Water Master Plan
Big Bear City
Community Services District

Prepared for Big Bear City Community Services District

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Preface

This Big Bear City Community Services District Water Master Plan was prepared jointly by Daniel B. Stephens & Associates, Inc. (DBS&A), Civiltec Engineering, Inc. (Civiltec), and IDModeling, Inc. (IDM). DBS&A prepared Sections 1 and 3, Civiltec prepared Sections 2, 4, 6, 7, and 8, and IDM prepared Section 5. As the prime consultant and author, DBS&A compiled and edited all the contributions to this final report.

Acronyms and Abbreviations

°C/m	degrees Celsius per meter
°F	degrees Fahrenheit
AC	asbestos concrete
ac-ft/d	acre-feet per day
ac-ft/yr	acre-feet per year
ADD	average day demand
AET	actual evapotranspiration
AMR	automated meter reading
ATP	Asset Transfer Program
BBCCSD	Big Bear City Community Service District
BBLDWP	Big Bear Lake Department of Water and Power
BBMWD	Big Bear Municipal Water District
BCM	Basin Characterization Model
CDF	cumulative distribution function
CF	centrifugal
CI	community industrial
CIMIS	California Irrigation Management Information System
CIP	capital improvement plan
Civiltec	Civiltec Engineering, Inc.
COP	certificate of participation
CRWQCB	California Regional Water Quality Control Board
DBS&A	Daniel B. Stephens & Associates, Inc.
DEM	digital elevation model
DFP	diesel fire pump
DPWM	Distributed Parameter Watershed Model
DWR	California Department of Water Resources
DWSRF	Drinking Water State Revolving Fund
EPA	U.S. Environmental Protection Agency

Acronyms and Abbreviations (Continued)

ET	evapotranspiration
ET_0	reference evapotranspiration
feet bgs	feet below ground surface
feet msl	feet above mean sea level
fps	feet per second
GAP	Gap Analysis Project
gpcd	gallons per capita per day
gpd	gallons per day
gpm	gallons per minute
gpm/ft	gallons per minute per foot
GPS	global positioning system
HGL	hydraulic grade line
IDM	IDModeling, Inc.
IRWM	Integrated Regional Water Management
IRWMP	Integrated Regional Water Management Plan
ISO	Insurance Services Office
LAFCO	Local Agency Formation Commission
LAI	leaf area index
MCL	maximum contaminant level
MDD	maximum day demand
MDD+FF	maximum day demand plus fire flow
mg	million gallons
mg/L	milligrams per liter
mgd	million gallons per day
mm	millimeters
mm/d	millimeters per day
mm/d/ºC	millimeters per day per degree Celsius

Acronyms and Abbreviations (Continued)

msl	mean sea level
NCDC	National Climatic Data Center
NWS	National Weather Service
PHD	peak hour demand
PLC	programmable logic controller
PRV	pressure-reducing valve
psi	pounds per square inch
PVC	polyvinyl chloride
R^2	correlation coefficient
RAW	readily available water
REW	readily evaporable water
s/m	seconds per meter
SAE	So & Associates Engineers, Inc.
SCADA	Supervisory Control and Data Acquisition
SCAG	Southern California Association of Governments
SDWA	Safe Drinking Water Act
SOI	sphere of influence
SS	steady-state
TAW	total available water
TB	turbine booster
TBD	to be determined
TDH	total dynamic head
TEW	total evaporable water
USDA	U.S. Department of Agriculture
USGS	U.S. Geological Survey
VFD	variable frequency drive
VGM	van Genuchten-Mualem model

Acronyms and Abbreviations (Continued)

WDF	water duty factor
WMP	water master plan
WSOB	water service operating budget

Executive Summary

Summary

The focus of this Big Bear City Community Services District (BBCCSD) water master plan update has been sources of supply, analyses of the existing system, and hydraulic modeling of the system. Hydrogeologists from Daniel B. Stephens & Associates, Inc. (DBS&A) focused on the water supply sources, primarily on the recharge to the Big Bear Valley groundwater basin. Civiltec Engineering, Inc. (Civiltec) focused on evaluating the physical aspects of the transmission and distribution system, including planned improvements. IDModeling, Inc. (IDM) provided hydraulic computer modeling of the system.

Conclusions

To estimate recharge, or net infiltration, to the bedrock and alluvial aquifers of the basin, DBS&A conducted modeling of the entire Big Bear Valley groundwater basin using a distributed parameter watershed model. Although the BBCCSD has its facilities and distribution areas in the eastern portion of the basin, conducting the modeling efforts for the whole basin allowed for incorporation of a larger dataset and comparison of the model to previous efforts by others.

Average net infiltration was determined to be 24,894 acre-feet, with a median of 17,322 acre-feet over the modeled years (1981 through 2003). Of this net infiltration, almost 60 percent is attributed to the Baldwin Lake portion of the watershed.

Groundwater discharge from the system occurs at springs east of the east boundary of the basin, where alluvium and/or fractured bedrock is saturated and exposed at ground surface. Estimates of the quantity of discharge out of the basin range from 313 to 1,340 acre-feet per year (ac-ft/yr). The total spring-fed runoff into Big Bear Lake ranges from 2,984 to 7,062 ac-ft/yr.

As part of the analysis of existing systems, Civiltec conducted studies of land use and water demand analysis, evaluated physical infrastructure components related to the sources of water

supply, and generated design criteria for the current and future system. In calendar year 2007, which was used as a basis for the master plan, the average daily demand was about 1.1 million gallons. Maximum production was on July 19, 2007, when 2.2 million gallons were demanded of the system. Projections based on a 1.68 percent annual increase result in an ultimate water demand of 1,796 acre-feet in the year 2030.

The BBCCSD meets its demand with 11 active vertical wells, 2 inactive wells, 2 active horizontal wells, and 2 completed springs. Emergency augmentation to this supply includes two interties with adjacent purveyor Big Bear Lake Department of Water and Power; the quality of the delivered water is compliant with Safe Drinking Water Act regulations. In BBCCSD's groundwater supply, fluoride is an issue that is addressed through a variance from the fluoride maximum contaminant level granted by the State of California and blending to decrease concentrations. Most wellheads have a component of disinfection, and water produced from Well 8 is treated for iron and manganese.

In 2007 there were 5,990 service connections in the system, 96.3 percent of which were residential. These connections are served by one main pressure zone and five dependent zones with four storage tanks. Within these tanks are 6.24 million gallons of storage. Pipelines for transmission and distribution range from 2 to 20 inches in diameter and total 81.7 miles. There are six pumping stations with a total of 18 booster pumps.

Design criteria were established with a maximum day demand twice that of the average day demand and a peak hour demand four times the average day demand. Fire flow protection considerations would result in 20 pounds per square inch residual pressure in the lines. Operational water storage was established at 30 percent of maximum day demand, plus 100 percent of emergency capacity, plus fire flow. Pipe velocities were designed at less than 10 feet per second in the distribution system and less than 6 feet per second for transmission at reservoir inlets and outlets. Pumping capacity design criteria were established at maximum day demand with the largest pump unit out of service.

IDM conducted hydraulic modeling of the system, with a focus on model updates, calibration, and evaluation. The modeling pinpointed problem areas where issues can occur so that

improvements in pipelines can be targeted. Updates included all facilities connectivity, configurations, and attributes. All pressure zone designations and zone breaks were reviewed and adjusted, with demands updated to represent recent growth.

Steady-state calibration was based on six hydrant tests conducted in the field with BBCCSD personnel. Twelve model scenarios were created, two for each test (one representing static and one representing dynamic conditions). During steady-state calibration of the model, a reasonably close difference between the model and the field pressure drops was obtained. Based on the model evaluations, system improvement criteria to address deficiencies included (1) providing a main line sized to handle the required fire flow, (2) providing smaller pipeline diameters when pipelines were looped, (3) providing larger pipeline diameters on dead-end mains, (4) improving overall system connectivity for fire protection, and (5) addressing locations where system looping was not currently in place.

Recommendations

Based on the conclusions of this planning effort, several recommendations are presented. With respect to postulated artificial recharge operations, natural recharge to the east portion of the Big Bear Valley groundwater basin appears to be adequate to meet the demands projected for the BBCCSD service area. Hence, artificial recharge is not recommended at this time. Continued efforts toward system monitoring and maintenance should be the primary focus of the capital improvements.

To address existing deficiencies in the system, a suite of primary capital improvements are recommended. Over the course of the time period through 2030, the estimated costs for these improvements would total $16.2 million (2008 dollars). A suite of secondary improvements are also recommended to support development. The primary recommended capital improvements based on the conclusions of this planning effort include the following:

- Replace 10.2 miles of pipeline.
- Install new water meters with an automated meter reading device (fixed network).
- Replace 5 booster pumps to improve pumping capacity.

- Upgrade 11 booster pumps to increase efficiency.
- Equip Well 3B with pump and motor to increase capacity.
- Rehabilitate and maintain wells.
- Enhance fluoride blending pipeline.
- Construct fluoride treatment facility (if necessary).
- Conduct additional studies.

Secondary recommended capital improvements, which would be required to support new development, in-fill development, or redevelopment, are estimated to cost $9.6 million (2008 dollars); much of this cost could be borne by developers. These secondary recommendations would include:

- Replace 4.1 miles of pipeline.
- Establish new pressure zones.
  - Cohila Pressure Zone
    - New gravity storage
    - Tanglewood and Abbott Booster Station reconfiguration
    - Zone boundary reconfiguration
  - Section 16 Pressure Zone
    - New gravity storage
    - New booster station
- Install new fire hydrants.

1. Introduction

1.1 General Description

Located in the San Bernardino Mountains, the Big Bear City Community Services District (BBCCSD) was formed in 1966 by a formation and consolidation election. The BBCCSD encompasses 11 square miles and serves a population of approximately 10,500. It provides water, sewer, fire protection/emergency medical services, and refuse collection services.

On December 4, 2007, the BBCCSD requested proposals from several firms for the update of its water master plan, last issued in 1991. The team of Daniel B. Stephens & Associates, Inc. (DBS&A), Civiltec Engineering, Inc. (Civiltec), and IDModeling, Inc. (IDM) submitted the successful proposal for the planning effort. After board approval, a contract to implement the proposed scope was issued on March 28, 2008. After several meetings with the managers, board members, and BBCCSD staff, the scope was modified slightly to increase the focus of the supply efforts toward available natural recharge, increased support for certain improvements, and ongoing support of supervisory control and data acquisition (SCADA) and modeling efforts. Various members of the team contributed jointly and independently to meet the needs of the BBCCSD as directed.

1.2 Scope of Report

This report represents a collaborative culmination of the master planning efforts for the BBCCSD. The report is divided into eight sections, each representative of efforts conducted in the proposed scope of work. Appendices and modeling efforts are also submitted to provide additional details.

2. Land Use, Population, and Water Requirements

Water requirements vary with population, occupancy, and other social, economic, political, and environmental factors. An adequate and reliable water supply plays a very important role in BBCCSD's sustained development. In order to achieve that sustained development, BBCCSD must be prepared to deliver the estimated future water demand requirements. For this study, future water demand was projected for near-term, ultimate, and build-out conditions. Near-term conditions represent the projected demand in the year 2014, which includes the completion of pending development. Ultimate conditions represent the projected demand in the year 2030, which coincides with the planning horizon of San Bernardino County. Build-out conditions represent the complete development of the BBCCSD Service Area based on current zoning and land use, which directly influence future water demands.

2.1 Sphere of Influence

San Bernardino County Government Code Section 56076 defines a sphere of influence (SOI) as "a plan for the probable physical boundaries and service area of a local agency, as determined by the commission." It is an area within which a city or district may expand, over an undefined period of time, through the annexation process. In simple terms, an SOI is a planning boundary within which a city or district is expected to grow into over time.

The purpose of an SOI is to encourage the "logical and orderly development and coordination of local government agencies so as to advantageously provide for the present and future needs of the county and its communities." The following enumerated items comprise the statement of purpose adopted by San Bernardino County Local Agency Formation Commission (LAFCO) for SOIs:

1. To promote orderly growth of communities, whether or not services are provided by a city or district (board governed or independently governed)

2. To promote coordination of cooperative planning efforts among the county, cities, special districts, and identifiable communities by encouraging compatibility in their respective general plans

3. To guide timely changes in jurisdiction by approving annexations, reorganizations, etc., within an SOI only when reasonable and feasible provision of adequate services is assured

4. To encourage economical use and extension of facilities by assisting governmental agencies in planning the logical and economical extension of governmental facilities and services, thereby avoiding duplication of services

5. To provide assistance to property owners in relating to the proper agency to comprehensively plan for the use of their property

6. To review, update, and/or change existing SOIs periodically to reflect planned, coordinated changes in factors which impact on SOIs

7. To encourage the establishment of urban-type services only within an adopted SOI

Figure 2-1 shows the BBCCSD SOI and service area boundaries. The study area for this Water Master Plan is the service area as shown in Figure 2-1. Portions of the BBCCSD Service Area are served by Big Bear Lake Department of Water and Power (BBLDWP).

2.2 Existing and Proposed Land Use

Land use defines the types of uses for which a lot or parcel may currently be used. The BBCCSD Service Area lies within the jurisdiction of unincorporated San Bernardino County. The 2007 San Bernardino County General Plan identifies the vicinity of Big Bear City as part of the Mountain Planning Region, which consists of those portions of the San Bernardino and San Gabriel ranges that fall within the county border. The goal of the General Plan is to encourage growth that preserves the forest character of the region. This goal is accomplished by limiting

agricultural and industrial land use in preference of low-density residential housing that does not detract from the alpine environment.

Table 2-1 summarizes the land use categories found within the BBCCSD Service Area. Figure 2-2 shows the distribution of these categories throughout the BBCCSD Service Area.

An adjunct to the General Plan is the 2007 Bear Valley Community Plan, which addresses specific planning concerns in the vicinity of Big Bear City. The purpose of the Community Plan is to provide for orderly growth that will preserve the mountain character of the area and protect the area's natural resources. Certain guidelines from the Community Plan have been adopted by this Water Master Plan where they pertain to the unique needs of BBCCSD.

2.3 Existing and Projected Population

San Bernardino County has one of the highest projected annual growth rates in Southern California: 2.37 percent per year between 2000 and 2030 according to population forecasts published by the Southern California Association of Governments (SCAG). According to SCAG, projected growth in the vicinity of Big Bear City is more modest. Big Bear City lies within Census Tract 114, which includes portions of unincorporated San Bernardino County to the east of Big Bear Lake. Based on data published by SCAG for Census Tract 114, the estimated annual growth rate is approximately 0.95 percent over the same period as shown in the following calculation:

$$P = P_0(1+i)^n \quad \Rightarrow \quad i = \left(\frac{P}{P_0}\right)^{\left(\frac{1}{n}\right)} - 1 = \left(\frac{13{,}725}{10{,}337}\right)^{\left(\frac{1}{2030-2000}\right)} - 1 = 0.95\%$$

where P = the estimated population of Census Tract 114 in 2030

P_0 = the population of Census Tract 114 in 2000

n = the number of years under consideration

i = the annual population growth rate

Another population projection for the region was published in the 2007 Bear Valley Community Plan by the San Bernardino County Land Use Services Department. Growth in population, households, and employment was projected for the combined communities of Big Bear City, Fawnskin, Sugarloaf, Erwin Lake, Baldwin Lake, Lake Williams, and Moonridge (Table 2-2). These communities are more representative of the BBCCSD Service Area, and more conservative, than the Census Tract 114 used by SCAG.

The 2000 Urban Water Management Plan estimated the full-time population served by BBCCSD at 7,884. Based on the growth projected in the 2007 Bear Valley Community Plan, the service population will increase to 13,464 by 2030 (Table 2-3).

However, population growth may not be the most effective measure of BBCCSD's future water demand. Due to the recreational nature of the Big Bear City economy, occupancy within the district fluctuates seasonally, typically peaking in July and at the lowest level during the winter. According to U.S. Census 2007 data, the residential vacancy percentage for Big Bear City is approximately 52.3 percent. BBCCSD consumption meter readings for 2007 were analyzed to correlate demand with the percentage of unoccupied residences. Because meters are read bimonthly, the year was divided into 2-month periods, resulting in each meter being read only once per period. The analysis showed that residences with meter readings of 500 cubic feet (ft^3) or less during a 2-month period were consistent with the U.S. Census vacancy percentage. To put this result into perspective, a meter reading of 500 ft^3 is equivalent to an occupancy rate of roughly 9 days per 2-month period, assuming an average household of 2.52 people and an estimated average consumption rate of 167 gallons per capita per day (gpcd):

$$\left(\frac{500\,ft^3}{household}\right)\left(\frac{household}{2.52\ persons}\right)\left(\frac{7.48\ gallons}{ft^3}\right)\left(\frac{person \times day}{167\ gallons}\right) \cong 9\ days$$

In other words, an estimated 52.3 percent of Big Bear City residences are occupied less than five days per month. Therefore, Big Bear City has the potential to experience a large change in demand if full-time occupancy increases, even if the number of service connections remains constant.

A random sampling of 212 residential meters was analyzed to determine how to scale up residential demand to account for the potential increase as a result of 100 percent occupancy. The sample included meter readings from residences within each of the residential land use categories identified in Table 2-1. For 2007, the average demand per unit for all residences in the random sample was found to be 8,568 ft^3 per year per unit. For the same period, the average demand per unit for full-time residences was found to be 13,986 ft^3 per year per unit. A full-time residence was considered any meter with no bimonthly reading lower than 500 ft^3. Dividing the full-time unit demand by the unit demand for all residences yields a scaling factor of 1.63 that may be applied to the average residential demand to determine the potential demand at full occupancy:

$$\left(\frac{\text{full time occupancy demand}}{\text{average demand}}\right) = \left(\frac{13{,}986\ \text{ft}^3\ \text{per year per unit}}{8{,}568\ \text{ft}^3\ \text{per year per unit}}\right) \cong 1.63$$

Scaling the projected 2030 population by 1.63 and dividing by an average of 2.52 people per household produces the projected number of households in 2030 at 8,709:

$$\frac{1.63 \times 13{,}464\ persons}{2.52\ persons\ per\ household} \cong 8{,}709\ households$$

This value is slightly lower than the growth rate in new connections derived from BBCCSD data. Over the past 25 years, from 1982 to 2007, the number of new service connections has increased at an annual average rate of 1.48 percent. Based on input from BBCCSD staff, the annual growth rate over the next 5 years is expected to be much lower, at approximately 0.25 percent. Using BBCCSD input and the historical service connection growth rate to project the number of connections in 2030 yields an estimate of 7,901 connections (Table 2-4).

A portion of the total connections serve institutional, commercial, and industrial customers. According to BBCCSD staff, there were 5,795 (96.3 percent) residential connections and 220 (3.7 percent) non-residential connections as of September 2008. Applying the same breakdown of residential and non-residential connections to the 2030 projection for new

connections (Table 2-4) yields 7,609 new residential connections and 292 new non-residential connections.

The top 50 BBCCSD consumers from January 1, 2000 to March 15, 2008 are listed in Table 2-5. As is evident in the table, although the relative number of non-residential connections is quite small, their consumption is more significant, as they make up almost half of the top users and account for 8 of the top 10 consumers. Non-residential demand was calculated by adding the consumption from the 22 non-residential customers listed in Table 2-5 to an average consumption for the remaining non-residential connections equivalent to occupied residences for the same time period. Non-residential demand is estimated to be approximately 13 percent of total consumption.

2.4 Water Meter Codes

The BBCCSD uses 12 water meter rate codes that are generally associated with meter size. Table 2-6 lists the rate codes, meter sizes, number of meters, total demand per rate code, and average demand per meter based on 2007 BBCCSD consumption records.

2.5 Water Loss

Water loss, also called unaccounted-for water, is the difference between water production and water consumption. Typical sources of water loss include leaks or breaks in pipes, inaccurate meters, illegal connections, fire flow, maintenance, and unmetered water use for construction, landscaping, street sweeping, and dust control, among other uses. Because operations are based on water production, any calculations derived from water consumption data must be scaled up to reflect water loss.

Figure 2-3 demonstrates how water production and consumption totals have fluctuated since 2001. The difference between the two curves in Figure 2-3 represents water loss. Over the past five years, the average water loss for the BBCCSD water distribution system was approximately 9.4 percent. As shown in Table 2-7, water loss has increased substantially over the past few years, from 3.8 percent in 2001 to 11.2 percent in 2007.

2.6 Duty of Water and Water Requirements

A water duty factor (WDF) is a tool used to estimate future demand based on area and land use category. Water demand within each of the San Bernardino County land use categories as described in Section 2.2 was analyzed to determine the WDFs.

WDFs were calculated by dividing demand (consumption data scaled up by 9.4 percent to bring them in line with production data) by area. For each San Bernardino County land use category except community industrial (CI), representative lots were chosen and analyzed. The total annual demand for representative lots of each land use category was divided by their total area and tabulated in Table 2-8. Also included is the average demand per meter for each land use category.

Water demand for industrial purposes is highly dependent on the type of industry and must be examined on a case by case basis. No industrial customers were considered in this analysis.

2.6.1 How to Use a Water Duty Factor

WDFs can be used to estimate the demand generated by a future development by performing the following calculations using the appropriate factors from Table 2-8 and taking the higher of the two totals.

$$\text{WDF} * \text{Area (acres)} = \text{Demand}_1 \text{ (ac} - \text{ft/yr)}$$

$$\left(\frac{\text{ac} - \text{ft/yr}}{\text{meter}}\right) * (\text{number of units}) = \text{Demand}_2 \text{ (ac} - \text{ft/yr)}$$

2.6.2 Build-Out Demand

Based on current land use designations and policy as provided in the 2007 Bear Valley Community Plan, build-out demand has been estimated for the BBCCSD Service Area excluding those areas presently served by the BBLDWP. Table 2-9 provides the acreage and

maximum density of each land use category, the number of potential connections, the WDFs developed in Table 2-8, and the potential build-out demands based on those WDFs.

2.6.3 Near-Term Development (2014)

BBCCSD staff have identified three pending or proposed developments that will account for a portion of the projected new connections (Figure 2-4). Table 2-10 summarizes this near-term development. As indicated in Table 2-10, a total of 345 new connections will be made as a result of the three near-term developments.

2.6.4 Ultimate Development (2030)

In addition to the 345 near-term new development connections (Section 2.6.3), an additional 1,566 connections are projected to be added by 2030, for a total of 1,911 new connections. Most of the additional 1,566 connections will be the result of development of unimproved areas within the BBCCSD Service Boundary under the current land use designations. The remaining connections will be due to densification of established areas (e.g., building on vacant lots, splitting large lots into smaller lots) and extension of services to rural areas. Table 2-11 lists the areas that have been identified for potential development, and Figure 2-4 shows the locations of those areas. Densification may occur anywhere within the Service Boundary.

Table 2-12 summarizes projected increases in connections and average day demand (ADD) for the existing, near-term, and ultimate planning periods.

2.7 Demand Fluctuations

The study period for demand fluctuations is the calendar year 2007. Daily BBCCSD production data were used in this analysis. The ADD was calculated by dividing the total volume of water produced during the study period by the total number of days in the period. Volumes for maximum day and minimum day were identified by sorting the data. Because no hourly data were available, peak hour demand (PHD) was calculated as two times the maximum day demand (MDD) in accordance with industry standards (see Appendix A for methodology and

calculations of MDD). Demand fluctuation is represented as a ratio of MDD, PHD, and minimum day demand to the ADD. Table 2-13 summarizes the peaking factors, the flows associated with those factors, and the period when the peaks occurred.

2.8 Fire Flow Requirements

The availability of sufficient flow and water pressure is a basic requirement of the fire department. The goal for residential fire flow, which includes single-family residences and duplexes, is 1,500 gallons per minute (gpm) at 20 pounds per square inch (psi) for a duration of 2 hours; fire hydrant spacing in residential areas shall be 600 feet on center. For all other areas, the goal is 3,000 gpm at 20 psi for 3 hours, with fire hydrant spacing 300 feet on center. These areas include multi-family residential other than duplexes, commercial, institutional, and industrial zoning. Combining simultaneous flow from multiple adjacent fire hydrants to reach the goal of 3,000 gpm is acceptable.

3. Source of Supply

BBCCSD's primary source of water supply is groundwater, whether developed through springs, wells, or adits (horizontal wells). The springs, horizontal wells, and other geographical features in the BBCCSD Service Area are shown in Figure 3-1. BBCCSD wells and their locations are discussed in Sections 3.3 and 4. Precipitation within the watershed is the ultimate source of recharge to the groundwater.

3.1 Water Quality

Groundwater quality within the Big Bear Valley groundwater basin is generally very good. As in many areas where granitic bedrock is a source for sediment, a calcium-bicarbonate water character is prevalent. Such a character indicates good mixing of waters and rapid natural recharge. Water quality issues tend to be limited to fluoride and localized contaminants, namely trichloroethylene (TCE) near Well 3.

Fluoride appears to be concentrated in the deepest alluvial aquifers of the basin, as indicated by zone testing in pilot boreholes for the more recently constructed water wells. Fluoride can be attributed to two sources: (1) hydrothermal springs where fluorite is a significant deposit along veins and (2) evaporitic deposits where the element has been concentrated in prehistoric drying lakes. Although flow from hydrothermal springs may ultimately have contributed to both modern sources, the presence of fluoride in aquifers targeted for extraction by BBCCSD wells is likely associated with the concentration of the element in waters of drying lakes throughout the past few hundred thousand years. Current researchers (Kirby et al., 2006) have been using cores through Baldwin Lake to identify ages and paleoclimatic indicators of the depositional environments. Findings indicate a very old age (about 40,000 years of record within the upper 25 feet of sediment) of intermittently drying lake sediments in the area, which would support the concept of a long-term buildup of fluorite concentrations in lake water and resultant evaporite deposits. When the dry lake sediments are buried and vertically emplaced adjacent to the sands and gravels of aquifers, they can dissolve into groundwater. Extraction of the groundwater from these aquifers can induce an advective component of fluoride contribution from the finer-grained units of ancient lacustrine sediments.

In recent years, TCE concentrations have been increasingly present in a few wells near the east end of the airport. The source(s) of this contaminant is not yet known, but some investigation should be conducted to determine the full extent of the problem. The highest TCE concentrations have been detected in Well 3; this is likely due to a combination of factors, including the well's proximity to the source(s), shallow perforations, and lack of a competent deep annular seal. Wells 3A and 3B were constructed to replace Well 3, which should be either modified or destroyed to prevent downward migration of contamination to deeper usable aquifers.

3.2 Spring Water Reliability

The BBCCSD extracts groundwater from two completed springs in the south end of the alluvial portion of the Erwin subunit of the Baldwin subwatershed. Greenspot Spring and Fish Hatchery Spring (Figure 3-1) appear to flow due to a combination of factors, including a shallowing of bedrock depths and a facies change of alluvial fan deposits from coarser-grained gravels and sands to a finer clayey unit, possibly associated with ancient lake deposits. Recharge from some of the highest elevations and areas that receive the greatest amounts of precipitation within the Bear Valley tributary area flows as groundwater that is impinged on these structures and forced toward the surface as springs. Decades of development from these springs, a lack of significant nearby pumping of groundwater, and a lack of development in much of the area between the springs and the recharge areas indicate that these springs will continue to be a highly reliable source for the BBCCSD.

In connection with the Big Bear Valley watershed recharge model efforts (Section 3.4), DBS&A reviewed the alluvial geology, topography, creek flow, and spring flow in the Erwin subunit. Field observations identified the discharging groundwater and the resulting spring flow to Arrestre Creek (Figure 3-1). Based on these factors, DBS&A has estimated a range of potential losses from the groundwater basin to Arrestre Creek to the east to be 313 to 1,340 ac-ft/yr. This flow would occur where alluvium from Gocke Valley (Figure 3-1) crops out in the canyon wall to the east of the basin. Several seeps and springs are present in that area, and flow in Arrestre Creek increases significantly as the creek traverses the area east of the study area. Discharge would also occur from the northeastern portion of the creek that links Gocke Valley and Erwin

Lake (Figure 3-1), in locations northeast of the east end of Deadman's Ridge (Figure 3-1). Recharge to the Gocke Valley alluvium and bedrock to the south is potentially susceptible to outflow from the basin as springs to the east. The recharge potentially lost is between 313 and 1,340 ac-ft/yr, likely on the lower end due to surface flow and pumped groundwater that would remain in the system. Recharge that is tributary to the portion of the basin overlain by Deadman's Lake may also be susceptible to this outflow, as it is near a surface water divide that induces flow either east or west of Deadman's Ridge. However, several wells extract groundwater from the alluvium in this area, and actual outflow would likely remain within the 313- to 1,340-ac-ft/yr range.

Potentially lost recharge occurs in an area of less than 1 square mile of the study-area groundwater basin. The estimate of this discharge was calculated based on the cells of the recharge model affected by the discharge, their respective areas, and the amount of recharge calculated for those cells. As such, the estimate can be thought of as the amount of recharge "lost," rather than a direct estimate of discharge. Nonetheless, the discharge is real and has been observed. Additional investigation beyond the scope of DBS&A's contract with the BBCCSD is required to provide a more rigorous estimate of the discharge.

