing this section, as this a general observation and recommendation. The Town may also want to consider replacing other AC pipe prior to significant road surface treatment projects; in order to reduce to number of cuts in new asphalt surfaces to repair leaks. Sections of AC pipe include Mesa Avenue and Bayfield Parkway.