In addition to the BBCCSD-developed springs, other springs east and northeast of the basin provide additional exit points for Big Bear Valley groundwater basin water. Several named springs (Figure 3-1) are fed by water within the bedrock that is impounded by a low-angle reverse fault and lithologic changes:

- Smart Spring
- Squirrel Spring
- Canyon Spring (may also provide exit of alluvial groundwater)
- Granite Spring

Volumes of discharge from these springs are not quantified. Some have been "completed" by users along Lone Valley (Figure 3-1). Outflow from these extra-basin springs, especially those associated with faulting and bedrock northeast of the basin, could likely increase as fractures

are mobilized during earthquakes. Such a phenomenon could conceptually have occurred when water levels in the basin decreased following the Big Bear earthquakes of 1992.

3.3 Well Water

The current well water sources for the BBCCSD include 11 active vertical wells, 2 inactive vertical wells, and 2 horizontal wells (Section 4). Wells 1, 1B, 2, 3, 3A, 4, 4A, 6, 8, 9, and 10 are currently active. Wells 5 and 7, which have low production rates, are currently inactive. Well 3B, which will replace Well 3, has been constructed but is not yet registered as an active source. Van Dusen Wells 1 and 2 comprise the active slant (horizontal) wells (Figure 3-1).

Pumps lift groundwater into the transmission system from the vertical wells. The slant wells flow under gravity into the BBCCSD system. Active wells are linked into the BBCCSD's SCADA system, which automatically records water levels, pumping rates, and certain water quality parameters. The following paragraphs describe conditions of active wells as observed during a field visit by DBS&A on June 9, 2008.

Van Dusen Wells 1 and 2 are near the locations where springs historically flowed from granitic bedrock fractures in Van Dusen Canyon. Both wells are completed with steel piping and redwood boxes. Each well pipe has a slight downward slant, at an angle of approximately 3.5 degrees from horizontal as measured at ground surface. Van Dusen Well 1 trends 305 degrees (55 degrees west of north) and was advanced 395 feet into the mountainside. Van Dusen Well 2 trends 290 degrees (20 degrees north of west) and was advanced 400 feet into the mountainside. The two Van Dusen slant wells typically produce at a combined flow rate of 40 to 300 gpm, depending on the amount of water in storage and head in the fractured bedrock aquifers supplying the wells.

Well 1 was pumping at 220 gpm during DBS&A's field visit. Earlier in the day, the static water level was 48.6 feet and the pumping level was 71.5 feet; this equates to a specific capacity of 9.6 gallons per minute per foot (gpm/ft) of drawdown. Well 1 is 111 feet deep and consists of a 12-inch-diameter casing.

Well 1B was pumping at 360 gpm during DBS&A's field visit. This 332-foot-deep well was drilled in 1958 and consists of a 16-inch-diameter casing perforated from 100 to 312 feet below ground surface (bgs).

Well 2 was producing at 86 gpm during DBS&A's field visit, with a pumping water level of 60.5 feet. Drilled to a depth of 248 feet bgs in 1958, this 16-inch cased well is perforated from 40 to 218 feet bgs. Water quality is adequate to be pumped directly into the system.

Well 3 was not pumping during DBS&A's field visit as it is designated for emergency use only, but had a static water level of 28.6 feet bgs. This well was drilled to 324 feet bgs in 1958, with its 16-inch-diameter casing perforated from 52 to 115 feet bgs, 135 to 240 feet bgs, and 304 to 316 feet bgs. Due to the recently increasing TCE concentrations and historical high fluoride concentrations, this well should be converted to a triple completion monitoring well as soon as feasible. Depth-discrete casings that isolate the two lowest sets of perforations can provide key information regarding TCE, fluoride, and individual aquifer hydrologic characteristics.

Well 3A, also known to be high in fluoride, was pumping at 150 gpm during DBS&A's field visit. The pumping water level was 42.7 feet, drawing down 26.2 feet from a static level of 16.5 feet. The specific capacity of the well was 5.7 gpm/ft of drawdown in this 203-foot-deep well drilled in 1987. Perforations are from 91 to 129 feet bgs and 136 to 166 feet bgs.

Well 3B has yet to be equipped, but was drilled in 2000 and cased to a depth of 810 feet bgs. Anticipated production rates of 1,000 gpm were recommended, but because of fluoride concerns, the produced water will likely need treatment or blending. The 16-inch-diameter casing is perforated from 130 to 250 feet bgs, 295 to 530 feet bgs, and 630 to 790 feet bgs.

Well 4 was drilled in 1980 to a depth of 110 feet. Based on video log surveys, the 12-inch-diameter steel casing has louvered perforations from 63 to 102 feet. Typical pumping rates are 190 gpm with about 30 feet of drawdown; hence, the specific capacity of the well is 6.3 gpm/ft of drawdown. The pump is set at 84 feet with a pressure transducer set at 80 feet.

Well 4A was drilled in 1987 to 120 feet and cased to a depth of 116 feet. The 8-inch diameter casing is equipped with continuous wire-wrapped screen from 42 to 80 and 86 to 106 feet. Despite the higher perforations, a 50-foot-thick cement annular seal is placed around a 24-inch-diameter conductor casing; gravel approaches the ground surface between the conductor casing and the 8-inch-diameter casing. The typical production rate is 100 gpm using the submersible pump set at 85 feet. Approximately 30 feet of drawdown is typical in this well, indicating a specific capacity of 3.3 gpm/ft of drawdown.

Well 6 was drilled to 150 feet bgs and has a 10-inch-diameter casing perforated from 80 to 140 feet bgs. During DBS&A's site visit, Well 6 was pumping at 140 gpm, inducing a pumping water level of 61.1 feet. From a static water level of 23.4 feet, drawdown was 37.7 feet, resulting in a specific capacity of 3.7 gpm/ft of drawdown.

Well 8 was drilled in 2003 and cased to a depth of 380 feet bgs with 12¼-inch-inside diameter casing. Perforations are from 90 to 175 feet bgs, 195 to 245 feet bgs, and 260 to 360 feet bgs. Iron and manganese concentrations require treatment using greensand vessels.

Well 9 was producing groundwater at about 175 gpm with a pumping water level of 163 feet. This 536-foot-deep well is perforated from 200 to 362 feet bgs and 470 to 516 feet bgs. Water quality is adequate for direct pumping into the system.

Well 10 was drilled in 2003 and completed to a depth of 640 feet bgs with perforation intervals from 195 to 295 feet bgs and 545 to 620 feet bgs. During the site visit this well was pumping at 94 gpm with a pumping water level of 188 feet, representing drawdown of 20 feet and a specific capacity of 4.7 gpm/ft drawdown.

3.4 Groundwater Recharge

DBS&A evaluated natural recharge to groundwater within the Big Bear Valley groundwater basin in the southwestern corner of San Bernardino County, California. The objective of the recharge evaluation was to quantify the portion of precipitation that recharges the basin. As shown on the location map (Figure 3-1), the Big Bear Valley groundwater basin is defined

topographically as the area in which precipitation runoff and snowmelt flow toward Big Bear Lake and Baldwin Lake.

DBS&A's recharge evaluation analyzed the surface and shallow subsurface. The method consisted of a basin-wide soil-water balance analysis. Site-specific data collected during the investigation were used in the computations. In addition, the site-specific results were compared to the results of various previous estimates of recharge and perennial safe yield. In general, the project approach used a water balance methodology previously applied in Nevada, one similar to that applied by the U.S. Geological Survey (USGS) in the Big Bear Valley (e.g., Flint et al., 2007).

3.4.1 Basin-Scale Soil-Water Balance Model

A soil-water balance model is a tool that allows evaluation of the magnitude of various components of the hydrologic cycle as it is applied to the soil. Such models have been available for many years (e.g., Leavesley et al., 1983) and applied in arid areas, including Nevada and the Tule Desert (Flint et al., 2004; Flint and Flint, 2007) and Big Bear Valley (Flint et al., 2007). These models generally simulate water within a certain soil depth interval and incorporate topography, the hydraulic properties of soil and bedrock, and meteorological data in order to distribute precipitation among snow sublimation, evapotranspiration, runoff, soil moisture storage, and deep percolation.

In the model, the basin surface is discretized so that the water balance is computed over relatively small areas. It is assumed that the deep percolation below the root zone, sometimes referred to as net infiltration, will eventually become groundwater recharge. There is a lag time before deep percolation reaches the water table and becomes recharge, so the calculated recharge may not be immediately available as extractable groundwater.

These models can be useful predictors of the amount and spatial distribution of recharge at the basin scale, especially when site-specific data are used for model input and small time steps are used. The model that DBS&A used for the Big Bear Valley model is called a distributed parameter watershed model (DPWM) and is described further in Section 3.4.4.

3.4.2 General Description of the Big Bear Basin and Model Domain

BBCCSD is located in Big Bear City, California. The primary source of water supplied to its customers is groundwater derived from the Big Bear Valley groundwater basin. The Big Bear Valley watershed and groundwater basin is located in the San Bernardino Mountains of southwestern San Bernardino County California.

3.4.2.1 Location, Physiography, and Climate

The Big Bear Valley basin lies within the San Bernardino Mountains, a transverse mountain range on the southern edge of the Mojave Desert. The basin is roughly 14 miles long from east to west and 7 miles wide from north to south. There are two lakes in the middle of the basin: Big Bear Lake and the ephemeral Baldwin Lake. Big Bear Lake empties on the west into Bear Creek, which is a tributary of the Santa Ana River. Baldwin Lake sits in a local depression and does not empty to any other body of water. Big Bear Lake lies at an elevation of approximately 6,800 feet above mean sea level (msl) and Baldwin Lake lies at approximately 6,700 feet msl, with the surrounding mountains rising to between approximately 7,800 and 8,600 feet msl. Big Bear Lake is unique among southern California populated areas in that it normally receives significant winter snowfall, averaging around 60 inches at lake level.

The climate in the valley is mild, with temperatures increasing slightly to the east. The warmest month of the year is July, with average high and low temperatures of 80.9°F and 47.1°F, respectively. The coolest month is January, with average high and low temperatures of 47.1°F and 20.7°F, respectively. At the Big Bear Lake Dam, the mean maximum and minimum temperatures are 58.9°F and 30.1°F, respectively. At the Big Bear Lake weather station (currently measured at the fire station), the mean maximum and minimum temperatures are 62.2°F and 31.8°F, respectively. In Big Bear City (measured at the BBCCSD), the mean maximum and minimum temperatures are 64.9°F and 31.9°F, respectively.

3.4.2.2 Precipitation

Precipitation in the Big Bear Valley basin has been measured and recorded at sites along the valley for many decades, but little data are available for the mountains that surround the basin. Precipitation in the mountains should be greater than in the valley, especially to the south where

mountains are the highest within the basin. Precipitation data in the valley show that the amount of precipitation decreases from west to east. At the Big Bear Lake Dam, the mean annual measured precipitation is 35.6 inches. In Big Bear Lake (currently measured at the fire station), the mean annual measured precipitation is 21.8 inches. In Big Bear City (measured at the BBCCSD), the mean annual measured precipitation is 14.5 inches. At the Green Canyon Springs, the mean annual measured precipitation is 15.4 inches. Precipitation normally occurs about 45 days a year.

In June 2008, DBS&A evaluated seven precipitation gauges in the Big Bear Valley basin:

- Two of the precipitation gauges are National Weather Service (NWS) gauges. These gauges are monitored by other agencies and are located at the Bear Valley Dam and the Big Bear Lake Fire Department. They have been in operation for decades and provide long-term precipitation data for the basin.

- The Big Bear Municipal Water District (BBMWD) owns and monitors a precipitation gauge at the Bear Valley Dam, located close to the NWS gauge.

- Three of the evaluated gauges are maintained by the BBCCSD: one gauge at Green Canyon Springs and two at BBCCSD headquarters, although the older of the headquarters' gauges has since been retired. The newer gauge was installed in a different setting, and both gauges were monitored concurrently for many months to provide a means of evaluating differences in measured precipitation.

- The California Irrigation Management Information System (CIMIS) Big Bear Lake weather station #199 (Big Bear CIMIS) is located at the Bear Mountain Golf Course.

Each station was visited to observe what type of precipitation gauge was being used and where each gauge was sited. Elevations and locations of the precipitation gauges and weather stations were measured with a Garmin GPSMAP 60Cx global positioning system (GPS) unit, which is accurate to ±10 feet horizontally and 33 feet vertically.

The NWS precipitation gauges are identified as Big Bear Lake and Big Bear Lake Dam. Both gauges are of the same type with an 8-inch orifice at the top for accepting precipitation. The gauges have been moved since their original dates of installation. Information to only the nearest minute of a degree was available on the detailed siting of stations at their previous locations. The Big Bear Lake gauge was originally installed in 1960 and the Big Bear Lake Dam gauge was installed in 1933. Precipitation data and locations to the nearest minute can be found at the National Climatic Data Center (NCDC) website (NCDC, 2009).

The NWS Big Bear Lake gauge is monitored daily, on behalf of the NWS, by the City of Big Bear Lake Fire Department. The station was not found during the survey, but a description was provided by the fire department. The gauge is located at the Big Bear Lake fire station at an elevation of 6,760 feet msl and 34° 15’ north latitude and 116° 54’ west longitude. The gauge sits in a yard about 4 feet from and slightly below a cinder block wall. The gauge is not near any trees that may affect its performance. In the past, the Big Bear Lake gauge has been sited 1 minute east or west of its present location, but the latitude has not changed. The gauge has no windshield. Precipitation data date back to 1960.

The Big Bear Lake Dam NWS gauge data are collected remotely, on behalf of the NWS, by the San Bernardino County Flood Control District (SBCFCD). Staff from the SBCFCD visit the gauge periodically to download data and empty the gauge. The gauge is on a hill to the southeast of the dam, located at a ground surface elevation of 6,840 feet msl and 34° 14.489’ north latitude and 116° 58.507’ west longitude. In the past, the NWS Big Bear Lake Dam gauge has been sited 1 minute further north and 1 minute further west. The gauge is approximately 12 feet above ground surface and has a windshield to mitigate the effects of air currents. The gauge is not designed to melt snow, so it may fill up to capacity during a prolonged snowfall event. Once the gauge is full, any additional snow would be prevented from entering the gauge and thus being recorded, so the gauge may be misreporting total precipitation. Nearby trees that may not have been mature when the gauge was originally installed may also cause the gauge to misreport precipitation. To accurately record precipitation, the gauge requires open space above the gauge in the shape of an inverted cone (NWS, 1989; WMO, 2006). The upper reaches of some of the trees in the area are within the cone and can possibly deflect precipitation that would have entered the gauge. Precipitation data date back to 1933.

The BBMWD 8-inch precipitation gauge is approximately 25 feet from the NWS Big Bear Lake Dam gauge. The gauge is located at a ground surface elevation of 6,839 feet msl and 34° 14.494' north latitude and 116° 58.507' west longitude. The gauge is approximately 8 feet above ground and has a windshield to mitigate the effects of air currents. The gauge contains antifreeze to melt snow, so the full amount from a heavy snowfall will be reported. Similarly to the NWS gauge, the BBMWD gauge may be impacted by nearby trees.

The BBCCSD maintains a precipitation gauge and weather station on the west side of its building. The current precipitation gauge replaced one that was located at a ground surface elevation of 6,773 feet msl and 34° 15.678' north latitude and 116° 50.661' west longitude. The new precipitation gauge, not accessible at the time of the evaluation, is approximately 30 feet northwest and at the same elevation as the previous precipitation gauge. The precipitation gauge is approximately 4 feet above the ground and has a windshield to mitigate the effects of air currents. The gauge is sufficiently far from overhead obstacles. The weather station sits about 10 feet south of the precipitation gauge.

The current precipitation gauge went into service in December 2007. The gauges were kept in service concurrently to determine the comparability of the data. The new gauge recorded greater amounts of precipitation for the same precipitation events (personal communication with Tim Moran, BBCCSD). The old gauge was maintained at 8 feet above ground surface atop the corner of a cinder block wall. Ground level air currents were likely deflected over the wall, and the upward air flow likely caused some of the precipitation to be deflected away from the gauge. This effect may have resulted in erroneously low precipitation measurements by the old gauge. Precipitation data date back to 1950.

BBCCSD also maintains an 8-inch precipitation gauge at Green Canyon Springs. The gauge is located at a ground surface elevation of 6,944 feet msl and 34° 14.141' north latitude and 116° 48.301' west longitude. The gauge is approximately 8 feet above ground and does not have a windshield to mitigate the effects of air currents. The gauge is free of overhead obstacles, but nearby small trees and a large water tank may mitigate the effects of wind. Precipitation data date back to 1963.

The CIMIS weather station (#199) is located in a field at the Bear Mountain Golf Course. The precipitation gauge is located at a ground surface elevation of 6,903 feet msl and 34° 14.257' north latitude and 116° 51.953' west longitude. The gauge uses a tipping bucket that records in hundredths of an inch. The gauge is approximately 5 feet above ground and does not have a windshield to mitigate the effects of air currents, but the chain link fence enclosure may act as a windshield. The gauge does not melt snow and may under-report during heavy snow events. There is a screen over the collection funnel for keeping objects from interfering with the gauge's tipping mechanism. Snow may build up on the screen, though it will be recorded as it melts. The weather station is located in an open field in the middle of the golf course. Precipitation data date back to 2005 and can be found on the CIMIS website (CIMIS, 2009).

3.4.2.3 Surface Drainage

Surface drainage within the Big Bear basin flows either toward Big Bear Lake or Baldwin Lake. The basin is divided into 11 subunits. Flow from 7 of the subunits is directed toward Big Bear Lake and flow from 4 of the subunits is directed toward Baldwin Lake. Flow occurs either as a result of runoff and snowmelt or from springs. Some of the streams are ephemeral and flow only after rainfall or as the snowpack is melting. The length of the snowmelt is variable and, therefore, so are the times when the streams have water. Baldwin Lake is an endpoint for surface drainage in the eastern part of the basin. Big Bear Lake flows into Bear Creek, which is tributary to the Santa Ana River.

3.4.2.4 Vegetation

The vegetation in the Big Bear basin is dominated by two general types of vegetation: (1) coniferous forests of pines and firs and (2) piñon and juniper woodlands. The coniferous forests dominate the western and southern areas of the basin and the southern part of the Van Dusen subunit. The piñon and juniper woodlands dominate the eastern and northern areas of the basin. The eastern and northern areas also contain regions of sagebrush and agricultural land used mostly for fodder crops.

3.4.2.5 Geology (Flint et al., 2007)

Big Bear basin is a geologically complex area on the north side of the San Bernardino Mountains. For the purposes of this report, the geological units are generalized into pre-Tertiary basement rocks, Tertiary sedimentary deposits, and Quaternary alluvial deposits.

- The basement rocks are dominated by (1) large Cretaceous granitic bodies ranging in composition from monzonite to gabbro, (2) metamorphosed sedimentary rocks ranging in age from late Paleozoic to late Proterozoic, and (3) Middle Proterozoic gneiss. These rocks are deformed by normal, reverse, and thrust faults, and in places are tightly folded. In general, the basement rocks are of low permeability and are not considered a major water-bearing unit except in fractures and weathered zones.

- The Tertiary sedimentary deposits overlie the basement rocks throughout most of the study area. This stratigraphic unit consists primarily of consolidated to partly consolidated alluvial fan deposits and probably yields only small quantities of water to wells. These deposits are predominantly exposed on the south side of the Baldwin Lake at the base of the mountains and are reported to be greater than 1,000 feet thick in some areas.

- Quaternary alluvial deposits overlie the Tertiary sedimentary deposits and basement rocks throughout much of the groundwater basin. Mapped quaternary alluvial deposits were generalized into older alluvium and recent alluvium:

  - The older alluvium consists predominantly of clay and sandy clay with interbedded layers of sand and gravel near Big Bear Lake and coarsens to predominantly sand with some gravel and interbedded layers of silt and clay toward Baldwin Lake. Beneath Baldwin Lake the alluvial deposits consist primarily of lacustrine clay deposits. The thickness of the clay deposits range from 0 feet at the margins of Baldwin Lake to more than 270 feet. The coarse-grained layers in the older alluvium are probably the major water-bearing units in the groundwater basin.

- Recent alluvium is present predominantly in the surface drainages in the watershed outside of the groundwater basin and in the shallow subsurface between Big Bear and Baldwin Lakes. The recent alluvium consists of permeable sand and gravel deposits with minor interbedded layers of silt and clay. Most of the recent alluvium is above the water table. Where present, the permeable recent alluvium allows for rapid infiltration of available rainfall and runoff.

3.4.3 Water Balance Modeling

3.4.3.1 Conceptual Model

Groundwater recharge in the Big Bear Valley basin occurs through various processes. As used herein, recharge is defined as water that reaches the regional water table within the Big Bear Valley basin and does not exit the basin through underflow. In places, the regional water table lies within the bedrock, based on water levels in deep wells and the locations of springs in the uplands. But in portions of the interior of the basin, the water table lies within the alluvium. The water table in the alluvium is seasonally variable and ranges from artesian to a few tens of feet below land surface.

Groundwater within the bedrock and alluvial aquifers in the Big Bear Valley basin is derived from recharge that originates from precipitation falling within the basin. Groundwater discharge from the basin occurs by flow in alluvium in the southeast part of the basin, specifically the Erwin subunit. Groundwater discharge out of the basin has been evaluated external to the water balance model for this project. Groundwater also discharges within the basin from springs. Flow from these springs either percolates back into the ground or enters Big Bear Lake as runoff. Groundwater flow that discharges from springs and enters Big Bear Lake as stream runoff has also been evaluated external to the water balance model for this project.

The recharge potential depends on a variety of factors, including the amount and duration of precipitation, surficial and bedrock geology, soil depth and permeability, land surface slope and orientation, and vegetation, among others. Although potential evapotranspiration may exceed precipitation on an annualized basis over a basin, during winter or early spring months or during

brief and intense summer storms, potential evapotranspiration can be less than precipitation; at this time, there is the greatest potential for recharge.

In the more mountainous terrain where bedrock is exposed, recharge occurs where infiltration of rain and snowmelt penetrates below the root zone. In these areas, where there is no soil and bare rock is exposed at the surface, vegetation is very sparse because there is little potential to store moisture during dry periods. Infiltrability is high where the bedrock is permeable, such as in carbonate outcrops, and is believed to be relatively lower in the volcanic rock outcrops, except where these rocks are highly fractured. Greater recharge will be expected in the alluvial fan heads at elevations where precipitation is greater than in the valley floor and where the soils are thick and have high permeability.

Precipitation that does not result in runoff infiltrates permeable surfaces, percolates through surficial soils, and is stored as soil moisture. A portion of this soil water is captured by vegetation and is lost by evapotranspiration; what is not captured migrates downward toward the water table. This areally distributed water that percolates below the plant root zone and ultimately reaches the water table is called diffuse natural recharge.

3.4.3.2 Model Domain

The model domain includes the Big Bear Valley basin and the components of the water balance. For the model, the basin is defined as the topographical area where surface water flows toward either Big Bear Lake or Baldwin Lake. The lakes are outside the model domain. Groundwater flow out of the basin and spring flow, defined as groundwater discharge, are estimated outside the model and subtracted from the model's recharge estimate.

3.4.3.3 Distributed Parameter Watershed Model Description

DBS&A has developed a DPWM similar in concept to water balance models used by the USGS in Nevada (e.g., Basin Characterization Model [BCM] [Flint and Flint, 2007], INFIL [Hevesi et al., 2003]). At the time of model development, the USGS had not yet made these codes publicly available, therefore necessitating the development of DBS&A's proprietary code. A detailed description of the DPWM is included in Appendix B.

The DPWM generally uses a daily time step over variable grid cell sizes that can range up to 8,100 square meters (m^2) (90 meters by 90 meters). Water balance components accounted for in the model include precipitation, bare soil evaporation, transpiration, runoff, run-on, snow accumulation, snowmelt, snow sublimation, soil-water storage, and net infiltration. Transpiration is the evaporation of water from plants, which replace the water by absorbing soil moisture. Snow sublimation is a process by which snow evaporates without melting first. A schematic of the water balance components for a model cell is shown in (Figure 3-2).

A bedrock boundary is placed at the bottom of cells with shallow soil depths that will restrict infiltration when the saturated hydraulic conductivity of the bedrock is less than that of the soil. Unlike the USGS BCM model, the DPWM accounts for the routing of runoff through the watershed. Runoff is the only means by which water can travel from one cell to another. Water that infiltrates the surface layer can drain to deeper layers (eventually becoming recharge), be lost by evapotranspiration, or added to soil water storage.

The model generally relies on the widely accepted FAO-56 method for computing actual evapotranspiration (AET) from the reference evapotranspiration (ET_0), which is estimated using the Penman-Monteith method (Allen et al., 1998). Evapotranspiration is the combination of evaporation and transpiration into one overall process that accounts for moisture extracted from soil. The FAO-56 method was developed by the Food and Agricultural Organization of the United Nations to provide a standardized and relatively simple method to accurately estimate crop water requirements under a wide range of climatic conditions. In 2005, the American Society of Civil Engineers (ASCE) reviewed several common methods for determining ET_0. The report concluded that the FAO-56 and ASCE standardized methods perform equally well and are the two recommended methods for calculating the ET_0 (Allen et al., 2005a). Temesgen et al. (2005) compared the FAO-56, ASCE standardized, and one other method at 37 agricultural weather sites in California and found good agreement among the methods for semiarid, arid, and humid locations.

The DPWM computes drainage using either the field capacity model or the van Genuchten-Mualem model (VGM). The field capacity model is commonly used (e.g., in the HELP, INFIL, BCM, and PRMS models) and computes drainage as the water in excess of the field capacity of

the soil. The VGM method uses the Mualem (1976) equation along with the van Genuchten parameters of fit to the water content-capillary pressure relationship. The VGM method is an intermediate approach between the simpler field capacity bucket models (e.g., in the HELP, INFIL, and BCM models) and the computationally intensive numerical solution of the Richards equation (Richards, 1931). The VGM method averages the water content over the cell thickness and assumes that with a unit downward gradient, drainage occurs at a rate equal to the corresponding unsaturated hydraulic conductivity.

The FAO-56 method (Allen et al., 1998) computes an ET_0 value using the Penman-Monteith equation that represents evapotranspiration from an extensive surface of green grass of uniform height, actively growing and adequately watered. The ET_0 is modified for any agricultural or natural vegetation type using crop coefficients (K_{cb}). A coefficient of 1.0 represents the reference grass vegetation. Coefficients less than 1 represent less dense vegetation, and coefficients greater than 1 represent more dense vegetation. The FAO-56 method supplies equations for computing K_{cb} for natural vegetation using site-specific climate data and a measure of the vegetation density (e.g., leaf area index [LAI]). Further adjustments to the K_{cb} provided by FAO-56 include stomatal resistance adjustments that account for the ability of desert vegetation to conserve water.

The Penman-Monteith evapotranspiration computation in FAO-56 includes adjustments for cloud cover when precipitation occurs and adjustments for sky clearness. The sky clearness coefficient was chosen as the standard value for locations where air masses are influenced by a nearby water body. The value chosen represents a conservative condition because it leads to a greater amount of evapotranspiration, which results in less water available for recharge. In addition, the solar radiation is modified for slope and aspect of the land surface.

3.4.3.4 Model Input Data

Data were collected and model parameters evaluated using, in order of preference: published site-specific data, site-specific literature sources, general literature sources, and estimates based on properties. Table 3-1 lists the sources of data for each input category to the water balance model.

3.4.3.5 Climate

Direct climate inputs to the DPWM are for one reference cell location and include daily total precipitation, maximum daily air temperature, minimum daily air temperature, average daily wind speed, and duration of precipitation events. For the daily total precipitation, maximum daily air temperature, and minimum daily air temperature inputs, DBS&A used the DAYMET climate model data provided online by the University of Montana (DAYMET, 2007). For the precipitation intensity and wind speed, DBS&A used observed hourly data collected at the Big Bear CIMIS station for water year 2006. The Big Bear CIMIS station went online in 2005, so 2006 water year is the earliest complete water year data providing wind speed data and hourly precipitation. The Big Bear CIMIS station is the only weather station in the basin that collects and publically posts these data, which is one of the reasons the Big Bear CIMIS station location was chosen as the reference cell location for the model. Another reason this station was chosen is that the station is centrally located between the eastern and western boundaries of the basin

DAYMET performs a spatial and temporal analysis of existing regional weather station data to estimate a 23-year record (water years 1981 through 2003) for any location in the U.S. DAYMET uses numerical methods to account for the typical heterogeneous distribution of stations in complex terrain. For the reference cell location, the DAYMET modeled precipitation, maximum daily air temperature, and minimum daily air temperature and these data were included in the climate data input file for the DPWM. The mean precipitation interpolated by DAYMET for the Big Bear CIMIS for water years 1981 through 2003 was 24.2 inches per year (in/yr).

Although DAYMET provides several data types, including solar radiation and relative humidity, only total daily precipitation and minimum and maximum air temperature data from DAYMET were used. Relative humidity data provided by DAYMET were not used because of a concern that the DAYMET model may not accurately predict relative humidity in semiarid environments. Instead, the relative humidity was calculated from the dewpoint temperature, which in semiarid environments is typically less than the minimum daily air temperature (Allen et al., 1998).

The other inputs to the reference cell were the wind speed and duration of precipitation events. The Big Bear CIMIS station is the only source of wind speed data, and the 2006 water year data

was the only complete water year data set, so it was applied to each year in the model. The duration of precipitation events is calculated from DAYMET's modeled daily precipitation and the mean hourly precipitation intensity calculated from CIMIS data for the 2006 water year. For each day in the model, the daily precipitation divided by the hourly precipitation intensity yields the duration in hours. If the calculated duration exceeded 24 hours, the 24-hour time frame was used in the model. Using the Big Bear CIMIS data, the mean hourly precipitation intensity was calculated by dividing the total precipitation by the total number of hours the precipitation was recorded.

The precipitation lapse rate is also a direct input into the DPWM. The precipitation lapse rate is a precipitation correction for elevation relative to the reference location precipitation. It is also computed from DAYMET model data. The precipitation lapse rate was estimated for the Bear Valley basin using DAYMET mean annual precipitation estimates from 11 locations within the basin (Figure 3-3), including the Big Bear CIMIS. The precipitation gauge sites at Big Bear Lake Dam, Big Bear Lake fire station, BBCCSD, and Green Canyon Springs were also included in the 11 locations. The other locations were chosen to provide a wide distribution of elevations and lateral coverage around the basin (Flint et al., 2007). For each location, the DAYMET precipitation model data were generated and analyzed to provide the mean annual precipitation. The lapse rate was estimated by performing a linear regression on the mean annual precipitation versus elevation for the 11 locations. The precipitation history from 1981 to 2003 for each cell in the model domain was then computed by applying the elevation lapse rate to the reference cell precipitation.

The temperature data were corrected for elevation using a temperature lapse rate estimated by the same method and locations used for the precipitation lapse rate. The lapse rates in DPWM were constant for all years and were based on average conditions over the interval 1981 to 2003.

3.4.3.6 Soils

Soils data were obtained from the U.S. Department of Agriculture (USDA) SSURGO database, which contains electronic soil type and characteristic data from field surveys conducted by the USDA in San Bernardino County, California, including the Big Bear Valley basin (Figure 3-4,

Table 3-2). In particular, the range of saturated hydraulic conductivity values for the upper surface soil layer was obtained from the SSURGO database (Figure 3-5).

Because actual site measurements are not available for the soil types mapped by the USDA, the water retention characteristics were estimated using the Rosetta pedotransfer model (Table 3-3) published by the U.S. Salinity Laboratory in Riverside, California (Schaap et al., 2001) with the soil texture data provided by the USDA.

The USDA reports depths to lithic (Figure 3-6) and paralithic (Figure 3-7) bedrock that is shallower than 5 feet. Paralithic bedrock is weathered and weakened bedrock, possibly fractured, that can be dug up with a shovel. Depths greater than 5 feet were assumed to be greater than the maximum rooting depth of a vegetation association.

3.4.3.7 Bedrock

The distribution of bedrock geology in the Big Bear Valley basin (Figure 3-8) was obtained from the geologic map published by Dibblee (1971). The geologic formation codes within the basin are defined in Table 3-4. The geologic units were consolidated into general hydrogeologic units that were expected to have similar hydraulic characteristics (e.g., gneiss, plutonic rocks and quartzite, metamorphic rocks and porphyry complex, quaternary clay deposits, and other quaternary deposits). The bedrock geologic units are summarized in Table 3-4, including saturated hydraulic conductivities and predominant mineral types (Brassington, 1988; Belcher et al., 2002, 2008a, Maidment, 1993; Stober and Bucher, 2007). Major faults are shown on Figure 3-8. None of the faults shown are associated with the 1992 Big Bear Earthquake. The Southern California Earthquake Data Center (2009) reported, “The Big Bear earthquake rupture did not break the surface; in fact, no surface trace of a fault with the proper orientation has been found in the area.”

3.4.3.8 Vegetation

There are two sources of vegetation data for the Big Bear Valley basin: (1) USGS Gap Analysis Project (GAP) (USGS, 2008a) codes and ecological systems for the distribution of vegetation types (Figure 3-9) and (2) literature sources (e.g., UCSB, 1998; Rundel and Gibson, 1996; Tirmenstein, 1999; Huxman et al., 1999; Huxman and Smith, 2001; Naumburg et al., 2003;

Hamerlynck et al., 2000a, 2000b, 2002, 2004; Pataki et al., 2000; Smith et al., 1995; Schenk and Jackson, 2002) describing the dominant vegetation properties including mean maximum plant height, rooting depth, and stomatal resistance. Table 3-5 summarizes the vegetation associations and corresponding GAP ecological descriptions.

3.4.3.8.1 Rooting Depths of Vegetation Associations. Table 3-6 summarizes the rooting depths assigned to vegetation associations in DPWM simulations (Rundel and Gibson, 1996; Schenk and Jackson, 2002). The rooting depth determines the thicknesses of Layers 1 and 2 unless bedrock is present at a shallower depth. It is assumed that rooting density in bedrock is not significant. Rooting depths range from 1 meter in the grassland vegetation association to 3.3 meters in the piñon-juniper and cottonwood vegetation associations (Schenk and Jackson, 2002).

3.4.3.8.2 Mean Leaf Stomatal Resistance. The FAO-56 method adjusts crop coefficients for native vegetation based on the leaf stomatal resistance (Allen et al., 1998). The leaf stomatal resistance of the reference vegetation in FAO-56 is 100 seconds per meter (s/m). Resistance values greater than 100 s/m will decrease transpiration, while values below 100 s/m will increase transpiration. Desert vegetation has adapted to use increased stomatal resistance to conserve water usage. Outside of the peak growth season (typically February through April in the Northern Mojave Desert), when precipitation is scarce or unpredictable, desert vegetation will have increased stomatal resistance and corresponding decreased transpiration.

Several literature sources (Huxman et al., 1999; Huxman and Smith, 2001; Hamerlynck et al., 2002, 2004; Naumburg et al., 2003; Pataki et al., 2000; Smith et al., 1995) were used to obtain values for leaf stomatal conductance, which is the inverse of stomatal resistance. Values were averaged for each species and for each growth stage and then averaged by vegetation group (annuals, drought-deciduous, and evergreen species). Sagebrush and chaparral vegetation associations in the Big Bear model were modeled after a mix of drought-deciduous and evergreen species, with mean leaf stomatal resistance values for the growth stages averaged for each vegetation association (Table 3-7). The grassland vegetation association used the stomatal resistance values for Bromus. In the non-Mojave Desert vegetation associations (piñon-juniper, scrub oak, pine and fir) stomatal resistance values were assumed to be at the

FAO-56 reference (100 s/m) (Allen et al., 1998) for all growth stages. Table 3-7 provides the parameter values used as input to the DPWM.

3.4.3.8.3 Mean Maximum Plant Height. Table 3-8 summarizes the mean maximum plant height for vegetation associations in the Big Bear Valley. The Big Bear Valley data were obtained from a variety of published sources (Rundel and Gibson, 1996; Tirmenstein, 1999; Holland, 1986). Plant heights range from 0.1 meter for grasses up to 60 meters for the conifers, although plant heights greater than 2 meters are treated as 2 meters in the FAO-56 method (Allen et al., 1998). .

3.4.3.8.4 Leaf Area Index and Phenology. LAI is the ratio of one-sided leaf area over the total land area. Values range from 0 for bare ground up to 15.5 for evergreen forests. However for any given suite of vegetation, the FAO-56 method sets transpiration at a maximum for LAI values greater than 3 (Allen et al., 1998, Equation 97). Monthly LAI values are summarized in Table 3-9 (DBS&A, 2008; Breuer and Frede, 2003).

3.4.3.9 General Model Parameters

Table 3-10 summarizes general model parameter inputs for the DPWM simulations. These include, for example, average latitude and elevation for the basin, initial saturation for cell nodes, evaporation parameters, and precipitation and temperature lapse rates.

3.4.3.10 Topography

Topography in the model was derived from USGS digital elevation models (DEMs) at a 90-meter grid scale. The mapped boundary of the basin defining the Big Bear Valley basin corresponds closely with the watershed boundary developed by the USGS, but differs along a few edges due to the 90-meter grid scale.

3.4.4 Description of Water Balance Model Methodology

To conduct the water balance, the watershed is divided into a grid of 21,004 cells, each with an area up to 87,188 square feet (2 acres). In each cell, the soil profile is divided into three layers with four nodes (Figure 3-2). The upper layer (Layer 1) has bare soil evaporation and

transpiration, and its thickness is based on the maximum depth of bare soil evaporation (evaporation layer depth [Z_e] in FAO-56 [Allen et al., 1998]). This layer was conservatively assumed to be present on bare rock mapped in the GAP study to allow for evaporation from surface pools of water.

Layer 1 is divided into two nodes (Nodes 1 and 2). Node 1 is the bare soil fraction of the cell where evaporation is dominant, and Node 2 is the fraction of the cell surface covered by vegetation canopy where transpiration is dominant. Bare soil evaporation does not occur in Node 2, but transpiration occurs to some degree in both Nodes 1 and 2. The areas of Nodes 1 and 2 are adjusted over the year as the vegetation grows, peaks, and then declines based on the basal transpiration coefficient (K_{cb}).

The second layer (Layer 2 and Node 3) is the remainder of the root zone for the vegetation type; its thickness is the maximum rooting depth minus the thickness of Layer 1. Transpiration is dominant in Layer 2, but some diffuse evaporation also occurs.

The final layer (Layer 3) is below the root zone and does not have any transpiration or evaporation. Its thickness is the depth to bedrock minus the thicknesses of Layers 1 and 2. In cells with deep alluvium, the thickness is limited to 20 meters minus the root layer thicknesses. Drainage from Layer 3 is limited by the bedrock saturated hydraulic conductivity when less than the soil saturated hydraulic conductivity. Drainage from Layer 3 is always computed using a field capacity approach because any water added to this layer is beyond the root zone and is net infiltration.

3.4.4.1 Precipitation

Precipitation is provided as input to the model for one reference location (CIMIS #199 Big Bear Lake) based on the spatial and temporal analysis described in Section 3.4.3.5, and is then estimated for all other cells in the model using the precipitation lapse rate. When precipitation occurs in a given day, it is assumed that it occurs over the entire modeled watershed domain. On some days precipitation occurs only in some parts of the basin, but these days tend to have low precipitation where it falls. The model uses climatic information from one reference cell and computes the precipitation for every other cell based on adjustments for elevation. The duration

of precipitation is estimated by linear regression through measurements of duration and total daily precipitation. Precipitation occurs as snow when the daily average temperature is below 32°F.

3.4.4.1.1 Adjustment for Elevation. Precipitation totals increase with increasing elevation. The DPWM takes the reference precipitation amount (at the reference elevation for the Big Bear CIMIS) and adjusts the amount of precipitation at the other model domain cell locations in the basin using an elevation lapse rate, as follows:

$$P_{cell} = P_{ref} * (1 + (elev_{cell} - elev_{ref}) * C_{precip}) \qquad \text{(Equation 3-1)}$$

where P_{cell} = the precipitation amount at a given cell

P_{ref} = the reference precipitation amount at the Big Bear CIMIS

$elev_{cell}$ = the elevation of the given cell from the 90-meter USGS DEM grid

$elev_{ref}$ = the reference elevation of the Big Bear CIMIS (6,963 ft msl, or 2,122 meters)

C_{precip} = the lapse rate (mm/(day∗m))

3.4.4.1.2 Duration of Precipitation Events. Each daily time step of the water balance model is divided into two steps when precipitation occurs: (1) the water balance for the duration of precipitation event and (2) the water balance for the remainder of the day. The duration of precipitation events for the 1981 through 2003 water years was estimated using the relationship between the total annual precipitation and the total duration of all precipitation events obtained from the measured 2006 water year to yield a mean precipitation intensity per hour. This relationship was used with the DAYMET estimates for daily precipitation from Big Bear CIMIS totals to calculate the estimated duration, in hours, for each precipitation event in the DAYMET input data. In general, a high mean precipitation intensity will yield shorter durations for large precipitation events. Shorter durations yield more runoff and less recharge, so a high mean precipitation intensity is conservative with respect to recharge. For some high daily precipitation events for the 1981 through 2003 water years, the duration was estimated to be greater than 24 hours, in which case the duration was set at 24 hours. In the case of a day with snowmelt, the duration of precipitation was assumed to be 12 hours. Except in the case of snowmelt,

duration was calculated outside of the DPWM code and was provided to the model as input with the climate file.

3.4.4.2 Snow

When the daily average temperature is below 32°F and precipitation occurs, the precipitation is assumed to occur as snow. The snow is stored as an equivalent depth of water in the model. In the time step with snow accumulation and in subsequent time steps, the snow sublimates at the snow sublimation rate provided as input to the model. The model uses a sublimation rate of 15 percent of the snowpack per day based on research in the Colorado Front Range (Hood et al., 1999).

When snowpack is present and the average daily temperature is above 32°F, the snowmelts at the snowmelt rate specified in the input files. The rate of snowmelt averages 2.6 millimeters per day per degree Celsius (mm/d/ºC) based on the average melt rate from December 1 through April 1 estimated using the methodology in the HELP model (Schroeder et al., 1994). As noted in Section 3.4.4.1.2, if there is snowmelt, the duration is set to 12 hours.

3.4.4.3 Water Transport and Storage

The DPWM has the ability to use either a field capacity balance approach (similar to the HELP model, INFIL, and BCM) or a VGM approach to compute drainage from Layer 1 to Layer 2 and from Layer 2 to Layer 3; the field capacity approach is always used for Layer 3. The field capacity approach, also referred to as a bucket model, allows instant drainage of all water that exceeds the field capacity of the soil (estimated by the soil water content at ⅓ bar). The VGM approach calculates the unsaturated hydraulic conductivity associated with average water content of the model layer and, assuming a downward unit gradient, allows the layer to drain at that rate. The VGM more closely simulates realistic soil drainage rates than does the field capacity approach. In the VGM approach, water drains more slowly after a precipitation event than in the bucket model but, unlike the bucket model, will continue to drain at decreasing rates until the water content reaches the residual water content.

In both water balance models water is routed downward at a rate that is limited by the hydraulic conductivity of the layer or, where the layer is overlying bedrock, by the saturated hydraulic

conductivity of the bedrock. Once net infiltration has been computed from the bottom layer, any water that exceeds the maximum holding capacity of that layer is retained in the overlying layer. If water in the uppermost layer exceeds the maximum holding capacity, then the water is passed to the downstream cell as runoff.

Water is accounted for in the cell nodes as an equivalent depth of water in the node per unit area, termed "water level." Water level is related to the average volumetric water content (θ) in a node, as follows:

$$\text{Water level (mm)} = \theta\ (m^3/m^3) * \text{node thickness (mm)} \qquad \text{(Equation 3-2)}$$

Similarly, the maximum water level, (θ_{ws}), field capacity, (θ_{fc}), water level, and wilting point, (θ_{wp}) water levels are computed as follows:

$$\text{Field capacity water level (mm)} = \theta_{fc}\ (m^3/m^3) * \text{node thickness (mm)} \qquad \text{(Equation 3-3)}$$

$$\text{Maximum water level (mm)} = \theta_{ws}\ (m^3/m^3) * \text{node thickness (mm)} \qquad \text{(Equation 3-4)}$$

$$\text{Wilting point water level (mm)} = \theta_{wp}\ (m^3/m^3) * \text{node thickness (mm)} \qquad \text{(Equation 3-5)}$$

The field capacity and wilting point water contents are computed from the capillary pressure head provided by the user as model input using the van Genuchten parameters. The model used a field capacity of –⅓ bar and a wilting point of –60 bars. The wilting point is much greater than the typical –15-bar limit found for agricultural vegetation because arid and semiarid vegetation can extract water at much greater capillary pressures. This approach is conservative in that the amount of water retained as stored soil water is increased, thereby reducing the amount of recharge.

3.4.4.4 Runoff and Run-on

Runoff occurs in a cell when the water content in the upper model layer (Layer 1) exceeds saturation or the rate of water application over the storm fraction of the day exceeds the saturated hydraulic conductivity of the soil or rock. Runoff is routed to the lowest adjacent grid

cell elevation. A cell can receive run-on from up to seven adjacent cells, but provides runoff to only one downstream cell.

3.4.4.5 Evapotranspiration

Evapotranspiration (ET) is estimated using the FAO-56 Penman-Monteith method (Allen et al., 1998). Evapotranspiration is computed after the computation of infiltration and runoff. The actual total evapotranspiration for a cell on a given day is calculated as follows:

$$ET = (K_e + K_s \cdot K_{cb})\, ET_0 \qquad \text{(Equation 3-6)}$$

where K_e = the soil evaporation coefficient

K_s = the water stress coefficient

K_{cb} = the basal transpiration coefficient

ET_0 = the reference evapotranspiration based on climate parameters

3.4.4.5.1 Reference Evapotranspiration. The reference evapotranspiration (ET_0) represents the evapotranspiration from a reference crop (turf grass) without any water limitations.

Adjustment of Air Temperature for Elevation. The temperature lapse rate was calculated from the same 11 locations used to obtain the precipitation lapse rate described in Section 3.4.3.4. A linear regression was run for the temperatures for each location. The lapse rate was estimated as 0.0094 degree Celsius per meter (°C/m) based on 1981 through 2003 DAYMET annual temperature normals.

Adjustment of Solar Radiation for Sky Clearness. The FAO-56 method has a clearness index (Krs) that can adjust the solar radiation for dust or particulates in the air (Allen et al., 1998). The value of Krs used in the model was 0.19, which represents air masses being influenced by a large body of water.

Adjustment of Solar Radiation for Sun Angle, Turbidity, Atmospheric Thickness, and Precipitable Water. The FAO-56 method provides adjustments for solar radiation based on the

average angle of the sun for the time of year, turbidity in the atmosphere, atmospheric thickness, and precipitable water:

- The average sun angle for the day of the year was computed using Equation D-5 in Allen et al. (2005b).

- The adjustment for turbidity ranges from 0.5 or less for extremely turbid, dusty, or polluted air up to 1.0 for clean air. As recommended by Allen et al. (2005b) for routine prediction of solar radiation and based on the calibrated sky clearness index, a value of 1.0 was used for the turbidity coefficient.

- The adjustment for atmospheric thickness is based simply on the atmospheric pressure at the cell estimated from elevation (Allen et al., 2005b, Equation D-3).

- The amount of precipitable water in the atmosphere was estimated based on the vapor pressure and cell elevation (Allen et al., 2005b, Equation D-3).

These adjustments of solar radiation for precipitable water, sun angle, and atmospheric thickness and turbidity are discussed in detail in Appendix B.

Adjustment of Solar Radiation for Slope and Azimuth. The author of the FAO-56 method, Dr. Richard Allen, along with another collaborator, Dr. Ricardo Trezza, developed an unpublished method to adjust the daily solar radiation for slope and azimuth (Allen and Trezza, 2006) that follows the methods published by Reindel et al. (1990) and Duffie and Beckman (1980, 1991). All equations used in the slope and azimuth adjustment were obtained from peer-reviewed publications. The adjustment uses the average latitude for the basin, which is acceptable for basins with less than 40 miles between the northernmost and southernmost locations within the basin (Allen and Trezza, 2006).

Surface Albedo. Surface albedo is the reflectance of the ground and ranges from 0.1 for dark wet soil, to 0.23 for dense green vegetation, and up to 0.6 and 0.9 for snow cover. The DPWM conservatively uses 0.23 for the surface albedo across the basin except in the case of snow

cover, where the albedo is adjusted to 0.60. The value of 0.23 is less than expected for the light-colored dry soils with sparse vegetation cover in parts of the Big Bear Valley. The adjustment for snow cover to 0.60 follows the methodology in the HELP model.

Actual Evapotranspiration. AET is computed from the reference evapotranspiration based on the relative fractions of bare soil evaporation and transpiration. Without any water limitations, the actual evapotranspiration will be equivalent to crop evapotranspiration (ET_c), as follows (Allen et al., 1998, Equation 69):

$$ET_c = (K_{cb} + K_e)\, ET_0 \qquad \text{(Equation 3-7)}$$

where ET_c = the crop evapotranspiration

K_{cb} = the basal crop coefficient (e.g., transpiration component of ET_c)

K_e = the surface evaporation coefficient

ET_0 = the reference evapotranspiration based on climate parameters

The sum of K_{cb} and K_e is the crop coefficient K_c and can never exceed a maximum value ($K_c$max) based on the available energy from the sun. As water is depleted from the root zone, the values of K_e and K_{cb} are reduced using the evaporation reduction coefficient (K_r) and the water stress coefficient (K_s), respectively. The relative proportion of surface evaporation is also controlled by the fraction of ground covered by the vegetation canopy. Surface evaporation is only dominant in the uncovered or bare soil fraction (Node 1 of Layer 1 in the model). The basal crop coefficient includes a small fraction of diffuse evaporation from beneath the vegetation canopy but is dominated by the transpiration. Transpiration occurs from Node 2 of Layer 1 and all of Layer 2 (Node 3) in the model.

Evaporation Reduction Coefficient (Kr). As the depth of water stored in the soil is depleted below field capacity (termed the depletion level [De]), the rate of evaporation and transpiration can decrease from the maximum. Several variables are computed in the FAO-56 method based on field capacity, wilting point, and soil texture (Allen et al., 1998). Evaporation is computed only for Node 1 in Layer 1.

Total evaporable water (TEW) is the total amount of water available for evaporation and is computed in the FAO-56 method as follows:

$$TEW = (\theta_{fc} - (0.5)\theta_{wp})\ Z_e \qquad \text{(Equation 3-8)}$$

where θ_{fc} = the field capacity

θ_{wp} = the wilting point

Z_e = the thickness of layer (typical range is 0.1 to 0.15 meter)

The term $(0.5)\theta_{wp}$ implies that the water content can be reduced with evaporation below a level that can be removed by transpiration. The Z_e was conservatively set to the upper end of the range recommended in FAO-56 (0.15 meter); the greater depth of the evaporative layer allows for more water to be depleted by evaporation.

The amount of water that is readily evaporable (REW) is determined from soil texture (Allen et al., 1998, Table 19) and ranges from 2 to 7 millimeters (mm) for sand up to 8 to 12 mm for clay. A value of 8 mm was conservatively selected for the model (the upper end of the range for sand and loamy sands found in the basin).

TEW and REW are used to determine the reduction in evaporation rate (K_r, ranges from 0 to 1) (Allen et al., 1998, p. 145), as follows:

$$K_r = \frac{TEW - D_e}{TEW - REW} \qquad \text{(Equation 3-9)}$$

Water Stress Coefficient (Ks). The water stress coefficient is based on the total amount of water available for transpiration (TAW), defined in FAO-56 (Allen et al., 1998) as follows:

$$TAW\ (mm) = 1{,}000\ mm/m\ (\theta_{fc} - \theta_{wp})\ Z_r\ (m) \qquad \text{(Equation 3-10)}$$

where θ_{fc} = the field capacity water content

θ_{wp} = the wilting point water content

Z_r = the mean maximum rooting depth

At first, as the water level is reduced below field capacity, no water stress reduction in transpiration occurs (e.g., $K_s = 1$). This fraction of TAW that is readily available for transpiration (RAW) is computed as follows:

$$\text{RAW (mm)} = p \cdot * \text{ TAW (mm)} \qquad \text{(Equation 3-11)}$$

where p = the average fraction of TAW that can be depleted from the root zone before moisture stress occurs (ranges from 0 to 1) (Allen et al., 1998, p. 162).

This depletion factor (p) depends on the evapotranspiration rate and the rooting depth of the plants. Where daily evapotranspiration is high (> 8 millimeters per day [mm/d]) and plant roots are shallow, p is typically 0.3. Where daily evapotranspiration is low (< 3 mm/d) and plant roots are deep, p is typically 0.7. Typically a value of 0.5 is chosen (Allen et al., 1998). A conservative p value of 0.7 was chosen to allow more transpiration (resulting in less recharge).

The transpiration reduction factor, or water stress coefficient (K_s), is computed from TAW, root zone depletion (D_r = field capacity water level – water level), and RAW, as follows:

$$K_s = \frac{\text{TAW} - D_r}{\text{TAW} - \text{RAW}} \qquad \text{(Equation 3-12)}$$

Canopy Coefficients. As plant growth increases, the fraction of ground covered by the plant canopy (f_c) increases. Bare soil evaporation does not take place under the canopy, leaving more water for transpiration. As plants begin to decline and enter a dormant phase, the plant canopy decreases. This change is modeled using the equation recommended by the FAO-56 method (Allen et al., 1998, Equation 76), as follows:

$$f_c = \left(\frac{K_{cb} - K_c\text{min}}{K_c\text{max} - K_c\text{min}} \right)^{\left(1 + \frac{1}{2} h_{plant}\right)} \qquad \text{(Equation 3-13)}$$

where K_{cb} = the basal crop coefficient

$K_c$min = the minimum crop coefficient

$K_c$max = the maximum crop coefficient based on available energy

h_{plant} = the mean maximum plant height

The canopy coefficient defines the relative fractions of Nodes 1 and 2 in each cell. As f_c changes and the sizes of Nodes 1 and 2 change, water is appropriately transferred between Nodes 1 and 2 to maintain the water balance.

3.4.5 DPWM Results

The DPWM results for the Big Bear Valley basin and individual subunits are provided in Table 3-11. The DAYMET model predicts the basin-wide precipitation, which is used as input to the DPWM. The spatial distribution of mean annual precipitation for the basin is shown in Figure 3-3. The mean (average) precipitation is 91,196 ac-ft/yr and the median is 78,771 ac-ft/yr (Figure 3-10).

The model water balance has five components that account for all the water that falls as precipitation: actual evapotranspiration, recharge, net runoff, sublimation, and change in storage (water content of the soil above the water table). Actual evapotranspiration accounts for the highest water use. The mean (average) annual actual evapotranspiration of 50,389 ac-ft/yr accounts for 55 percent of the precipitation and is generally greatest along the base of the valley (Figure 3-11). Recharge has the second largest consumption of water, followed by net runoff, sublimation, and change in storage.

The mean (average) recharge to the Big Bear Valley basin computed by the water balance model is 24,894 ac-ft/yr with a median (midpoint) annual recharge of 17,322 ac-ft/yr (Figure 3-12). As shown in Figure 3-12, where the data are plotted as a cumulative distribution function, the median recharge value is the amount of recharge that can be expected to occur 50 percent of the time, based on historical data used as input to the model. The mean (average) recharge is 40 percent higher than the median (midpoint) due to a few years with very high recharge. Because the mean (average) recharge is functionally weighted by the magnitude of recharge in any given year, a few years of very high recharge will yield a mean (average) that is skewed to the higher end of recharge values. The median (midpoint) recharge

is not weighted by individual values in the data set. The median merely locates the midpoint value in the dataset; half of the values fall above the median (no matter how high those values may be), and half fall below. Thus, the median (midpoint) may be a more useful statistic in terms of anticipating the amount of precipitation in any given year and, thus, recharge that will be available to water managers. The spatial distribution of recharge in the basin is shown in Figure 3-13.

The recharge as a percentage of precipitation (recharge / precipitation x 100) was plotted for the years 1981 through 2003 (Figure 3-14). Figure 3-14 shows that the recharge is about 25 percent of the precipitation nearly 50 percent of the time. The mean (average) annual recharge accounts for over 27 percent of the precipitation in years that the precipitation is average or greater. This observation is consistent with the rule of thumb proposed by Maxey and Eakin, who concluded that approximately 25 percent of the precipitation will recharge the groundwater when the precipitation exceeds 20 in/yr (Maxey and Eakin, 1949).

Between 1999 and 2002 the basin had three of the four driest years in the model domain. Plants require a certain minimum amount of moisture to survive and, when the precipitation is low, will extract the maximum amount of water they can. Evaporation also tends to be higher when precipitation is low. Accordingly, evapotranspiration accounted for between 76 and 92 percent of precipitation in those three dry years (1999, 2001, and 2002), corresponding to a lower percentage of precipitation resulting in recharge during those years.

Recharge to the basin is spatially variable and is the highest at the heads of the alluvial fan deposits and along water courses (Figure 3-13). On the south side of the valley, there is a line of high recharge running from east to west where alluvial deposits abut the bases of the mountains.

Adjustments to the recharge that reduces the recharge estimates are (1) spring discharge that flows into the lakes and (2) groundwater underflow outside of the basin to the east.

The DPWM accounts for runoff from precipitation, but does not account for spring discharge that flows into the lakes, which are outside the model domain. The California Regional Water

Quality Control Board (CRWQCB), Santa Ana Region estimated surface flow into Big Bear Lake for a nutrient loading report (CRWQCB, 2005). The total surface flow into Big Bear Lake for 1990 through 2003 was estimated to be 14,032 ac-ft/yr with a median of 7,797 ac-ft/yr. The difference between this CRWQCB estimate and the DPWM-estimated runoff into Big Bear Lake over the same time period (mean of 6,970 ac-ft/yr, median of 4,813 ac-ft/yr) provides an estimated total of spring flow runoff into Big Bear Lake ranging from 2,984 to 7,062 ac-ft/yr, based on use of the medians and means, respectively.

Baldwin Lake, which is outside the model domain, does not have a known flow path outside the basin; therefore, runoff into Baldwin Lake can potentially contribute groundwater recharge to the basin even though in this approach the two major lakes are treated as being outside of the modeled domain. The extent of this Baldwin Lake contribution is unknown, but the DPWM mean annual runoff from the East Baldwin, Erwin, Van Dusen, and West Baldwin subunits in the Baldwin Lake watershed is 2,444 ac-ft/yr. Recharge that occurs directly beneath Baldwin Lake is conservatively not included in the estimate of the total annual recharge to the basin.

Based on a review of the alluvial geology, topography, creek flow, and spring flow, coupled with data from the Big Bear Valley recharge model, DBS&A estimated a range of potential losses from the groundwater basin to Arrestre Creek to the east. This flow would occur where alluvium from Gocke Valley crops out in the canyon wall to the east. Several seeps and springs are present in that area, and flow in Arrestre Creek increases significantly as the creek traverses the area east of the study area. As discussed in Section 3.2, waters discharging into Arrestre Creek would, at least in part, be sourced by Bear Valley watershed recharge. Potentially lost recharge occurs in an area of less than 1 square mile of the study area.

Losses from groundwater underflow occur on the eastern side of the basin in the Erwin subunit. Recharge to the Gocke Valley alluvium and bedrock to the south is potentially susceptible to outflow from the basin as springs to the east. The range of recharge potentially lost is estimated to be between 313 and 1,340 ac-ft/yr, likely on the lower end due to surface flow and pumped groundwater that would remain in the system. Recharge that is tributary to the portion of the basin overlain by Deadman's Lake may also be susceptible to this outflow, as it is near a surface water divide that induces flow either east or west of Deadman's Ridge. However,

several wells extract groundwater from the alluvium in this area, and actual outflow would likely remain within the 313- to 1,340-ac-ft/yr range.

The total estimated recharge to the Big Bear Valley basin, after adjusting for spring flow runoff to Big Bear Lake and groundwater underflow losses from the Erwin subunit, ranges from 16,531 to 21,534 ac-ft/yr. The 16,531 ac-ft/yr is calculated by subtracting the upper estimates of both discharge adjustments (i.e., outflow from the basin on the east and spring-fed flow lost to the lakes) from the DPWM estimate of 24,894 ac-ft/yr. The 21,534 ac-ft/yr is calculated by subtracting the lower discharge estimates of both adjustments from the DPWM estimate of 24,894 ac-ft/yr. These are mean (average) estimates, and can be adjusted for wetter or drier years. As an example, if the model estimate for a wet year is 10 percent greater than the mean estimate, then the adjusted ranges will also increase by 10 percent.

3.4.6 Discussion

DBS&A used the DPWM to predict recharge throughout the basin. The DPWM mean annual precipitation for each of the modeled cells within the basin was calculated using the DAYMET data for the reference location and the lapse rate calculated for the basin. The DPWM mean annual precipitation was compared to the measured precipitation at the NWS stations: Bear Valley Dam and Big Bear Lake (at the fire station). Gaps in the Bear Valley Dam data were filled in with dam precipitation data from the San Bernardino County rain gauge dataset. Gaps in the Big Bear Lake data were filled in with data from the Big Bear Hospital, also monitored by San Bernardino County. The Big Bear Hospital is approximately 1.5 miles east of the Big Bear Lake station.

The measured annual precipitation at the Bear Valley Dam is higher than that at the Big Bear Lake station, with the DPWM basin-wide mean annual precipitation falling between the two for 19 of the 23 years of record (Figure 3-15). Thus the DPWM precipitation results are consistent with the data from the two stations and adequately represent the precipitation within the basin.

DBS&A's adjusted estimate of mean total recharge, 16,531 to 21,534 ac-ft/yr, is similar to the USGS-adjusted (to include potential recharge of runoff) BCM estimated recharge of

15,796 ac-ft/yr but higher than other estimates (Table 3-12). The adjusted estimate of mean total recharge using the DPWM is twice that of the USGS INFILv3 model of 9,571 ac-ft/yr and four times that of Geoscience's maximum perennial yield estimate of 4,800 ac-ft/yr to 5,625 ac-ft/yr.

The cumulative distribution functions (CDFs) for the modeled precipitation and recharge (Figures 3-10 and 3-12) show similar behavior, with recharge increasing significantly in the same years as precipitation does. The recharge CDF (Figure 3-12) can be used to predict the probabilities that the recharge will be at least a certain amount during any year in the future. Based on the DPWM results, there is a 50 percent probability that the recharge will be 17,000 ac-ft/yr or greater. For years with higher than average precipitation, the 75 percent quartile shows that there is a 25 percent probability that the recharge will be 27,000 ac-ft/yr or greater. Of importance for dry years, the 25 percent quartile shows that there is a 75 percent probability that the recharge will be at least 11,000 ac-ft/yr or greater based on model estimates of recharge. From another perspective, the modeled annual recharge was greater than 11,000 ac-ft/yr for more than 75 percent of the modeled years. These probabilistic predictions can be used as a risk management tool attendant to precipitation variability.

The total estimated recharge, ranging from 16,531 to 21,534 ac-ft/yr, is based on adjustments to the DPWM mean recharge, 24,894 ac-ft/yr. If the estimated annual recharge for a future year differs from the mean annual estimated recharge, then the adjustments will also change. The simplest approach to estimating adjustments for spring flow runoff and groundwater underflow is that they change in direct proportion to the change in estimated annual recharge relative to the DPWM mean annual estimated recharge. As an example, if the model estimate for a wet year is 10 percent greater than the mean estimate, then the adjustments will also increase by 10 percent. Further analysis of the spring flow runoff and groundwater underflow to improve the estimates, by acquiring data and correlating those data to precipitation and the modeled estimated recharge, is recommended.

The modeled recharge as a percentage of DAYMET-modeled precipitation (recharge ÷ precipitation x 100) was plotted in order of increasing precipitation (Figure 3-16). The best empirical fit for the data is a logarithmic function that had a correlation coefficient ($R^2 = 0.8541$

[R^2 = 1.00 represents a perfect fit]). The modeled recharge, expressed as a percentage of DAYMET-modeled precipitation, is represented by the following mathematical relationship (all units in ac-ft):

$$(\text{Recharge/Precipitation})\ \% = 17.83\ \text{Ln}(\text{Precipitation}) - 178.86 \qquad \text{(Equation 3-14)}$$

If a similar relationship can be developed to predict recharge from measured gauge precipitation data, then a reasonable prediction of recharge is possible for any given year, assuming that historical data are accurate and that historical climate conditions remain a reasonable predictor of future behavior. DBS&A recommends that BBCCSD develop these mathematical relationships as watershed management and water planning tools.

3.4.7 Summary and Conclusions

DBS&A has analyzed total basin recharge with the DPWM:

- The adjusted estimate of median (midpoint) total recharge to groundwater within the Big Bear Valley basin is at least 8,959 ac-ft/yr and could exceed 13,962 ac-ft/yr.

- The adjusted estimate of mean (average) total recharge to groundwater within the Big Bear Valley basin is at least 16,531 ac-ft/yr and could exceed 21,534 ac-ft/yr.

DBS&A's estimate of recharge is in general agreement with the adjusted USGS BCM recharge estimate and significantly higher than an estimate using INFILv3. DBS&A's estimate of recharge is also significantly greater than Geoscience's (2007) estimate of maximum perennial yield.

The estimated recharge is a strong function of precipitation. A mathematical relationship between modeled recharge (as a percentage of precipitation) and DAYMET-modeled precipitation for the Big Bear Valley groundwater basin was established using historical regional precipitation data. The DPWM was also used to establish the locations of maximal recharge. The estimated recharge is greatest at the alluvial fan heads and along water courses. This

analysis provides a substantive initial basis for determining the optimal locations to protect and exploit groundwater resources in the future.

DBS&A makes the following recommendations for future investigations to enhance recharge estimates for the basin.

- Develop mathematical correlations between annual rain gauge data and the estimated total recharge.

- Develop a longitudinal lapse rate for inclusion into the model that accounts for increasing precipitation from east to west.

- Establish soil moisture monitoring stations in the basin to verify the model. The locations where the model estimates the recharge to be the greatest would be optimal for soil moisture monitoring.

- Establish snowfall monitoring at high elevations to refine the model. Locations within the Erwin subunit are likely candidates for snowfall monitoring.

- Establish streamflow monitoring stations to measure runoff in order to refine adjustments to the model results. Multiple stations on individual streams can be used to estimate the amount of infiltration that occurs in stream channels.

- Develop basin-wide estimates of evapotranspiration based on site-specific data and satellite imagery. Evapotranspiration data can be used to refine the model.

4. Existing Water Supply System

BBCCSD delivers pure mountain water to its customers through more than 5,990 service connections. The service area is divided into six pressure zones. The Rowe Pressure Zone is the largest, accounting for approximately 83 percent of the total demand. The five smaller pressure zones account for the remaining 17 percent. Major facilities include four reservoirs with a total capacity of 6.24 million gallons, 81.7 miles of pipeline ranging from 2 to 20 inches in diameter (in accordance with the water model described in Section 5), 13 vertical wells (Wells 5 and 7A are inactive), 2 slant wells, 2 springs, 6 booster stations, a fluoride blending system, an iron and manganese treatment facility, 426 fire hydrants, and more than 1,600 gate valves. Figure 4-1 shows the locations of the existing mainline pipeline system, the primary facilities, and the extents of the pressure zones.

The fluoride blending system, also called the Restricted Main, employs a significant network of pipelines (indicated in red on Figure 4-1). The network includes more than 20,000 linear feet of pipeline that conveys the production from 9 vertical wells to the Rowe Reservoir for fluoride blending. The system is isolated from the distribution network of the Rowe Pressure Zone and maintained under low pressure. Further information regarding the purpose and functionality of the fluoride blending system is provided in Section 4.8.

4.1 Distribution System

Over the past 25 years, BBCCSD production has averaged about 1,115 ac-ft/yr, with an average annual increase of only 0.3 percent. The number of connections has slowly but steadily increased from 4,373 connections in 1982 to 5,990 connections in 2007, for an average annual increase of 1.3 percent. In 2007, BBCCSD distributed 1,237 acre-feet to its customers at an average of 0.21 acre-foot per connection. Figure 4-2 shows the historical relationship between production and connections.

BBCCSD maintains 81.7 miles of pipeline ranging from 2 inches to 20 inches in diameter according to the current water model. The existing system is generally able to supply average

and maximum daily demands to existing customers, but is deficient in supplying the required fire flows in some areas.

4.1.1 Pressure Zones

A pressure zone is a sub-system of a larger distribution network designed to deliver water within a specified pressure range. A typical pressure zone consists of water storage at a higher elevation that feeds customers' connections at lower elevations through a system of pipelines. Source water is pumped to the storage tank and then flows by gravity through the system. Source water generally enters the system in zones with lower elevations and must be pumped to pressure zones with higher elevations as dictated by demand.

There are six pressure zones within the BBCCSD Service Area. The Rowe Pressure Zone fits the above description, featuring multiple sources and multiple elevated storage facilities. However, the other five pressure zones feature artificial hydraulic gradients. Such pressure zones have no elevated storage to provide pressure; rather, pressure is maintained mechanically using booster pumps instead of gravity to supply water to those sub-systems.

The Rowe Pressure Zone is the largest, containing 80 percent of all pipelines and supplying 83 percent of the total demand. Groundwater enters the Rowe Pressure Zone directly via Wells 2 and 9 and indirectly via (1) the Van Dusen slant wells, (2) the Greenspot Reservoir, which receives groundwater from Greenspot and Fish Hatchery Springs, and (3) the Rowe Booster Station, which pumps blended groundwater from the Rowe Reservoir. The Rowe Pressure Zone then feeds the other reservoirs and booster stations to supply the remaining smaller pressure zones.

There are no pressure relief valves to help maintain pressure within the smaller pressure zones. The pressure zone boundaries are defined by booster pump stations and closed gate valves.

Pressure zone changes since the previous 1991 Water Master Plan are as follows:

- The Greenspot Pressure Zone previously consisted of two sub-areas, which is no longer the case.
- The Gold Mountain and Marlowe Pressure Zones no longer exist.
- The Abbott Pressure Zone and Restricted Main Fluoride Blending Project have been added.
- The pressure in the main system is governed by the water level of the reservoirs.
- Water from the Rowe Reservoir is pumped to the other reservoirs via the Rowe Booster Station.

A description of each pressure zone is provided in Sections 4.1.1.1 through 4.1.1.6. Average annual demand is based on analysis of 2007 consumption data. The linear footage of pipelines and elevation range of each zone is based on an analysis of the water model database. The water model does not necessarily include small-diameter pipes such as laterals and dead-ends feeding cul-de-sacs. The static pressure range is calculated as the difference between the pressure head in each pressure zone according to the SCADA screenshot (Section 4.7) and the high and low elevations within each zone.

4.1.1.1 Rowe Pressure Zone and Restricted Main

- Hydraulic grade line (HGL): 6,960 feet
- Area: ±1,812 acres
- Linear feet of pipe in water model: 344,916
- Average annual demand: 1,027 acre-feet
- Elevation range: 6,708 to 6,920 feet
- System pressure range: 19 to 110 psi
- Equipment: Fluoride Blending Project (Restricted Main)
- Storage: 2.5-million-gallon Rowe Reservoir
- Source: Rowe Reservoir receives flow from:

  - Well 1
  - Well 1B
  - Well 3 (artesian)
  - Well 3A (artesian)
  - Well 4
  - Well 4A
  - Well 5 (inactive)
  - Well 6
  - Well 8
  - Well 10

- Source: Rowe Pressure Zone receives flow from:
  - BBLDWP Division Drive Intertie (inactive)
  - BBLDWP Erwin Ranch Road Intertie (inactive)
  - Well 2
  - Well 7A (inactive)
  - Well 9
  - Van Dusen 1 slant well
  - Van Dusen 2 slant well

- Booster stations: Rowe boosters pump water from Rowe Reservoir into Rowe Pressure Zone and into Hicks, Peery, and Greenspot Reservoirs.

4.1.1.2 Hicks Pressure Zone

- HGL: 7,055 feet
- Area: ±62 acres
- Linear feet of pipe in water model: 10,340
- Average annual demand: 32 acre-feet
- Elevation range: 6,840 to 6,960 feet
- System pressure range: 41 to 93 psi

- Storage: 1.14-million-gallon Hicks Reservoir
- Source: Hicks Reservoir receives flow from the Rowe Pressure Zone.
- Booster stations: Hicks boosters pump water from Hicks Reservoir to Hicks Pressure Zone.

4.1.1.3 Tanglewood Pressure Zone

- HGL: 7,030 feet
- Area: ±62 acres
- Linear feet of pipe in water model: 3,863
- Average annual demand: 4 acre-feet
- Elevation range: 6,800 to 6,920 feet
- System pressure range: 48 to 100 psi
- Storage: None
- Source: Tanglewood Pressure Zone receives flow from Rowe Pressure Zone.
- Booster stations: Tanglewood boosters pump water from Rowe Pressure Zone to Tanglewood Pressure Zone.

4.1.1.4 Abbot Pressure Zone

- HGL: 7,000 feet
- Area: ±41 acres
- Linear feet of pipe in water model: 10,644
- Average annual demand: 15 acre-feet
- Elevation range: 6,828 to 6,920 feet
- System pressure range: 35 to 75 psi
- Storage: None

- Source: Abbott Pressure Zone receives flow from Rowe Pressure Zone.
- Booster stations: Abbott boosters pump water from Rowe Pressure Zone to Abbott Pressure Zone.

4.1.1.5 Peery Pressure Zone

- HGL: 7,075 feet
- Area: ±306 acres
- Linear feet of pipe in water model: 29,191
- Average annual demand: 82 acre-feet
- Elevation range: 6,807 to 6,990 feet
- System pressure range: 34 to 113 psi
- Storage: 1.6-million-gallon Peery Reservoir
- Source: Peery Reservoir receives flow from Rowe Pressure Zone.
- Booster stations: Peery boosters pump water from Peery Reservoir to Peery Pressure Zone.

4.1.1.6 Greenspot Pressure Zone

- HGL: 7,075 feet
- Area: ±289 acres
- Linear feet of pipe in water model: 31,842
- Average annual demand: 78 acre-feet
- Elevation range: 6,833 to 7,000 feet
- System pressure range: 34 to 106 psi
- Storage: 1.0-million-gallon Greenspot Reservoir
- Source: Greenspot Reservoir receives flow from:
  - Greenspot Spring

- Fish Hatchery Spring
- Rowe Pressure Zone

- Booster stations: Greenspot boosters pump water from Greenspot Reservoir to Greenspot Pressure Zone.

4.1.2 Pipelines

Pipelines verified as active and included in the water model are broken down by diameter in Table 4-1. The total length of pipeline in accordance with the model is 431,536 feet (81.7 miles).

Main line pipelines (all pipelines 10 inches in diameter or larger) are shown in Figure 4-1. Main lines have a number of specific purposes including reservoir equalization, connectivity of distant neighborhoods to the grid, pump station intake, and support of fire suppression for institutional, commercial, and industrial areas. These main lines form a backbone for transmission throughout the BBCCSD Service Area and are one of the distribution system's most significant strengths. The extent and connectivity of the main line pipelines have clearly been carefully planned and implemented.

A variety of pipe materials make up the existing distribution system, including steel, polyvinyl chloride (PVC), and asbestos concrete (AC). In general, the steel pipes are the oldest and show the greatest signs of deterioration. AC pipes are generally in good condition. PVC pipes have been installed more recently. BBCCSD is making an effort to replace all 2-inch steel pipes with 8-inch PVC pipes to address the age and condition issues associated with the existing steel pipelines.

4.2 Booster Pump Facilities

A booster pump is typically housed with several other pumps at a single station; this is the case at BBCCSD. The 19 booster pumps are located at 6 booster pump stations named for the pressure zones they serve. The number of pumps and total flow capacity for each booster pump station are listed in Table 4-2.

The Rowe boosters pump water from the Rowe Reservoir into the Rowe Pressure Zone and into the Hicks, Peery, and Greenspot Reservoirs. The remaining booster pumps provide the energy required to maintain the artificial hydraulic gradient of their respective pressure zones.

Each booster pump is designed to produce a target flow. The actual flow that a booster provides may vary from the design flow due to the conditions under which the pump is tested, demand fluctuation, the age of the pump, and changes in the system not accounted for in the original design. The Rowe boosters and Peery boosters 1, 2, and 3 were tested in 2007. The Abbott and Greenspot boosters and Tanglewood booster 1 were tested in 2008. Test data for the Hicks boosters, Tanglewood boosters 2 and 3, and Peery booster 4 are out of date. Table 4-3 summarizes data concerning the BBCCSD booster pumps. The Rowe booster pumps are shown in Figure 4-3.

Each of the booster stations servicing the smaller pressure zones is equipped with a fire pump to meet the flow and pressure requirements mandated by the fire marshal. Figure 4-4 shows a typical fire pump.

4.3 Groundwater Supply System

BBCCSD's sole source of supply is groundwater from 13 vertical wells, 2 gravity slant wells, and 2 natural springs. Of the 13 vertical wells, 2 (Wells 5 and 7A) are inactive. The active vertical wells have a total design capacity of 2,317 gpm. Of the 11 vertical wells, 9 wells (1, 1B, 3, 3A, 4, 4A, 6, 8, and 10) pump to the Rowe Reservoir through the Restricted Main for fluoride blending. The other 2 vertical wells (2 and 9) pump directly into the Rowe Pressure Zone. Production from the gravity slant wells and springs is closely tied to precipitation and highly dependent on the hydrogeology of the region. Production from the slant wells in Van Dusen Canyon feeds the Rowe Pressure Zone by gravity. The slant wells are located on forest service property approximately 3,000 feet north of the north end of Wendy Avenue. The pipeline feeding production from the Van Dusen slant wells to the Rowe Pressure Zone was replaced in 2008 with 4-inch PVC pipe and a small booster pump. Flows from the natural springs are diverted to the Greenspot Reservoir. The springs are located south of the Greenspot reservoir and enter the reservoir through a common line and meter.

Data from BBCCSD's logbooks provide flow rate (gpm) and discharge pressure (psi) at each well. Chlorine residual is recorded at Wells 2, 6, 9, and 10, at the slant wells, and at the springs. (Chlorine residual is also monitored at the Well 8 filtration plant and Rowe Booster Station.)

In 2003, Well 5 was removed from service and Well 8 was brought on-line. Beginning in 2004, both Well 9 and Well 10 were brought into service. There are currently plans to replace Well 3 with a new well at the same site designated as Well 3B. Well 3B has high production capability (1,050 gpm) but also high fluoride concentration (7.5 milligrams per liter [mg/L]). As a result, Well 3B can only be operated at approximately 416 gpm to achieve compliance with the fluoride maximum contaminant level (MCL) after blending with the existing low-fluoride sources (Section 7.1.1). In 2011, a new well (the Sugarloaf Well) will be in operation at 300 gpm. Negotiations are currently underway to procure sufficient low-fluoride water from BBLDWP to maximize production from Well 3B. Figure 4-5 illustrates the 2007 contribution of each well to the monthly supply for BBCCSD.

Figure 4-6 shows Well 10. Table 4-4 summarizes well drilling data and Table 4-5 provides the well pump data. Wells 1, 2, 3A, 4, 6, and 9 were tested in 2007. The remaining active wells were tested in 2008 (Wells 5 and 7A are inactive.)

The slant wells and the springs feed the distribution system by gravity and typically require no pumping; however, a small pump has been installed at the site of the Van Dusen slant wells to overcome head loss in the new 4-inch discharge pipeline. The new discharge pipeline replaced the original riveted steel pipeline, which was larger in diameter but no longer met public health standards. The 1991 Water Master Plan indicates that the combined flow rate for the slant wells ranges from 40 to 300 gpm and the combined flow rate for the springs ranges from 100 to 550 gpm. For purposes of calculating available supply, gravity production totals from 2007 have been adopted. 2007 is considered by BBCCSD as a typical "dry" year. For 2007, the average combined flow from the slant wells was calculated at 138 gpm and the average combined flow from the springs was calculated at 172 gpm.

4.4 Emergency Water Connections

In 1995, BBCCSD entered into an agreement to provide emergency interties with BBLDWP. The interties consist of adjacent fire hydrants, one maintained by BBCCSD and the other by BBLDWP, that are connected hydraulically, thus linking the two districts. Each intertie has a capacity of 1,000 gpm and links to the Rowe Pressure Zone. The Division Intertie is located on the west side of the BBCCSD Service Area at the intersection of Division Drive and West Country Club Boulevard. The Erwin Ranch Intertie is located near the east side of the BBCCSD Service Area at the intersection of Erwin Ranch Road and Greenspot Boulevard (SR 38).

Figure 4-1 shows the locations of the interties. Figure 4-7 shows the Erwin Ranch Road Emergency Intertie site.

Emergency connections are typically used during times of natural or manmade disasters. In addition, the agencies may transfer water during repairs and maintenance. There are no formal agreements regarding water transfer. Each transfer is evaluated individually. However, if the interties are sized appropriately and institutional arrangements are made, a more formal transfer program could be implemented.

BBCCSD has been granted a variance by the State of California for an elevated fluoride concentration: BBCCSD may exceed the current fluoride MCL of 2 mg/L and deliver a fluoride concentration of 3 mg/L to its customers. However, this variance may interfere with the development of water transfer agreements because adjacent water districts may be held to a stricter standard.

Discharge of water from the BBCCSD system does occur from the Greenspot Reservoir to overflow waste during times of low demand with naturally high production from Greenspot and Hatchery Springs. BBCCSD has installed an overflow meter to document this formerly unaccounted water.

BBCCSD has no imported water connections. The nearest imported water wholesaler is Crestline Lake Arrowhead Water Agency, and the distance and mountainous terrain make a

potential intertie prohibitive. Other area water agencies for potential intertie and transfer include:

- Big Bear Lake Municipal Water District, which manages Big Bear Lake primarily for recreation and does not have any surface flow water rights
- Bear Valley Mutual Water Company, which owns lake water rights
- Big Bear Area Regional Wastewater Agency, which currently collects potential recycled water directly from the BBCCSD Service Area

4.5 Reservoirs

BBCCSD currently maintains four storage tanks with a combined storage capacity of approximately 6.24 million gallons (19.1 acre-feet). This volume is equivalent to 5.6 days of present ADD and 2.9 days of present MDD. Table 4-6 summarizes the reservoir data. Figure 4-8 shows the Greenspot Reservoir.

4.6 Pressure-Reducing Stations

BBCCSD maintains one pressure-reducing station, located on the southeast corner of the intersection of Maltby Boulevard and Bluebill Drive. The station's function is to relieve pressure from the Well 10 site before the water enters the low-pressure fluoride blending line (Restricted Main). The equipment includes a 4-inch Hytrol pressure-reducing Cla-Val, model #81-02-18A, which is currently set to reduce pressure from 110 psi to 10 psi.

4.7 SCADA System

The Water Distribution Telemetry System is designed to offer enhanced data acquisition through monitoring of the system status and alarms and to control of all pumps, tanks, and wells. BBCCSD maintains a software-driven computer-based SCADA system, which includes

instrumentation and control equipment for monitoring, communications, alarm, control, display, and reporting functions.

The heart of the system is the central computer system located at the BBCCSD District Office, which consists of a fault-tolerant server connected to an ethernet local area network. The server is equipped with a redundant power supply and hard disc. The system may be accessed remotely by laptop or by dial-up phone lines through a remote access server.

The central node of the SCADA system is the data concentrator. The data concentrator communicates with remote sites by radio using Netview software and notifies operators about alarms by phone through an automated system using AlarmWorX+ software by Iconics. The Human Machine Interface software is OpenEnterprise by Bristol Babcock. It serves as the main operator interface for monitoring and control of the SCADA system and features graphic screens, trends, and storage of collected historical data.

Figures 4-9a and 4-9b show images from the SCADA interface.

The precursor to and current backup to telemetry, logbook records, are still maintained by system operators. A total of 20 separate logbooks are kept for:

- Hicks Booster Pump Station
- Tanglewood Booster Pump Station
- Edward E. Abbott Booster Pump Station
- Greenspot Reservoir and Booster Station
- Peery Booster Pump Station and Reservoir
- Joseph A. Rowe Booster Station
- Greenspot Spring, Greenspot Reservoir, Fish Hatchery Spring Station
- Van Dusen slant wells and chlorinator
- Well 1
- Well 1B
- Well 2
- Well 3

- Well 3A
- Well 4
- Well 4A
- Well 6
- Well 8
- Well 8 Filtration Plant
- Well 9
- Well 10

4.8 Water Treatment Facilities

4.8.1 Fluoride Blending

Several BBCCSD wells exhibit high fluoride concentrations. Although treatment was considered, BBCCSD opted to blend the high fluoride concentration sources with low fluoride concentration sources to meet health standards. BBCCSD currently holds a variance from the State of California to exceed the prevailing fluoride MCL of 2 mg/L up to a concentration of 3 mg/L.

Fluoride blending occurs within a Restricted Main that connects a series of wells to the Rowe Reservoir. The current fluoride blending system has been operational since June 1990. The original system consisted of pipelines, a pumping station, a 2.5-million-gallon blending reservoir, and renovations to seven existing wells (1, 1b, 3, 3a, 4, 4a, and 6). Wells 8 and 10 were recently added to the fluoride blending system. Well 10 is a low-fluoride well drilled in 2004. Well 8 was added to the blending system in 2008 following completion of an iron and manganese treatment facility (Section 4.8.2). Operation of the existing water system was modified so that the nine wells all pump directly into a Restricted Main that then delivers the water to the Rowe Reservoir, located at the Paradise Maintenance Yard. Well waters with differing fluoride concentrations are blended in the Restricted Main en route to the Rowe Reservoir; from the reservoir the blended water is pumped directly into the Rowe Pressure Zone. This blended water is tested on a weekly basis as required by the State of California Public Health Department.

BBCCSD also samples the nine source wells weekly, and the source fluoride levels are updated on a theoretical fluoride blending spreadsheet immediately upon receiving the laboratory results of the fluoride concentrations. The well turn-on setpoints are then adjusted to achieve the most effective blend.

BBCCSD is currently exploring options for blending a new high-fluoride-concentration source (Well 3B). Insufficient low-fluoride-concentration sources are available at this time to offset the new high-fluoride source; however, a new well in the vicinity of the Sugarloaf community and imported water from BBLDWP are being considered. If a low-fluoride-concentration source can be found, BBCCSD will be able to maximize the production of Well 3B, which has an established production capacity of 1,050 gpm.

4.8.2 Iron and Manganese Removal at Well 8

Well 8 is fitted with a Kinetico groundwater treatment system that involves a two-step process of oxidation and filtration to remove iron and manganese. The oxidation step consists of adding chlorine using a chemical feed pump and raising the pH with sodium hydroxide. The filtration step involves downflow pressure filtration and upflow backwash through a ceramic filtration media. The system consists of three steel tanks, each limited to a flow of 200 gpm for a normal service flow capacity of 600 gpm. The entire system is controlled through a programmable logic controller (PLC). The PLC controls the various automatic sequences for filtration and chlorine dosing. Figure 4-10 shows the Well 8 treatment system tanks.

4.8.3 Chlorination

Disinfection by injection of 12.5 percent liquid sodium hypochlorite occurs at the seven locations indicated in Table 4-7.

Each facility is equipped with a positive displacement chemical feed pump that draws from a 55-gallon drum and injects the sodium hypochlorite solution directly into the distribution system. The pumps are sized to maintain a 1.0-mg/L chlorine residual throughout the system. Surplus

chlorine is stored at the Rowe Booster Station. All other sites have between 50 and 100 gallons on hand. Figure 4-11 shows the chlorine injection system for Well 2.

4.9 Backup Power Generation

BBCCSD maintains backup power generators at the Abbott, Greenspot, Hicks, Peery, Rowe, and Tanglewood Booster Stations and at the Well 8 site. A portable generator is also housed at the Well 1/1B site. Because there is no elevated storage in the Abbott, Greenspot, Hicks, and Peery Pressure Zones, demand during a power outage can only be met by means of backup power generation. Figure 4-12 shows a typical stationary power generator; Figure 4-13 shows the portable generator.

5. Hydraulic Model

In support of the BBCCSD Water Master Plan, IDM updated, calibrated, and evaluated the hydraulic model of the water distribution system and has developed this section to discuss the processes involved and assumptions made during the project. Sections 5.1 through 5.5 discuss the model facility data update, demand development, steady-state calibration, future scenario development, and master plan model evaluations, including recommendations for the existing and future water systems in planning years 2008, 2013, and 2030.

5.1 Model Facility Data Update

The model for the BBCCSD water system provided by So & Associates Engineers, Inc. (SAE) was used as the starting point for the project. This hydraulic model was updated using MWH Soft's H2ONET Version 8.0 for AutoCAD 2008.

It was confirmed with BBCCSD that the connectivity and/or configuration of several facilities in the original model should be updated to correctly reflect conditions in the field. BBCCSD, SAE, DBS&A, and Civiltec staff provided significant guidance during the update process. In addition, the AutoCAD basemap of the water system was used as a reference to complete the connectivity and configuration changes. These updates are discussed in Sections 5.1.1 through 5.1.3.

5.1.1 Tanks

Model attributes required for the tank facilities include base elevation, maximum water level, initial water level (assumed or known starting point), and diameter. The tank locations and attributes from the BBCCSD water system model provided by SAE were used in the updated model. Table 5-1 summarizes the model tank dimensions for each of existing tanks in the BBCCSD system.

5.1.2 Pumps

Pumps in the original model were typically modeled with design point curves. Pump test information was collected in May 2001 for several well and booster pumps and was provided to IDM by BBCCSD to update these pump design points. Table 5-2 displays the final pump design point attributes used in the model.

Several well pumps were assigned pump curves based on pump manufacturer curve information provided to IDM by BBCCSD. The following well pump curves can be found in Appendix C:

- Well 1
- Well 1B
- Well 3
- Well 4
- Well 4A
- Well 6
- Well 8
- Well 10

5.1.3 Pressure Zone Designation

The BBCCSD system has six pressure zones: Abbott, Greenspot, Hicks, Peery, Tanglewood, and Main. IDM attributed the pipes and junctions in the model with their appropriate pressure zone designation using the “BBCCSD Water System Map_AC” AutoCAD map provided by BBCCSD. The pressure zone designations and discussions with the BBCCSD staff helped to identify the closed valves that needed to be created in order to separate the pressure zones. Table 5-3 provides the locations of the pressure zone breaks added to the model.

5.2 Demand Allocation

The original water model had not been updated to represent recent growth. Areas in the water model with zero or very low demand allocation were examined in detail by Civiltec. Those areas include the Hicks, Peery, Abbott, Greenspot, and Tanglewood pressure zones. Consumption records between 2003 and 2007 were tabulated as percentages of total consumption on an annual basis. The highest percentage over that period for each area was multiplied by the 2008 ADD of 1,238 ac-ft/yr and distributed evenly.

Additionally, based on Civiltec's analysis, three individual consumers have demands an order of magnitude higher than typical customers: Whispering Pines Estates, Stickleback Fish Environment, and Big Bear High School. Demands for these large customers are represented in the model as a single node point demand as directed by Civiltec.

According to Civiltec, 2008 ADD for the BBCCSD water system is 767.5 gpm. This consumption demand provided by Civiltec for model allocation was scaled up by 10 percent to account for water loss. This demand is considerably lower than the demand allocation in the original model. BBCCSD confirmed that this reduced demand was the result of conversion from a flat fee to a metered billing system.

Hydraulic model demand allocation is largely driven by the amount of data available to correlate water usage with a customer location or land use type within the service area. This geographic relationship allows for accurate demand assignments to the appropriate hydraulic model junction. As geocoded customer locations were not available, the demands for the BBCCSD water system were allocated by evenly distributing each zone's demand across all of the service nodes within that zone. Table 5-4 displays the ADD table provided by Civiltec with the values that were allocated to the model per pressure zone as well as the ADD for the three largest water users, which were added to the model as single-node point demands.

In addition, the three large users listed at the bottom of Table 5-4 were loaded to specific junctions within the model. Table 5-5 displays the locations of these large users within the model.

The MDD peaking factor is the ratio of the MDD to the ADD and was calculated by Civiltec to be 2.0 for the BBCCSD system. This factor was applied to the model to convert the ADD into MDD.

5.3 Steady-State Calibration

Steady-state (SS) calibration was based on six hydrant tests performed and recorded by BBCCSD and IDM on June 10, 2008. Figure 5-1 illustrates the locations in the BBCCSD system where the hydrant tests were performed. Twelve unique model scenarios were created, two for each of the six field hydrant tests (one representing static conditions, one representing dynamic [flowing] conditions). The goal of the SS calibration was to obtain a reasonably close difference between the model pressure drop and the field pressure drop observed on June 10, 2008. The boundary conditions (well discharge pressure, well flow, and tank level) for each SS calibration scenario were gathered during hydrant testing and used to establish model boundary conditions for each scenario.

A separate demand set was created for each of the six static and six dynamic fire flow tests. Demand allocation as described in Section 5.2 was used as the base demand for each demand set in all static and dynamic scenarios. The fire hydrant flow for each fire flow test was added as a point load demand to the corresponding dynamic scenario within the model. Since the static and dynamic test runs occur within minutes of each other, the baseline system demands for each fire flow test's static and dynamic conditions were considered to be equal.

5.3.1 Demand Adjustments for Steady-State Calibration

SS calibration is typically sensitive to system demand. To achieve the desired calibration goal, adjustments were made to the demands for each calibration scenario. This section discusses the adjustments made to the model's MDD that were necessary for model calibration.

The total system demand (i.e., pump flows) was not available during the hydrant testing. Therefore, the initial demands for all steady-state scenarios were based on the MDD demand

loading of 1,535 gpm. These were then adjusted in the model, during the calibration, to match the pressures observed in the field. Table 5-6 shows the demand adjustments for each test.

5.3.2 System Connectivity and Pipeline Roughness Adjustments for Steady-State Calibration

Further adjustments to the model were found to be necessary to achieve this project's desired calibration goals. Specific calibration adjustments centered on two primary areas:

1. System connectivity
2. Pipeline roughness (C-factors)

Adjustments were made to the system connectivity based on conversations and remote computing sessions with SAE and BBCCSD staff.

The pipelines in the original model were not attributed with material and age. Further, these attributes were not included in the AutoCAD drawing supplied by BBCCSD; therefore, C-factors were considered to be a model parameter that could be adjusted to help achieve the desired calibration accuracy. Accordingly, IDM adjusted C-factors based on conversations and/or remote computing sessions with BBCCSD staff.

5.3.3 Steady-State Calibration Results

Final model SS calibration results (Table 5-7) showed close agreement with the field static and dynamic pressure measurements recorded during hydrant testing. As shown in Table 5-7, 89 percent of all model calibration points were within 5 psi of what was measured in the field.

A more detailed comparison between the results obtained from the model and the field values measured during the hydrant testing can be found in Appendix D. This file shows the comparison values for each hydrant examined at each test site.

For the static condition scenarios, the model was able to predict pressures within 5 psi of field values for 75 percent of the test locations and within 10 psi for 25 percent of all test locations.

For the dynamic condition scenarios, 73 percent of test values were within 5 psi, and 27 percent of values were within 10 psi of the field-measured values.

5.4 Master Plan Scenario Development

Future scenarios were developed using the SS calibrated model of the BBCCSD water system as a baseline. IDM used information provided by Civiltec to update the future demand sets for each planning scenario. Table 5-8 lists the scenarios that were created for this Master Plan.

5.4.1 Future Demand Development

Future ADD was provided by Civiltec for the years 2013 and 2030. These demands were input to the model and scaled using a factor of 2.0 to estimate system MDD and a factor of 4.0 to estimate PHD. Table 5-9 illustrates the ADD, MDD, and PHD for each planning year. Initial model simulations were completed using a more conservative demand projection for 2030 than indicated in Table 2-12. No hydraulic deficiencies were uncovered as a result of the more conservative demand projection; therefore, re-evaluation with the more accurate demand projection was deemed to be unwarranted.

5.4.2 Master Plan Model Evaluations

The SS calibrated hydraulic model was used to support the analysis of the BBCCSD's system and to identify existing and future deficiencies. The model was used to identify system and pipeline improvements that would address the areas identified as having deficiencies. Results from these analyses were used as the initial basis for the capital improvement plan (CIP).

5.4.3 Planning Scenarios

The hydraulic model was used to identify specific system deficiencies in the existing and future distribution system and to analyze recommended system improvements. At the direction of Civiltec, system deficiencies were identified by running the model under two specific operational conditions: maximum day demand (MDD) and maximum day demand plus fire flow (MDD+FF).

If the system can satisfy the specified design criteria under MDD+FF conditions, the system will typically be able to satisfy the design criteria at other times of the year when conditions are less severe.

The analysis runs completed for the project are summarized in Tables 5-10 and 5-11. Table 5-10 identifies the model facility set, type of simulation run, and the system demands used in each scenario to analyze the BBCCSD system prior to the suggested improvements. Table 5-11 identifies the model facility set, type of simulation run, and the system demands used in each scenario to analyze the BBCCSD system with the suggested improvements.

5.4.4 Design Criteria

Specific design criteria for MDD and MDD+FF demand conditions were provided by Civiltec. These design criteria were used to identify system deficiencies for each planning period. The design criteria are specific to a given demand condition and are as follows:

- MDD design criteria
  - Maintain service pressures between 40 psi and 120 psi.
  - Maintain pipeline velocities less than or equal to 5 feet per second (fps).
- MDD+FF design criteria
  - Maintain a minimum residual pressure of 20 psi at the flowing hydrant when the hydrant is supplying the full required flow.
  - Maintain pipeline velocities less than or equal to 10 fps during fire flow conditions.
  - A fire flow of 1,500 gpm was assumed (based on information from Civiltec) for residential areas. A fire flow of 3,000 gpm was assumed for commercial areas. For commercial areas, combining simultaneous flow from multiple hydrants is acceptable.

For smaller systems, the fire flow requirements are typically what determine the pipe sizing for the system improvements. The total fire flow value required for smaller systems normally

exceeds the PHD and MDD demands for the region affected by the fire flow. As such, most system pipeline improvements are driven (or sized) by fire flow requirements rather than other requirements. This was found to be the case for the BBCCSD system. Most pipeline improvements were controlled by fire flow deficiencies.

5.4.5 Pipeline Improvement Assumptions

Where system deficiencies were observed in the model, a pipeline improvement was identified to resolve the deficiency. The ability of a pipeline improvement to resolve an identified deficiency was verified by adding model piping to reflect the improvement to the piping network and then re-running the scenario to confirm that the deficiency was resolved.

The types of improvements added to the pipeline network in the model were added as parallel pipelines to the existing system network for pipelines larger than 12 inches in diameter. For improvements to pipelines 12 inches or smaller in diameter, a replacement pipeline was recommended. In general, the minimum pipeline size for any future system improvement was considered to be an 8-inch pipeline.

5.5 Existing Water System Evaluation Results

An evaluation of the existing system was completed as a first step in the hydraulic model analysis. This evaluation included MDD and MDD+FF runs using existing system piping, existing wells, and existing system demands. These model runs identified existing deficiencies in the water system. These existing system deficiencies were addressed first. The future system scenarios were then run with the proposed improvements to determine if any additional improvements would be needed.

5.5.1 Existing Maximum Day Plus Fire Flow Hydraulic Deficiencies

MDD+FF analysis of the existing system was completed using the EX_SS_MDP+FF scenario (Table 5-8). In this scenario, a required fire flow of 1,500 gpm was used at all residential hydrant junctions, while 3,000 gpm was used at all commercial hydrants. Individual hydrants

are not included in the model; thus the nearest junction was used to represent the hydrant location. The software calculated the available fire flow at the hydrant while maintaining a minimum pressure at the hydrant of 20 psi and maintaining pipeline velocities under 10 fps. Hydrants that were unable to provide the required available fire flow under the criteria were identified as deficient. A map of the system fire flow deficiencies is included in Appendix E.

Existing system MDD+FF analysis results were also stored within the model database tables. Available fire flow results for each fire flow junction were attributed to the respective junction's "EXFF_AQ" data field in the model.

A total of 159 out of 526 hydrants (~30 percent) analyzed were found to have deficient fire flow. This number includes 151 residential hydrants that did not meet the requirement of an available fire flow of 1,500 gpm and 8 commercial hydrants that did not meet the requirement of 3,000 gpm.

Additional analyses were performed on the commercial hydrants to confirm that the 3,000-gpm requirement could be met if used simultaneously with an adjacent hydrant. Each of the 8 hydrants was analyzed using the H2ONET's "Multi-Fireflow" tool. It was confirmed that 7 of these commercial locations could supply a total fire flow of more than 3,000 gpm when flowing simultaneously with an adjacent hydrant (Table 5-12). The hydrant at Junction 538 did not meet the 3,000-gpm fire flow requirement when flowing simultaneously with an adjacent hydrant and will need additional improvements.

The 151 residential hydrants were located throughout the system and their fire flow deficiencies appear to be caused by undersized pipelines, insufficient looping, and/or high elevations. Pipeline improvement projects were identified to resolve the identified deficiencies of the existing system under MDD+FF conditions caused by undersized piping and lack of looping. For areas with high elevations, deficiencies were reconciled by other means. Table 5-13 summarizes these high-elevation deficiencies and provides insight into their resolution.

5.5.2 Existing Maximum Day Deficiencies

The MDD analysis of the existing system was completed using the EX_SS_MDP scenario (Table 5-8). A map of the pressure and velocity deficiencies under MDD conditions is provided as Appendix F.

Existing system MDD analysis results were stored within the model database tables. Existing system MDD pressures can be found for any junction in the “EX_MDP” data field within the model. Existing system MDD velocities can be found on any pipeline in the “EX_VEL” data field within the model.

The analysis identified several locations in the system with service pressures below 40 psi. Many of these locations are on the upstream side of pumps and therefore will not impact service pressure. The other locations appear to have low pressures due to the elevation of the terrain. Locations that are higher than 115 feet below the zone’s HGL will typically have difficulty maintaining a minimum system pressure of 40 psi. As a result, pipeline improvements will typically not resolve low pressures in these areas. For example, the junctions along San Martin Drive are at elevations between 6,870 and 6,890 feet msl. The HGL along San Martin Drive is at approximately 6,953 feet msl, and the head is between 64 and 84 feet, which is equivalent to approximately 27 psi to 36 psi. Areas with low pressures due to high elevations could be due to the elevation uncertainty observed during model calibration. It is recommended that model areas with high or low pressures be reviewed to confirm the model elevations.

At several locations the service pressure exceeded the desired maximum of 120 psi. All except for one of these locations occur in the Peery Pressure Zone. The pressures in this zone range from 56 psi to 135 psi. IDM reduced the pressures at the Peery Booster Pumps by 15 psi, resulting in pressures within the target operating range of 40 psi to 120 psi. This area should be investigated to confirm the pressures in the Peery Pressure Zone. Based on the model results, customers located at an elevation below 6,840 feet msl may experience service pressures of 120 psi or greater. One option to mitigate these high service pressures would be to install individual pressure-reducing valves (PRVs) at the homes of the high pressure locations.

The existing MDD analysis run also identified seven pipelines with high velocities. Two of the pipelines were short interconnecting pipelines located at East Big Bear Boulevard and Shore Drive. The remaining five pipelines exceeding the design criteria pipeline velocities are located at pump stations.

5.5.3 System Improvements to Address Existing Deficiencies

A number of pipeline improvements were identified to address the deficiencies identified in the analysis of the existing system. Improvements were completed to address these issues:

- Provide a main line sized to handle the required fire flow (size was based on maintaining a velocity under 10 fps and supplying at least 1,500 gpm while keeping the pressures above 20 psi). Smaller pipeline diameters were suggested when pipelines were looped, while larger pipeline diameters were suggested on dead end mains.
- Improve overall system connectivity for fire protection.
- Address locations where system looping was not currently in place.

Pipeline improvements were simulated in the model to verify that the proposed improvements would resolve the identified deficiencies. The proposed pipeline improvements are mapped and summarized in Appendix G.

5.5.4 Future Water System Analysis

Evaluations of the proposed future system were completed using the hydraulic model for planning years 2013 and 2030. These evaluations included MDD and MDD+FF runs using the existing system piping plus proposed pipeline improvements and future system demands. These analysis runs were used to identify deficiencies in the system for each of the given planning years.

The deficiencies for the existing system were driven by fire flow requirements provided by the fire department. These fire flow requirements are assumed to be the same in the future system; therefore, future system model analyses did not identify any additional deficiencies.

5.5.5 Summary of Recommended Existing Water System Pipeline Improvement Projects

This section includes a summary of the pipeline improvement projects that were identified as part of the existing system model analysis. The CIP table in Appendix G includes a detailed list of all pipeline improvements. This section organizes those pipeline improvements into project groups within three different categories: improvements driven by inadequate fire flow caused by low pressure, improvements driven by inadequate fire flow caused by high velocity, and improvements driven by inadequate fire flow caused by low pressure and high velocity.

Within each category, the projects are given a priority. The priority is based on the number of hydrant locations impacted by the improvement. High-priority projects affect five or more locations. Medium-priority projects affect three or four locations. Low-priority projects affect one or two locations. A map highlighting each project area is included in Appendix G, and their priority is discussed in the following subsections. These projects, which are based entirely on hydraulic modeling, were later merged in whole or in part with other improvements recommended in analyses discussed in Section 7.

5.5.5.1 Improvements Driven by Inadequate Fire Flow Caused by Low Pressure

High-Priority Projects:

1. Fire flow improvement (undersized pipeline) along State Lane between Lakewood Drive and Cypress Lane
    a. Install 2,229 feet of new/parallel 12-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction IDs 522, 523, 524, 515, 516, 517, 519, 520, 529, 535, 538, 539, 540, 541, 542, 544, and 548.

2. Fire flow improvement (undersized pump) at Hicks Pump Station

    a. Replace existing fire pump with larger pump (flow: 1,500 gpm; total dynamic head [TDH]: 120 feet).

    b. Resolves insufficient fire flow for hydrants at model junction IDs J_7, J_1, J_2, J_5, 8, 6, J_23, 239, 9, J_13, and J_14.

3a. Fire flow improvement (undersized pump) at Abbot Pump Station.

    a. Replace existing fire pump with larger pump (flow: 1500 gpm; TDH: 320 feet).

    b. Resolves insufficient fire flow for hydrants at model junction IDs 201, 202, and assists in increasing fire flow for all other hydrants in the Abbott Pressure Zone.

3b. Fire flow improvement (undersized pipeline) along Beaumont Lane between Abbott Pump Station and Kean Way

    a. Replace existing 6-inch pipeline with 2,837 feet of 8-inch pipeline.

    b. Resolves insufficient fire flow for hydrants at model junction IDs 204, 257, 206, 310, and 311.

4. Fire flow improvement (undersized pipeline) along Cedar Lane, Cedarpine Lane, Shady Lane, Manzita Lane, and Fox Road between Highway 38 and Hatchery Drive

    a. Replace existing 4-inch pipeline and 6-inch pipeline with 2,613 feet of 8-inch pipeline.

    b. Resolves insufficient fire flow for hydrants at Model junction IDs 484, 489, 494, 499, and 506.

5. Fire flow improvement (undersized pipeline) along Fern Lane and Willow Lane

    a. Replace existing 6-inch pipeline with 258 feet of 10-inch pipeline along Fern Lane west of Willow.

    b. Replace existing 6-inch pipeline with 345 feet of 8-inch pipeline along Fern Lane west of Willow.

c. Replace existing 6-inch pipeline with 403 feet of 8-inch pipeline along Willow Lane between Fern and Fox Road.

d. Resolves insufficient fire flow for hydrants at model junction IDs 526, 527, 531, 532, and 533.

Medium-Priority Projects:

6. Fire flow improvement (undersized pipeline and new connection) along Lakewood Drive between Jenson Drive and Erwin Ranch

   a. Replace existing 6-inch pipeline with 656 feet of 8-inch pipeline.

   b. Install 1,572 feet of 8-inch pipeline to connect to 8-inch pipeline on Jenson Drive.

   c. Resolves insufficient fire flow for hydrants at model junction IDs 466, 467, 465, and 470.

7. Fire flow improvement (undersized pipeline) along Bernhardt Lane at Wallace Lane

   a. Replace existing 4-inch pipeline with 509 feet of 8-inch pipeline.

   b. Resolves insufficient fire flow for hydrants at model junction IDs 312, 313, and 351.

Low-Priority Projects:

8. Fire flow improvement (undersized pipeline) along Marlowe Drive west of Barrymore Road

   a. Replace existing 4-inch pipeline with 86 feet of 8-inch pipeline.

   b. Resolves insufficient fire flow for hydrants at model junction IDs 321 and N-55.

9. Fire flow improvement (undersized pipeline) along Adams Drive southeast of Barrymore Road

   a. Replace existing 4-inch pipeline with 340 feet of 8-inch pipeline.

   b. Resolves insufficient fire flow for hydrants at model junction IDs 323, 324, and 349.

10. Fire flow improvement (undersized pipeline) along Adams Drive northeast of Barrymore Road
    a. Replace existing 6-inch pipeline with 404 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 325.

11. Fire flow improvement (undersized pipeline) along Wesley Court
    a. Replace existing 4-inch pipeline with 184 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 144.

12. Fire flow improvement (undersized pipeline) along Monmouth Road west of Ringwood Trail
    a. Replace existing 6-inch pipeline with 946 feet of 8-inch pipeline.
    b. Improves insufficient fire flow for hydrants at model junction ID 469 (but does not increase fire flow above the 1,500-gpm design criterion).

13. Fire flow improvement (undersized pipeline) along Shady Lane cul-de-sac west of Wilderness Drive
    a. Replace existing 8-inch pipeline with 223 feet of 10-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID N_36.

14. Fire flow improvement (undersized pipeline) south of Hemlock Lane
    a. Replace existing 4-inch pipeline with 486 feet of 8-inch pipeline.
    b. Improves insufficient fire flow for hydrants at model junction IDs 553 and 554 (but does not increase fire flow above the 1,500-gpm design criterion)

5.5.5.2 Improvements Driven by Inadequate Fire Flow Caused by Low Pressure and/or High Velocity

High-Priority Projects:

15. Fire flow improvement to create loop and replace undersized pipeline along West Division Drive, Sugarloaf Boulevard, Sherwood Boulevard, and Robinhood Boulevard
    a. Replace existing 4-inch pipeline with 963 feet of 8-inch pipeline along West Division Drive between Big Bear Boulevard and Sugarloaf Boulevard.
    b. Replace existing 4-inch pipeline with 2,058 feet of 8-inch pipeline along Sugarloaf Boulevard between West Division Drive and Pineview Drive.
    c. Replace existing 4-inch pipeline with 1,089 feet of 8-inch pipeline along Sherwood Boulevard between Division Drive and Hillen Dale Drive.
    d. Replace existing 4-inch pipeline with 594 feet of 8-inch pipeline along Robinhood Boulevard between Division Drive and Gildart Drive.
    e. Replace existing 4-inch pipeline with 1,467 feet of 8-inch pipeline along Robinhood Boulevard between Gildart Drive and Pineview Drive.
    f. Resolves insufficient fire flow for hydrants at model junction IDs 122, 128, 123, 127, 134, 135, 136, 124, N-2, J22, J_146, 556, 153, 152, and 154.

16. Fire flow improvement (undersized pipeline) along Bernhardt Lane, Booth Way, Irving Way, Garrick Way, and Beaumont Lane
    a. Replace existing 4-inch pipeline with 857 feet of 8-inch pipeline along Bernhardt Lane between East Big Bear Boulevard and Garrick Way.
    b. Replace existing 4-inch pipeline with 322 feet of 8-inch pipeline along Booth Way east of Bernhardt Lane.
    c. Replace existing 4-inch pipeline with 173 feet of 8-inch pipeline along Irving Way east of Bernhardt Lane.
    d. Replace existing 4-inch pipeline with 217 feet of 8-inch pipeline along Garrick Way between Bernhardt Lane and Beaumont Lane.

e. Replace existing 6-inch pipeline with 179 feet of 8-inch pipeline along Beaumont Lane between Garrick Way and Kean Way.

f. Resolves insufficient fire flow for hydrants at model junction IDs 299, 300, 301, 303, 306, and 307.

17. Fire flow improvement to create two 8-inch pipelines loops along Wendy Avenue, Pioneer Lane, Michael, Avenue, and Sequoia Avenue to connect to 12-inch pipeline along North Shore Drive

    a. Replace approximately 1,099 feet of 6-inch pipeline with 8-inch pipeline along Wendy Avenue.

    b. Replace approximately 1,087 feet of 4-inch pipeline with 8-inch pipeline along Michael Avenue.

    c. Replace approximately 2,082 feet of 6-inch pipeline with 8-inch pipeline along Pioneer Lane.

    d. Replace approximately 294 feet of 4-inch pipeline with 8-inch pipeline along Sequoia Avenue.

    e. Resolves insufficient fire flow for hydrants at model junction IDs 60 and 59; improves fire flow (but does not increase fire flow above the 1,500-gpm design criterion) for model junction IDs 53, 55, and 246).

Medium-Priority Projects:

18. Fire flow improvement (undersized pipeline) along Paradise Way between East Big Bear and Kean Way

    a. Replace existing 4-inch pipeline with 611 feet of 8-inch pipeline.

    b. Resolves insufficient fire flow for hydrants at model junction IDs 235, 236, 237, and 238.

19. Fire flow improvement to create loop along Maltby Boulevard and Shore Drive
    a. Replace existing 4-inch pipeline with 1,431 feet of 8-inch pipeline along Maltby Boulevard between Pintail Drive and Shore Drive.
    b. Replace existing 6-inch pipeline with 1,639 feet of 8-inch pipeline along Shore Drive between Maltby Boulevard and County Club Boulevard.
    c. Resolves insufficient fire flow for hydrants at model junction IDs 557, 264, 265, and 332.

20. Fire flow improvement to create an 8-inch pipeline loop along Arbor Lane to Sierra Avenue, San Anselmo Drive, and Mt. Doble Drive
    a. Replace approximately 656 feet of 6-inch pipeline with 8-inch pipeline along Arbor Lane.
    b. Replace approximately 279 feet of 4-inch pipeline with 8-inch pipeline along Arbor Lane.
    c. Replace approximately 266 feet of 2-inch pipeline with 8-inch pipeline along Arbor Lane.
    d. Replace existing 4-inch pipeline with 570 feet of 8-inch pipeline and connect to 12-inch pipeline on Paradise Way.
    e. Replace existing 4-inch pipeline with 562 feet of 8-inch pipeline.
    f. Resolves insufficient fire flow for hydrants at model junction IDs 86, 96, 89, and 88.

21. Fire flow improvement to create looping along Mountain View Boulevard and Mt. Doble Drive
    a. Replace existing 4-inch pipeline with 1,138 feet of 8-inch pipeline.
    b. Replace existing 4-inch pipeline with 483 feet of 8-inch pipeline.
    c. Resolves insufficient fire flow for hydrants at model junction IDs 226, 216, and 217.

22. Fire flow improvement to create loop and replace undersized pipeline along Sugarloaf Boulevard and Dawn Drive
    a. Replace existing 4-inch pipeline with 766 feet of 8-inch pipeline along Sugarloaf Boulevard between Pineview Drive and Dawn Drive.
    b. Replace existing 4-inch pipeline with 612 feet of 8-inch pipeline along Dawn Drive, between Sugarloaf Boulevard and West Big Bear Boulevard.
    c. Resolves insufficient fire flow for hydrants at model junction IDs 162, 163, and 140.

23. Fire flow improvement to create 8-inch pipeline loop along Aeroplane Boulevard and Greenway Drive
    a. Replace existing 4-inch pipeline with 972 feet of 8-inch pipeline along Aeroplane Boulevard between Gold Mountain and Greenway Drive.
    b. Replace existing 4-inch pipeline with 802 feet of 8-inch pipeline along Greenway Drive between Aeroplane Boulevard and East Big Bear Boulevard.
    c. Resolves insufficient fire flow for hydrants at model junction IDs 210, 211, and 212.

24. Fire flow improvement (undersized pipeline) along Stanford Road between Whipple Road and Greenspot Road, along Sires Way between Montclair Drive and Greenspot Road, and along Greenspot Drive
    a. Replace existing 6-inch pipeline with 250 feet of 8-inch pipeline along Stanford Road.
    b. Replace existing 6-inch pipeline with 328 feet of 8-inch pipeline along Sires Way.
    c. Replace existing 2-inch pipeline with 530 feet of 8-inch pipeline along Greenspot Drive.
    d. Resolves insufficient fire flow for hydrants at model junction IDs 388, 398, and 387.

Low-Priority Projects:

25. Fire flow improvement (undersized pipeline) along Glencove Drive and Willow Lane
    a. Replace existing 6-inch pipeline with 1,134 feet of 8-inch pipeline along Glencove Drive.
    b. Replace existing 6-inch pipeline with 460 feet of 8-inch pipeline along Willow Lane between Glencove Drive and Tulip Lane.
    c. Resolves insufficient fire flow for hydrants at model junction IDs 551 and 534.

26. Fire flow improvement at Hale Drive and Holden Avenue
    a. Install approximately 300 feet of 8-inch pipeline to create loop between Wendt Avenue and Van Dusen Canyon Drive.
    b. Resolves insufficient fire flow for hydrants at model junction IDs 35 and 34.

5.5.5.3 Improvements Driven by Inadequate Fire Flow Caused by High Velocity

High-Priority Projects:

27. Fire flow improvement to create 8-inch pipeline looping Sequoia Avenue, Cinderella Drive, Hugo Lane, and Grenfall Lane, to connect to 12-inch pipeline along Paradise Way
    a. Replace approximately 364 feet of 4-inch pipeline with 8-inch pipeline along Sequoia Avenue.
    b. Replace approximately 901 feet of 4-inch pipeline with 8-inch pipeline along Cinderella Drive.
    c. Replace approximately 1,223 feet of 6-inch pipeline with 8-inch pipeline along Hugo Lane and North Shore Boulevard to Paradise Way.
    d. Replace approximately 1,045 feet of 4-inch pipeline with 8-inch pipeline along Hugo Lane and Grenfall Lane to Paradise Way.
    e. Resolves insufficient fire flow for hydrants at model junction IDs 116, 99, 100, 101, 98, 108, 109, and 110.

Medium-Priority Projects:

28. Fire flow improvement to create looping along West Meadow Lane from Sierra Avenue to Wendy Avenue
    a. Replace existing 4-inch pipeline with 878 feet of 8-inch pipeline.
    b. Replace existing 6-inch pipeline with 1,438 feet of 8-inch pipeline.
    c. Resolves insufficient fire flow for hydrants at model junction IDs 71, 74, 75, and 77.

29. Fire flow improvement (undersized pipeline) along Greenspot Road between Malabar Way and Mann Drive
    a. Replace existing 6-inch pipeline with 826 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction IDs 384, 385, and 386.

Low-Priority Projects:

30. Fire flow improvement (undersized pipeline) along Sawmill Drive between Aeroplane Boulevard and Rainbow Boulevard
    a. Replace existing 6-inch pipeline with 247 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction IDs 193 and 194.

31. Fire flow improvement (undersized pipeline) near White Mountain Drive cul-de-sac
    a. Replace approximately 294 feet of 6-inch pipeline with 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction IDs 10 and 11.

32. Fire flow improvement (undersized pipeline) along Brewer Way between Malabar Way and Zaca Road
    a. Replace existing 4-inch pipeline with 1,320 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction IDs 397 and 411.

33. Fire flow improvement (undersized pipeline) on Feather Mountain Drive
    a. Replace approximately 124 feet of 6-inch pipeline with 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 22.

34. Fire flow improvement (undersized pipeline) along Rosehill Drive between West Fair Way and West Country Club
    a. Replace existing 4-inch pipeline with 250 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 254.

35. Fire flow improvement (new pipeline) at Country Club Boulevard at Keiner Drive
    a. Install 25 feet of 8-inch pipeline and connect to 8-inch pipeline along Keiner Drive.
    b. Resolves insufficient fire flow for hydrants at model junction ID 717.

36. Fire flow improvement (undersized pipeline) along Big Tree Drive at Sherwood Boulevard
    a. Replace existing 4-inch pipeline with 60 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 181.

37. Fire flow improvement (undersized pipeline) along Big Tree Drive between Aeroplane Boulevard and Rainbow Boulevard
    a. Replace existing 4-inch pipeline with 149 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 179.

38. Fire flow improvement (undersized pipeline) at Sherwood Boulevard and Sawmill Drive
    a. Replace existing 4-inch pipeline with 38 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 191.

39. Fire flow improvement (undersized pipeline) at Monclair Drive north of Bodie
    a. Replace existing 6-inch pipeline with 11.92 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 422.

40. Fire flow improvement (undersized pipeline) along Panamint Mountain Drive, between Lookout Mountain Road west to cul-de-sac (Hicks Pressure Zone)
    a. Replace existing 6-inch pipeline with 479 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID J_15.

41. Fire flow improvement (new pipeline) along walkway between East Big Bear and Malabar Way
    a. Install 611 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 381.

42. Fire flow improvement (undersized pipeline) along North Shore Drive, east of Wendy Avenue
    a. Replace existing 6-inch pipeline with 88 feet of 8-inch pipeline.
    b. Resolves insufficient fire flow for hydrants at model junction ID 705.

5.5.5.4 Improvements Driven by High Velocity under Maximum Day Demand

43. Velocity improvement at East Big Bear Boulevard and Shore Drive
    a. Replace approximately 55 feet of 6-inch pipeline with 8-inch pipeline

Optional Project:

44. Place PRVs on all homes with pressures higher than 120 psi or adjust the variable frequency drive (VFD) setting at the Peery pump station that operates at a lower head.

5.5.6 Summary

A summary of all pipeline improvements with pipeline lengths and diameters of each pipeline improvement is provided in Appendix G, which also includes maps highlighting the specific locations of the pipeline improvements discussed in Section 5.5.5.

The suggested improvements satisfy the flow, pressure, and velocity design criteria for all planning years with the exception of the areas of high elevation where fire flow could not be achieved by pipeline improvements. As discussed above, these areas will need to be analyzed further to verify the elevation and determine the improvement needed to satisfy the design criteria.

6. Design Criteria

The future water requirements discussed in Section 2 were based on full development of the BBCCSD service boundary, applicable General Plans, and future population estimates up to the year 2030, which is the limit of this study. Water facilities have been sized to deliver system demands plus fire flow requirements based on the land and water use discussed in Section 2. Should major changes be made in the type of land use, this Plan should be reviewed and revised to accommodate the changes. Table 6-1 summarizes the BBCCSD design criteria discussed in this section.

6.1 Ratios for Maximum Day Demand and Peak Hour Demand

The following ratios have been developed from BBCCSD's water use data from 2007 and are used to predict the MDD and PHD that the water system must be capable of supplying:

- Present MDD = 2.0 times present ADD
- Present PHD = 4.0 times present ADD
- Present minimum day demand = 0.4 times present ADD

6.2 General Water System Pressure Criteria

The level of service that is provided for domestic use is based on the available water pressure. Typical water industry design criteria have an ultimate goal of 40 to 120 psi for system pressures in distribution mains under normal operating conditions. Due to the terrain undulations within the community, this goal may not be achievable, but under no circumstances should the pressure in the system exceed the pressure class rating of the pipe. During minimum hour demands, when booster pumps are operating to refill reservoirs, pressures should not typically exceed 200 psi, as an ultimate goal, or the pressure rating of the pipe, whichever is lower.

For fire conditions, residual pressures should not fall below 20 psi, in accordance with the Big Bear City Fire Department requirements during a fire event. One exception is fire hydrants that are located so close to reservoirs as to not achieve the requirement for pressure residual. These hydrants shall be designated as “draft hydrants” and piping shall be sized from the reservoir to the hydrant to provide the fire flow requirement as close to the static pressure as possible.

Individual pressure regulators should be installed on any services that could have pressure greater than 80 psi at the meter as recommended in Section 1007(b) of the Uniform Plumbing Code. It is BBCCSD’s policy that customers should install these pressure regulators at their own expense. It is solely the customer’s responsibility to maintain these regulators.

6.3 Pipeline Design Criteria

Transmission mains should typically be sized in conjunction with the design of pumping plants to deliver MDD. Reservoir inlet-outlet lines should be designed for MDD+FF or PHD, whichever is greater. Transmission mains should also be sized such that the operating storage of reservoirs is replenished during minimum demand periods at night. Economical pump operation is an essential element of system design. Distribution mains should be sized to provide PHD or MDD+FF for the conditions anticipated. To minimize pumping costs, the maximum velocity in distribution pipe systems shall not exceed 5 feet per second (fps) and head loss shall not exceed 10 fps under MDD+FF conditions.

To provide adequate fire flows, the minimum water main pipeline diameter shall be 8 inches constructed anywhere in the system, except when system computer modeling proves that a 6-inch pipeline is adequate.

6.4 Supply Criteria

The total water production capacity from the sources of water supply must be capable of collectively meeting the MDD with the largest single BBCCSD source out of service. This standby capacity provides system reliability during the worst-case scenario. Sources of supply

should also be sufficient to refill fire and emergency storage within 48 hours under MDD conditions with all sources operating.

6.5 BBCCSD Adopted Storage Criteria

The principal functions of storage in BBCCSD water system are:

- To equalize fluctuations in hourly demand so that extreme and rapid variations in demands are not imposed on the source of supply
- To provide water for fire fighting
- To meet demand during an emergency, such a disruption of the major source of supply

Operational Storage. Operational storage is determined by fluctuations in hourly demand during peak summer demand periods. Peak demands in excess of the maximum daily average are typically supplied from storage. Reservoirs are refilled during off-peak hours when demand is below the maximum day average. The volume of operational storage, as an industry standard, averages between 20 and 30 percent of MDDs. As a conservative approach, the recommended operational storage for the BBCCSD should equal to 30 percent of MDD for all zones with storage.

Fire Storage. The water system must be capable of meeting MDD and fire fighting requirements simultaneously. BBCCSD's fire storage criterion was developed based on the recommendations of the Insurance Services Office, the County of San Bernardino Fire Prevention Office, and the Big Bear City Fire Department. Section 2 records the acres per zoning and land use category and the fire storage requirements observed by this water master plan (WMP).

Emergency Storage. Emergency storage is required to meet demands during times of planned and unplanned equipment outages such as pump breakdown, power failure, and pipeline rupture. Emergency storage is estimated based on the water supply to a pressure zone being

out of service for a period of 24 hours under MDD conditions. This duration is based on a review of potential supply sources and their respective outages.

Equivalent ADD Storage. Equivalent ADD storage is equivalent to 100 percent of the ADD storage requirement for 3 days, not including fire flow requirements. This criterion provides for operational storage for the system for 3 days during an emergency or planed shutdown of supply sources, including wells and spring water.

6.6 Booster Pumping Units

The existing BBCCSD service demands are met conjunctively by the booster pumping stations, reservoirs, and a small number of groundwater sources. Booster pumping plants should be sized to supply the MDD in pump and reservoir systems with the largest pump out of service and all groundwater sources turned off. In addition, the booster pumps and storage should be capable of meeting PHD.

Each pumping station should have a minimum of two pumping units of equal capacity, each sized to provide the MDD, so that service will remain uninterrupted in the event that one pump is not operational. Pumping stations, which consist of more than two units, should have adequate capacity to meet the MDD with the largest unit out of service. This criterion provides system reliability and flexibility.

Maintaining high overall pumping plant efficiency is a top priority for the BBCCSD water system. Over a 3-year period or sooner, all pumps in the BBCCSD system are to be tested. Any time a pump falls below 65 percent efficiency, analyses of operations shall be performed to examine the potential capital improvements to increase efficiency.

6.7 Pressure-Reducing Stations

In general, pressure-reducing stations should be provided when needed to supplement deliveries to lower pressure zones. Pressure-reducing stations should also be considered when distribution piping is operated near or above the maximum pressure rating of the pipe.

Pressure-reducing stations shall be sized to meet PHD or MDD+FF, whichever is greater, within the continuous flow rating of the valves. It is recommended that three valves be installed within each pressure-reducing station that is intended to feed a small closed pressure zone. Two smaller valves should be installed that, combined, can provide the MDD. One larger valve should be installed that can provide all flow required in the zone, including fire flow.

Pressure-reducing stations that are intended to support lower zones from the upper zones should be analyzed for their specific tasks and sized appropriately. Three valves, staggered to deliver the required flow and pressure, should be installed in this type of station.

6.8 Fire Protection

6.8.1 Minimum Fire Flow for Buildings

BBCCSD has identified fire flow requirements by zoning and land use category. These values are recorded in Section 2 of this WMP and are based on:

- The *2001 California Fire Code,* Appendix III-A (which is based on 2000 Uniform Fire Code and the Uniform Building Code)
- Discussions with the San Bernardino County Fire Prevention Office
- Discussions with the Big Bear City Fire Department

6.8.2 Minimum Fire Hydrant Spacing Requirement

The three sources listed in Section 6.8.1 were also consulted to determine the following fire hydrant spacing requirements.

- Single family residences and duplexes: Minimum number of 1 fire hydrant with average space of 600 feet.
- All other areas: Minimum number of 4 fire hydrants with average space of 300 feet.

- Hydrants should be of approved type and have no less than a 6-inch-diameter connection with water mains. Hydrants should be placed at least 100 feet from the protected building and along fire apparatus access roads and drives.

7. Existing System Analysis and Proposed Improvements for Near-Term and Ultimate Development

According to the San Bernardino County General Plan, the area in which BBCCSD currently serves its customers will experience slow but steady population growth through the year 2030. As a result, water demand is anticipated to increase by 45 percent, from a current ADD of 1,238 ac-ft/yr to an ultimate ADD of 1,796 ac-ft/yr based on the demand projections presented in Section 2. It is essential to compare the future total water demand and the associated water supply system capacity requirement with the capacities of the existing supply facilities based on the design criteria described in Section 6 so as to set up a step-by-step system improvement plan to ensure an adequate water supply system that will meet future demand.

Analyses are presented for three planning periods: (1) existing (2009), (2) near-term (2014), and (3) ultimate (2030). CIPs are recommended to resolve any deficiencies uncovered through the course of the various analyses. Generally, funding and implementation of any CIPs to resolve deficiencies found under existing conditions are assumed to be the responsibility of BBCCSD. CIPs for near-term and ultimate conditions are developer driven.

Groups of CIPs have been assigned letter designations for easier reference, as follows:

- A – Existing Supply Improvements
- B – Existing Booster Pump Improvements
- C – Existing Pipeline Improvements
- D – Meter Replacement Program
- E – Near-Term and Ultimate Reservoir Improvements
- F – Near-Term and Ultimate Booster Pump Improvements
- G – Near-Term and Ultimate Pipeline Improvements

There are frequently multiple solutions to any given engineering problem. The CIPs presented in this section were determined to be the best alternatives to the cited deficiencies. Basically, where a deficiency arises due to planned or predicted development that necessitates a new pressure zone, independent (gravity) storage alternatives were recommended. The smaller

BBCCSD pressure zones currently share storage with the Rowe Pressure Zone (making them dependent) and are supplied via hydropneumatic pump stations that create a false hydraulic grade line to meet demands at higher elevations. However, independent (gravity) storage offers greater system redundancy and lower operating costs. Note that the location and capacity of storage affects other facilities, including supply, booster pumps, and distribution pipelines.

7.1 Analyses of Existing Storage Facilities

This section considers groundwater production, fluoride blending, fluoride treatment, well pump efficiency, and potential contamination issues as a basis for recommending CIPs.

An analysis of the supply sources determined the adequacy of supply facilities as compared to the supply source design criteria defined in Section 6. BBCCSD currently uses supply from gravity slant wells and gravity springs in addition to 12 groundwater production wells (Well 5 is inactive and therefore not included in any supply calculations.). The existing sources provide 3.74 million gallons per day (mgd) of system capacity when all sources are used at full capacity (Table 7-1).

There are currently plans to replace Well 3 with a new well at the same site, Well 3B. Pilot well testing indicates that Well 3B has high production capability (1,050 gpm) but will also yield high fluoride concentrations (up to 7.5 mg/L). As a result, Well 3B can only be operated at 416 gpm to achieve compliance with the fluoride MCL after blending with the existing low-fluoride sources. In 2011, a new well (the Sugarloaf Well) will be in operation at BBLDWP at 300 gpm. Negotiations are currently underway to procure sufficient low-fluoride water from this well to maximize production from Well 3B. The data provided in Table 7-1 represent existing well production and planned production of the Sugarloaf Well and Well 3B (assuming that Well 3B production can be maximized under ultimate conditions).

As discussed in Section 6, the supply design criteria require successful verification of two components. First, supply sources must be collectively capable of serving the system MDD when the largest single source of supply is out of service. Second, supply sources must be capable of refilling fire and emergency storage within 48 hours under MDD conditions with all

sources in operation. An analysis of the existing, near-term, and ultimate supply against the design criteria (Table 7-2) indicates that there are no supply deficits for the existing, near-term, or ultimate planning periods.

The supply analysis assumes that existing production remains constant and does not take into account decreased well pump efficiency, changes to prevailing contamination levels, or other factors that may affect the availability of groundwater. For this reason, Civiltec recommends maximization of Well 3B production. One alternative is for BBCCSD to continue its pursuit of negotiating an agreement with BBLDWP to develop an interdistrict fluoride blending plan with mutual benefits to both agencies. Another alternative is fluoride treatment of water produced at Well 3B under ultimate conditions.

Sections 7.1.1 and 7.1.2 describe blending and treatment options for managing high-fluoride production from Well 3B. The CIPs recommended in these sections have been designated by letter as Existing Supply Improvements (A) and prioritized by number.

7.1.1 Blending

An analysis was performed on BBCCSD wells that discharge into the Restricted Main for fluoride blending. Table 7-3 lists the average and maximum 2008 fluoride concentration as well as the design flow for each of these wells. In the absence of historical data, the fluoride concentration of Well 3B was set at 7.5 mg/L in accordance with a water quality analysis conducted in 2000 by P. and Ch. Laboratory.

Given the current fluoride concentrations and well pump flow rates, a mass balance calculation was performed to determine the maximum flow rate at Well 3B to achieve a total fluoride concentration of 2.4 mg/L (80 percent of the target MCL of 3.0 mg/L). Assuming the fluoride concentration listed in Table 7-3, the maximum allowable production of Well 3B for the current Restricted Main configuration is 416 gpm, which is less than 40 percent of potential productivity (280 gpm, assuming maximum fluoride concentrations).

If an additional low-fluoride source is introduced into the Restricted Main, Well 3B production may be increased. With all sources in operation, a mass balance calculation shows that 1,418 gpm of water imported from BBLDWP is required to maximize Well 3B production while achieving a fluoride concentration of 2.4 mg/L, assuming average fluoride concentrations for all wells (1,722 gpm assuming maximum fluoride concentrations for all wells). Water quality documentation for BBLDWP indicates that production from the Sugarloaf Well has a fluoride concentration of 0.12 mg/L. With proper production management and assuming minimal fluctuation of contamination levels, the target range for importing water from BBLDWP to maximize Well 3B production should be between 1,418 and 1,722 gpm. However, this high import flow may not be practical. If continued monitoring of Well 3B reveals a reduction in fluoride concentration, then blending is a feasible option. A maximum fluoride concentration of 4.4 mg/L at Well 3B would assure a proper blend with the importation of only 300 gpm of low-fluoride water from the Sugarloaf Well into the Restricted Main.

An ancillary concern to maximizing well production is the capacity of the Restricted Main. Accordingly, the state of the Restricted Main when all wells are in production was analyzed, as described below.

BBCCSD prefers to operate all wells simultaneously during the blending process unless water quality monitoring indicates that the fluoride concentration is approaching the MCL. Since Well 8 has come online in 2008, no water quality issues have affected production.

Generally, excessive pressure in the Restricted Main may result in low well pump efficiency. To assure that the well pumps are not overworked, a maximum pipe velocity of 5 fps is recommended. Figure 7-1 depicts the Restricted Main system with pipeline segments and points of well discharge labeled.

Table 7-4 shows calculated pipe velocity and headloss within the existing Restricted Main with all sources in operation and recommends additional parallel pipes to reduce excessive head losses due to friction.

Capital Improvement Project A.1: Restricted Main Upgrade

The following improvements will reduce back pressure due to friction losses in the Restricted Main:

- Install approximately 3,200 feet of new 10-inch pipeline parallel to pipeline segments 2 and 4.
- Install approximately 700 feet of new 12-inch pipeline parallel to the pipeline segment 3.

7.1.2 Treatment

In 2006, SAE completed a conceptual design of a fluoride treatment facility intended to reduce the fluoride concentration of Well 3B production to 1 mg/L (SAE, 2006 [Appendix H]). The conceptual design drew heavily upon the design of an existing plant used by the 29 Palms Water District to cope with fluoride contamination of similar magnitude. Four alternatives were presented, differing primarily in the disposal method for backwash.

Capital Improvement Project A.2: Fluoride Treatment Plant

The estimated capital cost of the treatment plant construction is approximately $7 million based on the conceptual design detailed in Appendix H. A later conceptual design estimated the cost of the treatment facility at $2 million.

The well pumps were also analyzed for efficiency. A minimum efficiency rating was established to ensure that the well pumps operate in their most productive condition. This ensures that energy usage is appropriate for the service provided and that wells are able to operate at their highest capacity. Table 7-5 details the booster pumps that currently exhibit less than the required efficiency. The efficiency of Peery Booster 4 is unknown and has been included as a precautionary measure.

Capital Improvement Project A.3: Well Pump Rehabilitation

Improvement to the nine existing well pumps listed in Table 7-5 is warranted to ensure reliability of the system for current and future conditions based on their low efficiency ratings. The priority of replacement or maintenance is based on efficiency, with the lowest efficiency having the highest priority. Well pumps should be tested regularly for efficiency. Whenever the efficiency of a well pump falls below 65 percent, it becomes a candidate for rehabilitation.

Capital Improvement Project A.4: Well Contamination Study

Wells 1, 1B, and 2 are among the oldest production facilities in operation at BBCCSD. Although they are still productive and reliable, they do not have sanitary seals and therefore pose a potential health hazard. These wells are tested weekly for bacteria. A study should be conducted to determine the risk of bacterial contamination.

Capital Improvement Project A.5: Equip Well 3B

Figure 7-2 shows the Well 3B wellhead. This project would equip Well 3B with a pump and motor and other minor appurtenances (meter, SCADA control, valves, disinfection equipment, discharge pipeline, etc.) as necessary to begin production.

7.2 Analyses of Existing Storage Facilities

The principal functions of storage in any water system are (1) to equalize fluctuations in hourly demand so that extreme and rapid variations in demands are not imposed on the source of supply, (2) to provide water for fire fighting, and (3) to meet demand during an emergency such as disruption of the major source of supply.

The analysis of sufficient storage capacity described here used existing demands. The design criteria in Section 6 specify the system storage required to meet operational demands, fire flow, and emergency demands, called the “total required storage,” as follows:

- *Operational Storage:* Operational storage is determined by fluctuations in hourly demand during peak summer demand periods. Peak demands in excess of the maximum daily

average are typically supplied from storage. Reservoirs are refilled during off-peak hours when demand is below the maximum day average. The volume of operational storage, as an industry standard, averages between 20 and 30 percent of MDD. For this analysis, 30 percent of MDD has been adopted.

- *Fire Storage:* The water system must be capable of meeting MDD and fire fighting requirements simultaneously. The fire storage criterion is typically developed based on the recommendations of the Insurance Services Office, the County Fire Prevention Office, and the Fire Marshal. For this analysis, pressure zones containing only residential zoning were assigned 1,500 gpm for two hours and all other pressure zones were assigned 3,000 gpm for three hours.

- *Emergency Storage:* Emergency storage is required to meet demands during times of planned and unplanned equipment outages such as pump breakdown, power failure, and pipeline rupture. Emergency storage is estimated based on the water supply to a pressure zone being out of service for a period of 24 hours under MDD conditions.

- Equivalent ADD Storage. Additional design criteria specify that the entire system is capable of providing three days of storage under ADD conditions.

An analysis of the system storage indicates that BBCCSD has sufficient storage for existing demand requirements, and no additional storage is currently needed (Table 7-6). The existing tanks are in excellent condition and have recently been painted and equipped with cathodic protection to minimize corrosion.

The design criteria in Section 6 specify that the system has sufficient storage to meet operational demands, emergency demands, and fire flow. Table 7-6 illustrates that the total required storage provides a surplus for the system. Even though Table 7-6 indicates that the Rowe, Tanglewood, and Abbot Pressure Zones have a deficit, the total system has a surplus of 1.57 million gallons. While the analysis calculates each pressure zone individually, the BBCCSD system operates essentially as one zone. Sufficient gravity storage in each pressure zone would provide superior redundancy; however, storage capacity may also be supplied by

pumping, provided that adequate emergency power generation is maintained at each booster station serving areas without access to gravity storage. The latter is the case at BBCCSD, where the smaller pressure zones are fed by hydropneumatic pump stations. Therefore, the system storage of 6.24 million gallons provides a surplus of 1.57 million gallons to serve the demand of 1,238 ac-ft/yr, although only the Rowe Pressure Zone benefits from the redundancies associated with gravity storage. Should BBCCSD wish to construct additional gravity storage, the design volume for each pressure zone is indicated in Table 7-6 under the column heading "Total Required Storage." Gravity storage is recommended for any future pressure zones.

An additional design criterion in Section 6 specifies that the entire system be capable of providing three days of storage for the existing demands under ADD conditions. The intent of this design criterion is to provide storage in the system in case of a planned shutdown of well production. Table 7-6 illustrates that shared storage as a whole is deemed feasible at 5.65 days, meeting the ADD three-day storage criterion.

7.3 Analyses of Existing Pumping Facilities

As indicated in Section 6, there are two sets of design criteria for booster pumping stations: one set for pressure zones with gravity storage and the other for pressure zones without gravity storage. The Rowe Pressure Zone is in the first category. The Rowe Booster Pumping Station should be sized to supply MDD with the largest pump out of service.

For the other pressure zones, where gravity storage is not available, the booster pumping stations must exceed the greater of either PHD or MDD+FF with the largest pump out of service. However, as indicated in Section 4.2, several booster pumping stations servicing the smaller pressure zones are equipped with a fire pump specifically designed to meet the fire flow and pressure requirements as mandated by the fire marshal. These fire pumps have dedicated power supplies to deliver fire flow during emergencies. Therefore, although the design criterion in Section 6 requires analysis with the largest pump out of service, the analysis for the BBCCSD was conducted with the fire pump online and the largest of the remaining pumps taken out of service.

When recommending booster pump improvements, the following guidelines help to ensure compatibility with existing infrastructure and the potential to meet system requirements through the practical lifetime of the equipment:

- If no pressure zone reconfiguration is anticipated through the practical lifetime of the recommended equipment, calculate MDD using demands for ultimate conditions.

- If an existing fire pump is found to be deficient, recommend replacement with a sufficiently sized pump rather than an additional pump.

- Pumps of similar size work more efficiently in tandem. When recommending a new pump other than a fire pump, choose a size similar to an existing pump at the same booster station.

Table 7-7 summarizes the data used in the analysis, including whether they represent design data or results from pump efficiency tests. The term “design flow” refers to the target flow the pump was intended to deliver. The term “actual flow” refers to the test flow recorded during the latest pump efficiency test. Design flow tends to be more conservative than actual flow; however, such data were not available for all booster pumps.

An analysis of the system booster pumps using existing demands indicates that the Rowe and Peery Pressure Zones have sufficient booster pumping capacity, while all other pressure zones are deficient due to fire flow requirements. Table 7-6 delineates this analysis by pressure zone. The maximum flow per pressure zone is the summation of pump capacities provided in Table 7-7 minus the largest pump (excluding fire pumps for the smaller pressure zones).

For the Rowe Pressure Zone, MDD is calculated as 2.0 times the ADD as indicated in Section 2. For the smaller pressure zones, the greater of MDD+FF and PHD (4.0 times ADD [Section 2]) was used. In every case concerning the smaller pressure zones, the MDD+FF was greater (Table 7-8). A surplus or deficit was calculated as the difference between required and available flow.

For existing demands, Table 7-8 demonstrates that the Hicks, Abbott, and Greenspot Booster Stations are significantly deficient in capacity. The Tanglewood Booster Station shows a minor deficiency. The Abbott and Tanglewood deficiencies were addressed in conjunction with the establishment of the Cohila Pressure Zone (Section 7.6). To solve the Hicks and Greenspot deficiencies, replacement of or addition to existing booster pumps with sufficiently sized new booster pumps is recommended.

The following CIPs are recommended improvements to booster pumping facilities that will solve the most critical deficiencies. Projects have been designated by letter as Existing Booster Pump Improvements (B) and prioritized by number.

Capital Improvement Project B.1: Hicks Booster 2A

Replace existing Hicks Booster 2 with a new 1,300-gpm fire pump.

Capital Improvement Project B.2: Hicks Booster 3A

Install a new 250-gpm booster pump at the Hicks Booster Station.

Capital Improvement Project B.3: Greenspot Boosters

According to BBCCSD staff, the Greenspot Booster Station is in poor condition. Accordingly, all three existing pumps are to be replaced. Replace existing Greenspot Boosters 1 and 2 with new 240-gpm booster pumps. Replace existing Greenspot Booster 3 with a new 1,500-gpm fire pump.

Capital Improvement Project B.4: Booster Pump Rehabilitation

The booster pumps have also been analyzed for efficiency. A minimum efficiency rating was established to ensure that the booster pumps operate in their most productive condition. This ensures that energy usage is appropriate for the service provided and that booster stations are able to operate at their highest capacity. The design criterion in Section 6 states that pumps should maintain a minimum 65 percent overall efficiency. As detailed in Table 7-9, all the booster pumps currently exhibit less than the required efficiency. The efficiency of Peery Booster 4 is unknown and has been included as a precautionary measure.

Although Tanglewood Boosters 1, 2 and 3 have the lowest efficiency ratings, their rehabilitation priority is low due to minimal usage. Furthermore, establishment of the Cohila Pressure Zone as a developer-driven project will render the Tanglewood Booster Station obsolete (Section 7.6). Greenspot Booster 3 is recommended for replacement as part of CIP B.3. Extensive maintenance to this pump may not be warranted prior to replacement, provided it is in working order. Improvement to the other six pumps listed in Table 7-9 is warranted to ensure system reliability for current and future conditions based on their low efficiency ratings. The priority of replacement or maintenance is based on efficiency, with the lowest efficiency having the highest priority for replacement.

Booster pumps should be tested regularly for efficiency. Whenever the efficiency of a booster pump falls below 65 percent, it becomes a candidate for rehabilitation.

7.4 Analyses of Existing Pipeline Facilities

Recommendations for pipeline improvements represent the compilation of multiple analyses and observations as described below.

An analysis of pipeline deficiencies was performed using the water model to determine where improvements need to be made on a hydraulic basis. Among other things, the water model analyzed the impact of drawing fire flow from any hydrant represented in the database with respect to the design criteria established in Section 6. Through this effort, initial CIPs were recommended (Section 5) to improve pipeline reliability and efficiency while maintaining the system within the design criteria. The initial CIPs presented in Section 5 provide solutions to all hydraulic deficiencies uncovered during the water modeling process. However, based on needs of the BBCCSD that extend beyond hydraulics and additional information, the CIPs presented in this section include refinements based on the following additional information:

- A hydrant spacing analysis (Figure 7-3) was performed to determine compliance with the San Bernardino County Fire Code. CIPs were recommended to ensure proper hydrant spacing and appropriate infrastructure to service those hydrant locations. The BBCCSD includes 426 existing hydrants, and installation of 30 additional hydrants is

recommended. These hydrant recommendations are either incorporated into related pipeline CIPs or listed by location as detailed in this section and in Section 7.9.

- Other hydraulic issues observed through examination of the BBCCSD base map were also included in the development of CIPs.

  - Isolated fire hydrants on undersized mains are assumed to have met the San Bernardino County Fire Marshal's requirements for fire flow and residual pressure at the time of installation and have been "grandfathered in" as sufficient pending future development. Recommendations to provide current fire flow standards to such fire hydrants have been identified as developer-driven projects.

  - BBCCSD has taken great strides at implementing and maintaining a strong and comprehensive main line pipeline system. Enhancements were recommended to the main line pipeline system that close significant loops and extend service to more distant areas.

Based on these analyses and observations, along with those presented in Section 5, pipeline improvements were recommended and grouped into projects based on proximity. The pipeline projects were arranged in order of priority based on the number and magnitude of deficiencies resolved. The following recommended improvements to pipeline facilities will solve the most critical deficiencies. Included in each CIP is a justification for the project and a description of the improvements necessary to meet design criteria or other criteria as indicated. Projects have been designated by letter as Existing Pipeline Improvements (C) and prioritized by number. Each project has also been assigned a name to help describe its location.

Capital Improvement Project C.1: State Lane (Appendix I, Figure CIP.1C)

Justification. The water model returned 17 pressure deficiencies throughout the southwest portion of the Rowe Pressure Zone downstream of the existing 8-inch main in State Lane between Lakewood Drive and Willow Lane (Appendix G, IDM CIP #1). This pipe segment acts as a bottleneck. Future development to the east in the vicinity of State Lane requires a strong main line pipeline system in this corridor. Creating a loop between Lakewood Drive and Willow

Lane will improve low residual pressures and high pipe velocities as well as promote development in the area immediately south of State Lane, which is designated as medium-density single-family residential (RS-20M [Table 2-1]). This area has been identified for projected ultimate development as Area 13 in Figure 2-4.

Recommended Improvements. Install a new parallel 10-inch main in State Lane between Willow Lane and Cypress Lane. Provide jumpers between the parallel pipes at Fir Lane and Hemlock Lane to provide adequate flow to the hydrants at those locations. Install a new 10-inch main in the extension of Cedarpine Lane between Hatchery Drive and Willow Lane (if the extension of Cedarpine Lane proves to be infeasible, installation of a parallel 10-inch main in State Lane between Hatchery Drive and Willow Lane will satisfy the hydraulic requirements of this CIP).

Capital Improvement Project C.2: Michael Avenue (Appendix I, Figure CIP.2C)

Justification. The water model returned pressure and velocity deficiencies at 5 junctions in Pioneer Lane and Michael Avenue (Appendix G, IDM CIP #17). The hydrant spacing analysis indicates that additional hydrants are required at the intersection of Nana Avenue and Curly Drive and the intersection of Tiger Lily Drive and Tinker Bell Avenue. The current infrastructure cannot meet velocity design criteria at these locations. Numerous pipes in the vicinity are 2-inch steel pipes, which are recommended to be phased out due to age and condition issues.

Recommended Improvements. Replace the following existing mains with new 8-inch mains:

- Michael Avenue between Pioneer Lane and North Shore Drive
- Sequoia Avenue between Pioneer Lane and North Shore Drive
- Anita Avenue between Pioneer Lane and North Shore Drive
- Nana Avenue between Pioneer Lane and Cinderella Drive
- Peter Avenue between Cinderella Drive and North Shore Drive
- Tiger Lily Drive between Wendy Avenue and Peter Avenue
- Cinderella Drive between Nana Avenue and Michael Drive

Install new hydrants at the intersection of Nana Avenue and Curly Drive and the intersection of Tiger Lily Drive and Tinker Bell Avenue

Capital Improvement Project C.3: Cinderella Drive (Appendix I, Figure CIP.3C)

Justification. The water model returned eight velocity deficiencies in the south side of North Shore Drive between Sequoia Avenue and Paradise Way and in Cinderella Drive between Sequoia Avenue and Hugo Lane (Appendix G, IDM CIP #27). The hydrant spacing analysis indicates that an additional hydrant is required mid-block in Tiger Lily Drive between Hugo Lane and Pan Springs Lane.

Recommended Improvements. Install a new 8-inch jumper across North Shore Drive at Sequoia Avenue. Replace the following existing mains with new 8-inch mains:

- Sequoia Avenue between North Shore Drive and Cinderella Drive
- Cinderella Drive between Sequoia Avenue and Hugo Lane
- Tiger Lily Drive between Hugo Lane and Paradise Way
- Hugo Lane between Tiger Lily Drive and Cinderella Drive

Install a new jumper across North Shore Drive at Hugo Lane. Install a new hydrant mid-block in Tiger Lily Drive between Hugo Lane and Pan Springs Lane.

Capital Improvement Project C.4: Greenway Drive (Appendix I, Figure CIP.4C)

Justification. In accordance with the requirements of the Fire Marshal, hydrant spacing in a multi-family zone shall be 300 feet on center. The existing pipeline in Greenway Drive between North Shore Drive and Meadow Drive is undersized to deliver the required fire flow of 3,000 gpm to this multi-family zone.

Recommended Improvements. Replace the existing main in Greenway Drive between North Shore Drive and Meadow Drive with a new 12-inch main. Install hydrants at the intersection of Greenway Drive and Coy Lane, at the intersection of Greenway Drive and North Shore Drive, at 1041 Greenway Drive, and at 1021 Greenway Drive.

Capital Improvement Project C.5: Fern Lane (Appendix I, Figure CIP.5C)

Justification. The water model returned multiple pressure and velocity deficiencies throughout the eastern portion of the Greenspot Pressure Zone (Appendix G, IDM CIP Nos. 5, 14, 25). These deficiencies are caused by a bottleneck in Glencove Drive.

Recommended Improvements. Install a new 8-inch main in Fern Lane between Hatchery Drive and E Lane to create a loop. Install a new 8-inch jumper between the existing parallel 8-inch main and 4-inch main in Willow Lane in the vicinity of the hydrant at the intersection of Tuplin Lane and Willow Lane.

Capital Improvement Project C.6: Division Drive (Appendix I, Figure CIP.6C)

Justification. The water model returned pressure and velocity deficiencies at 15 junctions in the southwest corner of the service area bounded by West Big Bear Boulevard, Sugarloaf Boulevard, Division Drive, and Pineview Drive (Appendix G, IDM CIP #15). Pipes in this area are generally undersized to deliver fire flow. By observation, there are hydrants on 4-inch mains in Sugarloaf Boulevard, Robinhood Boulevard, and West Big Bear Boulevard that cannot meet pipe velocity design criteria. The hydrant spacing analysis indicates that additional hydrants are required at the intersection of Division Drive and Robinhood Boulevard and the intersection of Holcomb View Drive and West Big Bear Boulevard.

Recommended Improvements. Replace the following existing mains with new 8-inch mains:

- Division Drive between West Big Bear Boulevard and Sugarloaf Boulevard
- Sugarloaf Boulevard between Division Drive and Pineview Drive
- Robinhood Boulevard between Division Drive and Gildart Drive

Install new 8-inch mains in:

- Gildart Drive between Robinhood Boulevard and Rainbow Boulevard
- Hillen Dale Drive between West Big Bear Boulevard and Rainbow Boulevard

- Holcomb View Drive between West Big Bear Boulevard and Sugarloaf Boulevard

Install new hydrants at the intersection of Division Drive and Robinhood Boulevard and at the intersection of Holcomb View Drive and West Big Bear Boulevard.

Capital Improvement Project C.7: Maltby Boulevard (Appendix I, Figure CIP.7C)

Justification. The water model returned five velocity and pressure deficiencies in the vicinity of Maltby Boulevard and Shore Drive (Appendix G, IDM CIP #19). By observation, there are hydrants on the 4-inch main in Maltby Boulevard that cannot meet velocity design criteria. The hydrant spacing analysis indicates that additional hydrants are required at the intersection of Maltby Boulevard and Shore Drive, the intersection of Pintail Drive and Elysian Boulevard, and mid-block in Meadow Lane between Bluebill Drive and Shore Drive.

Recommended Improvements. Replace the existing mains in Maltby Boulevard between Bluebill Drive and Shore Drive and in Meadow Lane between Bluebill Drive and Shore Drive with new 8-inch mains. Extend the 8-inch main in Bluebill Drive between Elysian Boulevard and Maltby Boulevard parallel to the existing Restricted Main with a new 8-inch main. Extend the 8-inch main in Widgeon Drive between Elysian Boulevard and Maltby Boulevard with a new 8-inch main. Install new hydrants at the corner of Maltby Boulevard and Shore Drive, the intersection of Pintail Drive and Elysian Boulevard, and mid-block in Meadow Lane between Bluebill Drive and Shore Drive.

Capital Improvement Project C.8: Barrett Way (Appendix I, Figure CIP.8C)

Justification. The water model returned five pressure and velocity deficiencies in the vicinity of Irving Way and Berhardt Lane (Appendix G, IDM CIP #16). The hydrant spacing analysis indicates that an additional hydrant is required at the intersection of Shakespeare Lane and Barrett Way. By observation, there is a hydrant on the 4-inch main at the intersection of Garrick Way and Shakespeare Lane that cannot meet velocity design criteria.

Recommended Improvements. Replace the following existing mains with new 8-inch mains:

- Berhardt Lane between Big Bear Boulevard and Barrett Way

- Irving Way between Bernhardt Lane and the hydrant east of Bernhardt Lane
- Barrett Lane between Big Bear Boulevard and the hydrant east of Berhardt Lane
- Garrick Way between Paradise Way and Shakespeare Lane

Install a new 8-inch main in Shakespeare Lane between Garrick Way and Barrett Way. Install a new hydrant at the intersection of Shakespeare Lane and Barrett Way.

Capital Improvement Project C.9: Mountain View Boulevard (Appendix I, Figure CIP.9C)

Justification. The water model returned pressure and velocity deficiencies in the vicinity of Mountain View Boulevard and Mt. Doble Drive (Appendix G, IDM CIP #21).

Recommended Improvements. Replace the existing main in Mountain View Boulevard between Paradise Way and Mt. Doble Drive with a new 8-inch main. Replace the existing main in Mt. Doble Drive between Mountain View Boulevard and Barker Boulevard with a new 8-inch main.

Capital Improvement Project C.10: 2-inch Steel Pipe Replacement

The BBCCSD distribution system contains a number of 2-inch steel pipelines. These pipelines are among the oldest in the system. Pipe leaks, pipe breaks, and customer complaints about insufficient pressure can be linked directly to the low capacity, age, and condition of these pipelines. Table 7-10 lists alignments containing existing 2-inch pipelines totaling approximately 25,410 linear feet. The hydrant spacing analysis indicates that five of the alignments require additional hydrants. Also, two alignments run along back fence lines. These should be abandoned and services reconnected to new service mains in adjacent rights of way.

There is no urgency to initiate replacement of 2-inch steel pipes from a hydraulics point of view with the possible exception of improving hydrant spacing. However, BBCCSD must remain aware that any existing 2-inch mains are at risk and should be accounted for in improvement planning. Replace the alignments listed in Table 7-10 as needed.

7.5 Meter Replacement

Meters are mechanical devices and therefore require regular maintenance to ensure that they are functioning correctly. Two general types of meters are used by BBCCSD:

- Meters on customers' service connections
- Meters on production facilities

Water meters that are not maintained may measure more or less than the actual flow, so it is important to maintain meters regularly to ensure proper billing. Water loss has been steadily increasing at BBCCSD. A portion of water loss can be attributed directly to meter inaccuracy. Because accurately functioning meters assure that customers are being charged for the water they use, a meter maintenance program may increase the amount of revenue generated at BBCCSD. Such a program would likely pay for itself as a result of increased revenue.

It is recommended that BBCCSD initiate a meter maintenance program to evaluate its meters on a regular basis. The maintenance schedule depends on the type and size of the meter, the volume of use, the water quality, and the meter maintenance history. As the number of meters that are found to be reading accurately increases, the maintenance frequency can be reduced.

It is recommended that BBCCSD institute a meter replacement program with the goal of replacing all meters greater than 20 years old with new meters equipped with an automated meter reading (AMR) device. An AMR system would assist the BBCCSD in detecting leaks on the customer's side of the meter and would streamline billing data collection.

Typical costs associated with meter replacement are unit costs for the meter and AMR transmitter, meter installation, and an initial software setup cost. Prices for the options that follow reflect typical ⅝-inch and ¾-inch residential meters, which comprise about 97 percent of the meters serviced by BBCCSD. Three alternatives for replacement meters are recommended for consideration:

- Alternative 1: Severn Trent SmartMeter
  - Meter: unit cost of $120
  - AMR: unit cost of $70
  - Union installation fee: $90
  - Setup costs for drive-by reading hardware and software: $18,000

  Total estimated cost to replace 7,000 meters is about $1.98 million.

- Alternative 2: The B.E.S.T. Meter Company of West Covina, California prepared an estimate for installation of their Master Meter Fixed Network System tailored to the specific needs of BBCCSD. Depending on the specific options and redundancies preferred by BBCCSD, the total estimated cost ranges from $2.31 million to $2.47 million.

- Alternative 3: Aclara Star Fixed Network metering system
  - Meter: unit cost of $112
  - AMR: unit cost of $118
  - Union installation fee: $90
  - Setup costs for fixed network hardware and software: $35,000

  Total estimated cost to replace 7,000 meters is about $2.28 million.

Capital Improvement Project D: Meter Replacement Program

Replace meters by route over a predetermined period with a fixed network. See Appendix J regarding estimates for B.E.S.T. Meter and Aclara Fixed Network systems.

7.6 Cohila Pressure Zone

As discussed at the beginning of Section 7, gravity storage for new pressure zones will be assumed for developer-driven CIPs. One such opportunity has the potential to solve numerous interrelated deficiencies and requirements across all three planning periods. The Cohila Pressure Zone (Figure 7-4), named after Camp Cohila south of the terminus of Hillen Dale Drive, combines two small pressure zones, portions of the Rowe Pressure Zone, one near-term

development, and one ultimate development to be served by common booster pumping and storage facilities. These areas are:

- Abbott Pressure Zone (existing)
- Tanglewood Pressure Zone (existing)
- Saw Mill Creek Estates (near-term [Table 2-10])
- Area 12 (Hillen Dale) (ultimate [Table 2-11])
- Portions of Rowe Pressure Zone:
  - South of and including Mountain Lane between Hillen Dale Drive and Pineview Drive
  - Pineview Drive between Mountain Lane and Sugarloaf Boulevard
  - South of and including Rainbow Boulevard between Big Tree Drive and Saw Mill Drive
  - South of and including Aeroplane Boulevard between Saw Mill Drive and Kean Way
  - Kean Way between Aeroplane Boulevard and Bernhardt Lane
  - Bernhardt Lane between Kean Way and Booth Way
  - Booth Way between Shakespeare Lane and Adams Drive

The concept for the Cohila Pressure Zone is to plan required developer-driven improvements in a way that benefits future and existing customers while providing superior redundancy and lower operating costs. Figure 7-4 shows the general vicinity and potential extent of the Cohila Pressure Zone. The area in question lies to the south of the Rowe Pressure Zone between Division Drive and Shore Drive. Rather than trying to resolve deficiencies and anticipated new requirements on an individual basis, combining the areas detailed above into a single pressure zone will reduce the overall storage and booster pumping needs while raising deficient residual pressures in several areas. The Cohila Pressure Zone will have the following parameters and consist of the following components:

7.6.1 Calculations

7.6.1.1 Storage

Design criteria require adequate fire storage, operational storage, and emergency storage in each pressure zone in accordance with the following guidelines:

- Fire storage: 1,500 gpm for two hours (for residential areas)
- Operational storage: 30 percent of one day at MDD
- Emergency storage: one day at MDD

As shown in Table 7-11, the MDD for the combined areas is 494,671 gallons per day (gpd).

The storage (S) required for the new zone configuration is approximately 1.0 million gallons:

$$S_{total} = S_{fire} + S_{operational} + S_{emergency}$$

$$S_{total} = \left(\frac{1{,}500\ gal.}{min.}\right)\left(\frac{60\ min.}{hour}\right)(2\ hours) + 0.30(494{,}671\ gal.) + 494{,}671\ gal. = 823{,}072\ gal. \approx 1.0\ MG$$

7.6.1.2 Booster Pumping

Design criteria require sufficient capacity to supply the MDD with the largest pump out of service. MDD for the new zone configuration (Q) is approximately 400 gpm:

$$Q = \left(\frac{494{,}671\ gal.}{day}\right)\left(\frac{day}{24 \times 60\ min.}\right) = 343.5\ gpm \approx 400\ gpm$$

7.6.1.3 System Pressure

Design criteria suggest a normal system pressure range between 40 and 120 psi. Assuming that an elevated tank provides pressure to the new zone, the tank level may vary from 0 to 30 feet above the base elevation. A maximum elevation differential within the new pressure zone of approximately 150 feet will meet these design criteria:

$$\Delta Z = \frac{\Delta P}{\gamma_w} - \Delta D_{tank\ depth} = \left(\frac{(120-40)\,lbs}{inch^2}\right)\left(\frac{ft^3}{62.4\,lbs}\right)\left(\frac{12\,inch}{ft}\right)^2 - 30\,ft = 154.6\,ft \approx 150\,ft$$

Where ΔZ = elevation differential

ΔP = pressure differential

γ_w = density of water

ΔD = tank depth differential

By observation, the lowest elevation in the new pressure zone is approximately 6,800 feet above mean sea level (msl). Therefore, the highest elevation to be included in this new pressure zone should be limited to 6,950 feet msl:

$$Z_{high} = Z_{low} + \Delta Z = 6{,}800\,feet + 150\,feet = 6{,}950\,feet$$

Where Z = elevation above mean sea level

The elevation of the base of the tank should be sufficiently high to provide 40 psi at the highest service elevation within the zone. The base elevation of the tank is calculated at approximately 7,045 feet msl:

$$Z_{Tank} = Z_{high} + \frac{P_{min}}{\gamma_w} = 6{,}950\,feet + \left(\frac{40\,lbs}{inch^2}\right)\left(\frac{ft^3}{62.4\,lbs}\right)\left(\frac{12\,inch}{ft}\right)^2 = 7{,}042.3\,feet \approx 7{,}045\,feet$$

Where P_{min} = minimum pressure

7.6.1.4 Summary

Based on the calculations detailed in Sections 7.6.1.1 through 7.6.1.3, the conceptual design for the Cohila Pressure Zone includes:

- Two storage tanks with a combined capacity of 1.0 million gallons are to be located at an elevation of approximately 7,045 feet msl in the vicinity of Saw Mill Creek Estates.

- Three booster pumps (each capable of supplying 200 gpm at 135 feet of head) are to be located at the site of the Abbott Booster Station, the Tanglewood Booster Station, or a site to be determined by the developer. The Tanglewood Booster Station site is preferable due to the existing 12-inch intake pipeline. The other two sites would require an intake pipeline upgrade to accommodate the increase in flow due to the demands posed by the entirety of the Cohila Pressure Zone. It may be possible to continue to use the existing Abbott and Tanglewood Boosters to serve the Cohila Pressure Zone; however, it is unlikely that these pumps will be able to operate efficiently or provide adequate head. Abbott Booster 3 and Tanglewood Booster 3 will no longer be required to supply fire flow. The hydropneumatic tanks will no longer be required to dampen localized pressure fluctuations.

- A pressure zone boundary reconfiguration will provide adequate looping and connectivity among the areas to be incorporated. The reconfiguration will include new valve positions, jumpers, and additional pipelines to improve looping.

- Service connections presently served by the Rowe Pressure Zone that will be reconfigured as part of the Cohila Pressure Zone will experience a pressure increase of approximately 50 psi. This pressure change may increase the risk of failure of various fixtures and laterals at the service connections. Care has been taken to minimize the number of affected service connections; however, in the interest of improving residual pressure deficiencies and promoting greater system redundancy, a number of service connections may require an individual PRV to reduce pressure between the main and the residence. Similarly, service connections in the existing Abbott Pressure Zone will experience an increase of approximately 30 psi, and service connections in the existing Tanglewood Pressure Zone will experience an increase of approximately 20 psi. As discussed in Section 6.2, individual pressure regulators should be installed and maintained by the customer.

- The upper elevation limit for service connections within the Cohila Pressure Zone is 6,950 feet msl. The tracts identified as the near-term development of Saw Mill Creek Estates range in elevation from approximately 6,820 to 7,100 feet msl. Construction of

infrastructure to provide service to connections at elevations higher than 6,950 feet msl is the responsibility of the developer and is not included in this analysis. It is recommended that a hydraulic analysis of such infrastructure be conducted when the extent of the Saw Mill Creek Estates development is known.

7.6.2 Benefits

Elevated storage provides superior redundancy under emergency conditions. With elevated storage in place, fire flow would be supplied by gravity rather than by pumping. In the event of a catastrophic failure of supply or transmission due to an earthquake or other disaster, gravity storage will allow the Cohila Pressure Zone to operate independently of the rest of the BBCCSD system until affected elements are repaired or restored.

Improvements in the Cohila Pressure Zone would be largely developer driven. The near-term development of Sawmill Creek Estates, the ultimate development of Area 12 (Hillen Dale), and any further development in the vicinities of the Tanglewood and Abbott Pressure Zones would benefit directly from the establishment of the Cohila Pressure Zone.

By specifying multiple tanks and booster pumps, improvements may be staged over several years. One tank and two booster pumps will be sufficient for existing and near-term requirements, with additional improvements to be phased in only when demands warrant them.

7.6.3 Phasing

Establishment of the Cohila Pressure Zone will take place in two phases that coincide with development of Saw Mill Creek Estates and Area 12 (Hillen Dale), respectively. Phase 1 combines the Abbott and Tanglewood Pressure Zones, the Saw Mill Creek Estates development, and portions of the Rowe Pressure Zone between Adams Drive and Big Tree Drive. Phase 2 connects the Area 12 (Hillen Dale) development and a portion of the Rowe Pressure Zone north of Mountain Lane to the rest of the Cohila Pressure Zone.

Analyses of near-term facilities (Sections 7.7 through 7.9) assume that Phase 1 of the Cohila Pressure Zone has been adopted. Analyses of ultimate facilities (Sections 7.10 through 7.12) assume that Phase 2 of the Cohila Pressure Zone has been adopted.

7.6.3.1 Cohila Pressure Zone Phase 1

The costs for Phase 1 will be borne jointly by the developer of Saw Mill Creek Estates and BBCCSD. The developer will be responsible for 64 percent of the off-site costs, which represents 60 percent of the storage tank costs, half of the booster station costs, and all of the costs for the new transmission mains required to bring adequate supply to the area. BBCCSD will be responsible for 36 percent of the costs, which represents 40 percent of the storage tank costs, half of the booster station cost, and all of the costs for the pipelines required to reconfigure the Abbott and Tanglewood Pressure Zones.

Tasks to be completed during Phase 1 include:

- Construct a 0.6 million-gallon cylindrical steel reservoir (59-foot diameter and 32-foot depth) with a base elevation of 7,045 feet msl.
- Equip the Tanglewood Booster Station with two pumps, each with a capacity of 200 gpm at 135 feet of head.
- Install pipelines as indicated in Appendix I, Figure CIP.1G:
  - Replace existing 4-inch pipeline in Booth Way between Adams Drive and Bluebill Drive with new 8-inch pipeline.
  - Replace existing 4-inch pipeline in Booth Way between Sheridan Drive and Bernhardt Lane with new 8-inch pipeline.
  - Parallel existing 4-inch pipeline in Bernhardt Lane between Booth Way and Garrick Way with new 8-inch pipeline.
  - Parallel existing 4-inch pipeline in Garrick Way between Bernhardt Lane and Kean Way with new 8-inch pipeline.

- Replace existing 4-inch pipeline in Aeroplane Boulevard between Sawmill Drive and Paradise Way with new 8-inch pipeline.
- Replace existing 4-inch pipeline in Rainbow Boulevard between Big Tree Drive and Sawmill Drive with new 8-inch pipeline.
- Parallel existing 4-inch pipeline in Big Tree Drive between Rainbow Boulevard and Sugarloaf Boulevard with new 8-inch pipeline.
- Replace existing 2-inch and 4-inch pipeline in Sherwood Boulevard between Sawmill Drive and Paradise Way with new 12-inch pipeline.
- Replace existing 4-inch pipeline in Paradise Way between Sherwood Boulevard and East Big Bear Boulevard with new 12-inch pipeline.
- Install sufficient piping to connect reservoir to southern terminus of Woodbridge Drive and either Sawmill Drive or Gold Mountain Drive.

- Reposition valves. Existing gate valves that will be repositioned to remain closed normally, to serve as the boundary between the Rowe and Cohila Pressure Zones, are indicated in Appendix I, Figure CIP.1G. The following existing gate valves will be repositioned as normally open to promote connectivity within the Cohila Pressure Zone:
  - All valves at the intersection of Adams Drive and Barrymore Road
  - All valves at 512 Marlowe Drive
  - All valves at 532 Bernhardt Lane
  - All valves at the intersection of Kean Way and Beaumont Lane
  - All valves at the intersection of Rainbow Boulevard and Starr Drive
- Reconnect or install fire hydrants as indicated in Appendix I, Figure CIP.1G.
- Install PRVs on service connections as required. In addition to service connections on new pipelines listed above, service connections on the following streets will experience an increase in pressure of approximately 50 psi and may require individual PRVs to reduce pressure at the residence.

- Booth Way between Bluebill Drive and Sheridan Drive
- Marlowe Drive
- Sheridan Drive
- Bernhardt Lane south of Garrick Way
- Kean Way
- Shakespeare Lane
- Rainbow Boulevard east of Sawmill Drive
- Deerhorn Drive
- Sawmill Drive south of Aeroplane Boulevard
- Hilltop Lane
- Sugarloaf Boulevard east of Big Tree Drive

The cost of PRV installations on service connections is assumed to be borne by the affected customers and was not included as an existing or near-term cost.

7.6.3.2 Cohila Pressure Zone Phase 2

The costs for Phase 2 will be borne entirely by the developer of Area 12 (Hillen Dale). Tasks to be completed during this phase include:

- Construct a 0.4-million-gallon cylindrical steel reservoir (48-foot diameter and 32-foot depth) with a base elevation of 7,045 feet msl.
- Equip the Tanglewood Booster Station with an additional pump with a capacity of 200 gpm at 135 feet of head.
- Install pipelines as indicated in Appendix I, Figure CIP.2G:
  - Parallel existing 4-inch and 6-inch pipeline in Sugarloaf Boulevard between Big Tree Drive and Pineview Drive with new 12-inch pipeline. Install a new hydrant at Dawn Drive.

  - Parallel existing 8-inch pipeline in Pineview between Sugarloaf Boulevard and Mountain Lane with new 8-inch pipeline.
  - Install new 8-inch pipeline in Mountain Lane between Hillen Dale Drive and the end of the existing 6-inch pipeline to close the loop.
  - Install sufficient piping to connect reservoir to either the southern terminus of Hillen Dale Drive or the intersection of Raleigh Drive and Salem Drive.

- Reposition valves. Existing gate valves that will be repositioned to remain normally closed, to serve as the boundary between the Rowe and Cohila Pressure Zones, are indicated in Appendix I, Figure CIP.2G.

- Reconnect or install fire hydrants as indicated in Appendix I, Figure CIP.2G.

- Install PRVs on service connections as required. Service connections on the following streets will experience an increase in pressure of approximately 50 psi and may require individual PRVs to reduce pressure at the residence.
  - Mountain Lane
  - Salem Drive
  - Kingston Lane
  - Eton Lane
  - Pineview Drive south of Mountain Lane
  - Bedford Court
  - Wesley Court
  - Hillen Dale Drive south of Mountain Lane
  - London Lane
  - Dorset Drive
  - Belmont Drive
  - Raleigh Drive

The cost of PRV installations on service connections is assumed to be borne by the affected customers and was not included as an existing or near-term cost.

7.7 Analysis of Near-Term Storage Facilities

The analysis of sufficient storage capacity is described here using near-term demands. The design criteria in Section 6 specify the system storage required to meet operational demands, emergency demands, and fire flow (total required storage). Additional design criteria specify that the entire system is capable of providing three days of storage under ADD conditions (equivalent ADD three-day storage requirement). Section 7.1 defines storage terminology and explains the storage calculations.

The most significant change to near-term conditions is the Saw Mill Creek Estates development (Table 2-10). As stated in Section 7.2, gravity storage is recommended for any future pressure zones. The analysis that follows assumes that the Greenspot, Hicks, and Peery Pressure Zones will continue to be supplied by hydropneumatic pumping facilities that are dependent on storage in the Rowe Pressure Zone, and that gravity storage will be constructed as detailed in Section 7.6 to serve the Saw Mill Creek Estates development and adjacent areas as the Cohila Pressure Zone Phase 1 project.

Using near-term demands, the analysis of system storage presented in Table 7-12 indicates that BBCCSD has sufficient storage and that the system also meets the equivalent ADD three-day storage requirement, assuming that the Cohila Pressure Zone Phase 1 project is completed.

Capital Improvement Project E.1: Cohila Reservoir 1

The storage provision of the Cohila Pressure Zone Phase 1 project consists of construction of one 600,000-gallon reservoir within the Saw Mill Creek Estimates boundary at an elevation of 7,045 feet msl. Preliminary planning calculations indicate that a 600,000-gallon cylindrical steel tank will have a depth of 32 feet and a diameter of 59 feet (the depth includes 2 feet of freeboard). A reservoir siting investigation must be performed by the developer in order to plan the implementation of any additional necessary improvements. The site must be sufficiently

large to accommodate one 59-foot-diameter cylindrical steel tank and one 48-foot-diameter cylindrical steel tank with adequate access for maintenance vehicles and personnel. The second tank will be installed at a later date as part of the Cohila Pressure Zone Phase 2 project.

No additional storage is required beyond Cohila Reservoir 1 to meet near-term requirements. As shown in Table 7-12, the system storage of 6.84 million gallons provides a surplus of 2.18 million gallons to serve the demand of 1,309 ac-ft/yr. Shared storage within the system as a whole is deemed feasible at 5.85 days, which exceeds the ADD three-day storage criterion.

7.8 Analysis of Near-Term Pumping Facilities

The near-term pumping analysis assumes that the Cohila Pressure Zone Phase 1 project as detailed in Section 7.6 has been completed and that recommendations for existing deficiencies have been addressed.

The analysis of the system booster pumps using near-term demands presented in Table 7-13 indicates that no additional booster pumps are required. All pressure zones have surplus pumping capacity as shown in Table 7-13.

Capital Improvement Project F.1: Cohila Boosters, Phase 1

During Phase 1 of the Cohila Pressure Zone, replace the existing service pumps at the Tanglewood Booster Station with two new booster pumps, each with a capacity to deliver 200 gpm at 135 feet of head.

There may be an opportunity to continue using the existing booster pumps at the Tanglewood and Abbott Booster Stations after establishment of the Cohila Pressure Zone. Once more accurate parameters for the Saw Mill Creek Estates development are known, a study should be conducted to determine whether Abbott Boosters 1 and 2 and Tanglewood Boosters 1 and 2 have sufficient capacity to service the Cohila Pressure Zone. In accordance with Section 7.6, the Cohila Pressure Zone Phase 1 project indicates that the equivalent of two booster pumps, each capable of supplying 200 gpm at 135 feet of head, is required.

7.9 Analysis of Near-Term Pipeline Facilities

Near-term demands were applied to the water model assuming that CIPs for existing deficiencies have been implemented. Several minor deficiencies were found, largely in rural areas. Due to the remote locations of the deficiencies, no improvements are necessary until further development occurs in the area. This section lists recommended improvements to pipeline facilities that will solve these less critical deficiencies. Section 7.4 describes the analysis and the basis of its implementation.

Included in each CIP is a justification for the project and a description of the improvements necessary to meet design criteria. In addition to deficiencies uncovered during the analysis, this section includes necessary pipeline improvements for implementation of the Cohila Pressure Zone as detailed in Section 7.6.

Some neighborhoods are serviced by pipelines that were adequately sized for the fire flow demands in place at the time of their construction. The Fire Marshal has since increased the fire flow requirements, rendering the capacity of certain pipelines in these neighborhoods as deficient. However, the requirements at the time of construction are assumed to apply provided that no new construction takes place. In the event of further development, the developer assumes responsibility for upgrading the existing pipelines to meet the current standards.

Capital Improvement Project G.1: Cohila Pipelines, Phase 1 (Appendix I, Figure CIP.1G)

Justification. A new pressure zone needs to be established, as discussed in Section 7.6.

Recommended Improvements.

- Replace existing 4-inch pipeline in Booth Way between Adams Drive and Bluebill Drive with new 8-inch pipeline.
- Replace existing 4-inch pipeline in Booth Way between Sheridan Drive and Bernhardt Lane with new 8-inch pipeline.

- Parallel existing 4-inch pipeline in Bernhardt Lane between Booth Way and Garrick Way with new 8-inch pipeline.
- Parallel existing 4-inch pipeline in Garrick Way between Bernhardt Lane and Kean Way with new 8-inch pipeline.
- Replace existing 4-inch pipeline in Aeroplane Boulevard between Sawmill Drive and Paradise Way with new 8-inch pipeline.
- Replace existing 4-inch pipeline in Rainbow Boulevard between Big Tree Drive and Sawmill Drive with new 8-inch pipeline.
- Parallel existing 4-inch pipeline in Big Tree Drive between Rainbow Boulevard and Sugarloaf Boulevard with new 8-inch pipeline.
- Replace existing 2-inch and 4-inch pipeline in Sherwood Boulevard between Sawmill Drive and Paradise Way with new 12-inch pipeline.
- Replace existing 4-inch pipeline in Paradise Way between Sherwood Boulevard and East Big Bear Boulevard with new 12-inch pipeline.
- Install sufficient piping to connect Cohila Reservoir to the southern terminus of Woodbridge Drive and either Sawmill Drive or Gold Mountain Drive.

Capital Improvement Project G.2: Cohila Pipelines, Phase 2 (Appendix I, Figure CIP.2G)

Justification. Establishment of new pressure zone in accordance with Section 7.6 will have cost-saving and efficiency benefits as outline in Section 7.6.2.

Recommended Improvements.

- Parallel existing 4-inch and 6-inch pipeline in Sugarloaf Boulevard between Big Tree Drive and Pineview Drive with new 12-inch pipeline. Install new hydrant at Dawn Drive.

- Parallel existing 8-inch pipeline in Pineview Drive between Sugarloaf Boulevard and Mountain Lane with new 8-inch pipeline.
- Install new 8-inch pipeline in Mountain Lane between Hillen Dale Drive and the end of the existing 6-inch pipeline to close the loop.
- Install sufficient piping to connect Cohila Reservoir to either the southern terminus of Hillen Dale Drive or the intersection of Raleigh Drive and Salem Drive.

Capital Improvement Project G.3: Wendy Drive (Appendix I, Figure CIP.3G)

Justification. By observation, a new pipeline is required to complete a key segment of the main line pipeline system between the 12-inch main in North Shore Drive and the 12-inch main in Mountain View Boulevard.

Recommended Improvements. Install a parallel 12-inch pipeline in Wendy Drive between the southwest end of the 12-inch main in North Shore Drive and the 12-inch main in Meadow Lane. If the existing main in Wendy Drive is in poor condition, the new pipeline should replace the existing main instead of paralleling it.

Capital Improvement Project G.4: Big Bear Boulevard (Appendix I, Figure CIP.4G)

Justification. By observation, a new pipeline is required to complete a key segment of the main line pipeline system in East Big Bear Boulevard between Bernhardt Lane and Shore Drive.

Recommended Improvements. Install a parallel 10-inch pipeline in East Big Bear Boulevard between Bernhardt Lane and Shore Drive.

Capital Improvement Project G.5: Meadow Drive (Appendix I, Figure CIP.5G)

Justification. The water model returned multiple velocity deficiencies in Meadow Drive between Wendy Avenue and Sequoia Avenue (Appendix G, IDM CIP #28). Deficiencies in Meadow Drive west of Anita Avenue are less critical and will be alleviated as older 2-inch mains in Nana Avenue, Tinker Bell Avenue, Peter Avenue, and Michael Avenue are replaced (Section 7.4, CIP C.10). However, deficiencies in Meadow Drive east of Anita Avenue are more critical.

Recommended Improvements. Replace the existing main in Meadow Drive between the 12-inch main in Anita Avenue and the hydrant at the corner of Meadow Drive and Sequoia Avenue with a new 8-inch main.

Capital Improvement Project G.6: Hale Drive (Appendix I, Figure CIP.6G)

Justification. The water model returned pressure and velocity deficiencies in the vicinity of Holden Avenue and Hale Drive (Appendix G, IDM CIP #26). By creating a series of loops between existing mains in Van Dusen Canyon Road, Holden Avenue, and Wendy Drive, system performance is enhanced and redundancy is improved.

Recommended Improvements. Install a new 8-inch main in Hale Drive between Van Dusen Canyon Drive and Wendy Drive. Extend the existing main in Holden Avenue to Hale Drive with a new 8-inch main.

Capital Improvement Project G.7: Woodland Drive (Appendix I, Figure CIP.7G)

Justification. The hydrant spacing analysis indicates that hydrants are needed at the intersection of Woodland Drive and Cedarpine Lane and the intersection of Woodland Drive and Manzanita Lane. The water model confirmed that there were deficiencies in the area (Appendix G, IDM CIP #4). The current infrastructure cannot meet velocity design criteria at these locations.

Recommended Improvements. Install a new 8-inch main in Woodland Drive between Cedarpine Lane and Manzanita Lane to create a loop. Install new hydrants at the intersection of Woodland Drive and Cedarpine Lane and the intersection of Woodland Drive and Manzanita Lane.

Capital Improvement Project G.8: Arbor Lane (Appendix I, Figure CIP.8G)

Justification. The water model returned four velocity and pressure deficiencies in San Anselmo Drive and in the vicinity of Sequoia Avenue and Arbor Lane (Appendix G, IDM CIP #20).

Recommended Improvements. Replace the existing main in San Anselmo Drive between Paradise Way and the hydrant at 301 San Anselmo Drive with a new 8-inch main. Replace the existing main in Mt. Doble Drive between Arbor Lane and the hydrant at the north end of Mt. Doble Drive with a new 8-inch main. Replace the existing main in Arbor Lane between Mt. Doble Drive and Green Way Drive with a new 8-inch main.

Capital Improvement Project G.9: Fairway Boulevard (Appendix I, Figure CIP.9G)

Justification. The water model returned a velocity deficiency for the hydrant at the corner of Fairway Boulevard and Rosehill Drive (Appendix G, IDM CIP #34).

Recommended Improvements. Replace the existing main in Rosehill Drive between Fairway Boulevard and Country Club Boulevard with a new 8-inch main.

Capital Improvement Project G.10: Greenspot Road (Appendix I, Figure CIP.10G)

Justification. According to the water model, fire flow at the hydrant at the intersection of Greenspot Road and Mann Drive produces high velocities in the 4-inch main in Mann Drive (Appendix G, IDM CIP #29).

Recommended Improvements. Parallel the existing main in Mann Drive with a new 10-inch main between Murdoc Drive and Greenspot Road.

Capital Improvement Project G.11: Downey Drive (Appendix I, Figure CIP.11G)

Justification. By observation, the hydrant at 337 Downey Drive cannot meet velocity design criteria.

Recommended Improvements. Replace the existing main in Downey Drive between Dutch Way and Zaca Road with a new 8-inch main.

Capital Improvement Project G.12: Mariposa Lane (Appendix I, Figure CIP.12G)

Justification. By observation, the hydrant at 2190 Mariposa Lane and the hydrant at the corner of Hemlock Lane and Glencove Drive cannot meet velocity design criteria. The hydrant spacing

analysis indicates that three additional hydrants are required, at the intersection of Mariposa Lane and Central Lane, the intersection of Mariposa Lane and Hemlock Lane, and the intersection of Glencove Drive and Willow Lane.

Recommended Improvements. Install a new 8-inch main in Central Lane between Glencove Drive and Mariposa Lane. Replace the existing main in Mariposa Lane with a new 8-inch main. Replace the existing main in Hemlock Lane between Glencove Lane and Mariposa Lane with a new 8-inch main. Replace the existing main in Glencove Lane between Hemlock Lane and the existing 8-inch main in Willow Lane with a new 8-inch main. Install new hydrants at the intersection of Mariposa Lane and Central Lane, the intersection of Mariposa Lane and Hemlock Lane, and the intersection of Glencove Drive and Willow Lane.

Capital Improvement Project G.13: Malabar Way (Appendix I, Figure CIP.13G)

Justification. By observation, hydrants on Malabar Way between Greenspot Road and Dutch Way, on Turlock Drive, on Whipple Drive, and on Stanford Way between Turlock Drive and Greenspot Road cannot meet velocity design criteria.

Recommended Improvements. Replace the following existing mains with new 8-inch mains

- Malabar Way between Greenspot Road and Dutch Way.
- Turlock Drive
- Whipple Drive
- Stanford Drive between Whipple Drive and Turlock Drive

For greater redundancy (if feasible), install a new 8-inch main in the Walkway connecting Turlock Drive to Big Bear Boulevard at 1400 East Big Bear Boulevard.

Capital Improvement Project G.14: Mt. Doble Drive (Appendix I, Figure CIP.14G)

Justification. By observation, the hydrant at 913 Mt. Doble Drive cannot meet velocity design criteria. The hydrant spacing analysis indicates that there should be two hydrants in Mt. Doble Drive between Arbor Lane and Mountain View Boulevard.

Recommended Improvements. Replace the existing main in Mt. Doble Drive between Arbor Lane and Mountain View Boulevard with a new 8-inch main. Install two new hydrants trisecting the length of the new main.

Capital Improvement Project G.15: Monmouth Road (Appendix I, Figure CIP.15G)

Justification. The hydrant in Monmouth Road cannot achieve adequate pressure or fire flow due to the small diameter of the pipe in Monmouth Road and the high head loss experienced in the dead end pipe connecting it to the 8-inch main in Cascade Street via Ringwood Trail. The hydrant on the 8-inch main in Ringwood Trail meets design criteria for fire flow and appears to have adequate range to service the residences in the area.

Recommended Improvements. As development proceeds in the Ringwood Trail area, close the loop by extending the dead-end in Monmouth Road to Bramble Bush Trail with a new 8-inch main, and install a new 8-inch main in Bramble Bush Trail between Monmouth Road and Cascade Street.

Capital Improvement Project G.16: E Lane (Appendix I, Figure CIP.16G)

Justification. The water model returned pressure deficiencies in the vicinity of Fox Road and Willow Lane.

Recommended Improvements. Two potential solutions are proposed:

- *Short-term solution:* Install a new zone valve south of the hydrant in Willow Lane between Manzanita Lane and Fox Road (Willow hydrant). Open the existing zone valve north of Willow hydrant and close the new zone valve. This action will reconfigure Willow hydrant as part of the Rowe Pressure Zone. The combined flow between Willow hydrant and the hydrant in Chaparral Court will meet fire flow requirements for the immediate area.

- *Long-term solution:* Future development is anticipated along E Lane between Fox Road and Fern Lane. Developer-driven improvements will include installation of a new 8-inch main in E Lane between Fox Road and Fern Lane and installation of a new 8-inch main

in Fox Road between E Lane and Willow Lane. These improvements will improve pressure in the vicinity of Fox Road and Willow Lane, at which time the Willow hydrant may be reconfigured back to the Greenspot Pressure Zone. Install a new hydrant at the intersection of E Lane and Fox Road.

Capital Improvement Project G.17: Lakewood Drive (Appendix I, Figure CIP.17G)

Justification. The water model returned pressure deficiencies in Jensen Drive, Lakewood Drive, and Erwin Ranch Road (Appendix G, IDM CIP #5). By closing the loop in Lakewood Drive, these deficiencies are solved and greater redundancy is built into the system.

Recommended Improvements. Replace existing main in Lakewood Drive with a new 8-inch main. Install additional 8-inch pipe to close the loop in Lakewood Drive between Erwin Ranch Road and Jensen Drive.

Capital Improvement Project G.18: Commercial Zone Hydrants

Justification. Much of Big Bear Boulevard is zoned as commercial, which requires hydrant spacing of 300 feet on center. Hydrants are currently spaced 450 to 600 feet on center. The existing main is capable of delivering the required fire flow.

Recommended Improvements. Install new commercial hydrants mid-block at the flowing locations:

- 633 West Big Bear Boulevard
- 521 West Big Bear Boulevard
- 421 West Big Bear Boulevard
- 325 West Big Bear Boulevard
- 119 West Big Bear Boulevard
- 121 East Big Bear Boulevard
- 221 East Big Bear Boulevard
- 325 East Big Bear Boulevard

Capital Improvement Project G.19: Residential Hydrants

Justification. The hydrant spacing analysis revealed several gaps in the coverage of various residential areas that were not otherwise incorporated into the pipeline CIPs outlined in this section or in Section 7.4. Hydrants should be installed to improve spacing; however, no additional infrastructure is required to deliver adequate fire flow.

Recommended Improvements. Install new residential hydrants at the flowing locations:

- Intersection of Hicks Drive and Mound Street
- Intersection of Fairway Boulevard and Drake Avenue
- Intersection of Sugarloaf Boulevard and Pinon Drive
- 1080 Whispering Forest Drive

Capital Improvement Project G.20: Panamint Mountain Drive

Justification. Velocity criteria are exceeded in the pipe serving the hydrant at the western end of Panamint Mountain Drive. The Fire Marshal may allow for combining the fire flow with the hydrant at the intersection of Panamint Mountain Drive and Lookout Mountain Road.

Recommended Improvements. Inform the Fire Marshal that combined fire flow for the West Panamint Mountain Drive cul-de-sac is adequate for the given land use.

Capital Improvement Project G.21: Shady Lane

Justification. The water model returned a pressure deficiency at the hydrant at the west end of Shady Lane (Shady hydrant). Currently, the Shady hydrant is capable of delivering 1,414 gpm at 20 psi (86 gpm short of target for residential hydrants). The hydrant at the intersection of Shady Lane and Wilderness Drive (Wilderness hydrant) delivers 1,530 gpm at 20 psi. These two hydrants are approximately 250 feet apart, which is well within the fire department's spacing requirement of 600 feet. Due to its elevation, the Shady hydrant cannot achieve the required fire flow for an individual hydrant.

Recommended Improvements. Make the fire department aware that the flow available at Shady hydrant is 1,414 gpm and that Shady hydrant and Wilderness hydrant are to be used in tandem if a higher flow is required.

7.10 Analysis of Ultimate Storage Facilities

The analysis of sufficient storage capacity is described here using ultimate demands. The design criteria in Section 6 specify the system storage required to meet operational demands, emergency demands, and fire flow (total required storage). Additional design criteria specify that the entire system be capable of providing three days of storage under ADD conditions (equivalent ADD three-day storage requirement). Section 7.1 defines storage terminology and explains the storage calculations.

The most significant changes to ultimate conditions are the additions of Area 12 (Hillen Dale) and the Section 16 Pressure Zone. As stated in Section 7.2, gravity storage is recommended for any future pressure zones. The analysis that follows assumes that the Greenspot, Hicks, and Peery Pressure Zones will continue to be supplied by hydropneumatic pumping facilities that are dependent on storage in the Rowe Pressure Zone and that gravity storage will be constructed similarly to the Cohila Pressure Zone Phase 1 and Phase 2 projects.

Using ultimate demands, the analysis of system storage presented in Table 7-14 indicates that the BBCCSD has sufficient storage and that the system also meets the equivalent ADD three-day storage requirement, assuming that the Cohila Pressure Zone Phase 2 project is completed.

Capital Improvement Project E.2: Cohila Reservoir 2, Phase 2

The storage provision of the Cohila Pressure Zone Phase 2 project consists of construction of one 400,000-gallon reservoir adjacent to Cohila Reservoir 1 at an elevation of 7,045 feet msl. Preliminary planning calculations indicate that a 400,000-gallon cylindrical steel tank will have a depth of 32 feet and a diameter of 48 feet. The depth includes 2 feet of freeboard.

Capital Improvement Project E.3: Section 16 Reservoirs

To serve the demands of the Section 16 Pressure Zone, which currently has a storage deficit, construct two 250,000-gallon reservoirs. A reservoir siting investigation must be performed by the developer in order to plan the implementation of any additional necessary improvements. Preliminary planning calculations indicate that each 250,000-gallon cylindrical tank at a depth of 24 feet will require a 44-foot-diameter footprint.

No additional storage is required beyond Cohila Reservoir 2 and the Section 16 Reservoirs. As shown in Table 7-14, the system storage of 7.24 million gallons provides a surplus of 1.67 million gallons to serve the demand of 1,620 ac-ft/yr. Shared storage within the system as a whole is deemed feasible at 5.01 days, which exceeds the ADD three-day storage criterion.

7.11 Analysis of Ultimate Pumping Facilities

The ultimate pumping analysis assumes that the Cohila Pressure Zone Phase 2 project as detailed in Section 7.6 has been completed and that recommendations for existing and near-term deficiencies have been addressed.

The analysis of the system booster pumps using ultimate demands, presented in Table 7-15, indicates that additional booster pumps are required for the Section 16 Pressure Zone. All other pressure zones have surplus pumping capacity (Table 7-15).

Capital Improvement Project F.2 Cohila Booster, Phase 2

The booster pumping provision of the Cohila Pressure Zone Phase 2 project consists of installation of a new booster pump at the Tanglewood Booster Station with a capacity to deliver 200 gpm at 135 feet of head.

Capital Improvement Project F.3: Section 16 Booster Station

To solve the Section 16 Pressure Zone booster deficiency, install a booster pump station to serve the Section 16 Pressure Zone with three 80-gpm pumps.

7.12 Analysis of Ultimate Pipeline Facilities

Ultimate demands were applied to the water model assuming that CIPs for existing and near-term deficiencies had been implemented. No additional deficiencies were found.

7.13 Time-Phased Improvements and Expenditures

The improvements described herein include certain operational and maintenance items, as well as new requirements identified in this investigation. Costs for each proposed CIP were estimated, and each project was scheduled for construction based on the perceived urgency. Total cost for pipeline installation and replacement was estimated at an appropriate rate to account for the current cost for materials and labor. The costs for reservoir construction were estimated at $1.25 per gallon of storage, assuming that steel construction is used. This value does not include costs for extensive site improvements or for the acquisition of property or rights of way. Cost escalation is assumed to be 4 percent per year.

The total cost has been estimated at 35 percent above construction cost, including:

- 25 percent engineering design and construction management cost
- 10 percent contingencies

The schedules for the identified improvements are provided in Table 7-16 so that BBCCSD management can prioritize expenditures based on the availability of funding. Market conditions, detailed design, and the extent to which designs are performed by consultants will determine actual cost.

Projections were made on a yearly basis and extended across multiple years to account for anticipated project duration. The current BBCCSD operating budget was used as a guide for scheduling and is provided as Appendix K. The total estimate for the 22-year CIP plan (projecting out to year 2030) to remedy existing deficiencies is $16,284,000.

Developer-driven improvements anticipated for near-term and ultimate conditions have been identified in Table 7-17, and costs have been estimated in 2008 dollars. It is anticipated that an additional $9,681,000 will be necessary to accommodate growth that is development- and developer-driven through 2030.

CIP improvements were prioritized to allocate greatest importance to those facilities that were deemed to be the most critical to the BBCCSD infrastructure:

- Supply facilities are considered to be the most integral to sustain growth and maintain a steady and redundant source for demand.

- Pump stations are considered secondary in importance due to the volume of water demand and varying undulation in topography within the BBCCSD service area. Within the pump station category, stations were prioritized based on need for immediate increases in capacity and upkeep of the existing facilities.

- Storage capacity through reservoir availability is considered third most important to ensure that sufficient water is available to the respective pressure zones.

- Programmed pipeline replacements follow in importance to the storage. Operations personnel play an integral role in determining system conditions based on field experience. Priority within this category has been determined based on interviews with these personnel and incorporating their professional opinion into pipeline replacement needs. Also, projects that solved multiple deficiencies were given priority over smaller projects that dealt with isolated and less critical issues. Figure 7-5 shows the locations of all pipeline CIPs. Appendix I provides maps of individual pipeline CIPs, organized by designation as indicated in Sections 7.4 and 7.9, showing the proposed alignments and locations of proposed fire hydrants.

8. Methods of Financing

The development of funding plans is based on an agency's unique set of financial conditions. This section identifies some of the issues that BBCCSD should consider in the development of a capital improvement financing plan. Many projects are financed by a combination of resources and funding methodologies. Some of the more common financing techniques, including common types of debt instruments applicable in this case, are reviewed in Sections 8.1 through 8.8.

8.1 Pay-As-You-Go

The accumulation of current funds for improvement projects is referred to as "pay-as-you-go" financing. This method of financing requires that an agency accumulate the total capital cost of the improvements in advance of the start of construction. Because several years may be required to accumulate the necessary funds for large projects, this method of funding is ideally applicable to smaller projects.

Pay-as-you-go financing eliminates interest costs. On the down side, current customers may pay for facilities that benefit future users, and system expansion or projects to increase the capacity of existing facilities to serve new users that are funded by pay-as-you-go financing are not equitable. Projects to modernize or otherwise improve an existing system are appropriate for this method of financing. The pay-as-you-go method requires matching of needed improvements to water revenues and an adequate water rate structure to ensure proper phasing of improvements.

8.2 Pay-As-You-Use

Capital items with a long useful life can be financed over the life of the project on a "pay-as-you-use" basis using debt instruments. The term of borrowing should coincide with the estimated useful life of the improvements, if market conditions permit and the debt obligation is within the community's ability to pay.

8.3 State Revolving Fund

The Drinking Water State Revolving Fund (DWSRF) program is an innovative method of financing for state water programs to further the goals of the Safe Drinking Water Act (SDWA). Under this program, the U.S. Environmental Protection Agency (EPA) provides grants to states to set up DWSRF programs. These programs consist of federal and state matching funds placed in a revolving loan fund to finance low-interest loans for construction of eligible water projects, which address the public health and compliance priorities of SDWA. Additionally, states can set aside federal funds from each grant to be used for state water program items and/or for direct assistance to water systems, subject to specified maximum limits for each activity.

Provisions in the SDWA require that at least 15 percent of the loan fund be used for the direct benefit of small systems. Other provisions allow for subsidization of loans for economically disadvantaged communities. A unique provision of the DWSRF requires that communities not receive a loan unless they can either demonstrate the technical, managerial, and financial capacity to comply with the SDWA or agree to make the changes necessary to come into compliance.

8.4 Water Bond 2000 (Proposition 13)

The Water Bond 2000 measure, Proposition 13 approved March 2000, provides loan and grant funding for urban and agricultural water conservation, groundwater recharge and storage, and infrastructure rehabilitation projects or feasibility studies. The Department of Water Resources (DWR) is developing administrative regulations, policies, and applications for these new programs, which include:

- *Groundwater Storage Program ($200 million)*: Provides money for grants to public agencies and mutual water companies to fund feasibility studies, project design, or the construction of facilities for conjunctive use projects.

- *Interim Water Reliable Supply and Water Quality Infrastructure and Management Program ($180 million)*: Provides grants and/or loans to local agencies in the Delta export service areas for programs and projects designed to increase water supplies, enhance water supply reliability, or improve water quality.

8.5 Contributions, Connection Fees, and Developer Impact Fees

Several methods of obtaining contributions to fund improvement projects exist, such as:

- Cash fees paid by developers to expedite construction of certain facilities
- Financing by assessment district (Section 8.6) property owners for system improvements
- Construction and dedication of facilities to BBCCSD by developers
- Connection and developer impact fees

Growth may be relatively aggressive at times, and at such times BBCCSD may also have capital needs for replacing facilities. Contributions may not provide adequate revenue to meet requirements for replacement of existing facilities.

Connection fees are often assessed on a lot-by-lot basis to offset the cost of connecting to an existing system and buying capacity in an existing system that has been previously constructed and paid for by the water purveyor. The logic involved is that when a water system is initially constructed by a city or water district, good planning dictates that the system be designed and constructed to provide for future growth and extension of that system. Thus, some main line elements of that system are oversized relative to the initial water conveyance requirements. It is then reasonable that as future extensions and connections to that system are made, the individual lot owner or developer should pay a connection fee or development impact fee to offset a fairly determined portion of the initial system cost.

In the case of BBCCSD, many of the recommended improvements are, in part, required to enhance the delivery capability of the system to fill-in lot development, to near-term planned developments such as Saw Mill Creek Estates, Wooden Door Estates, and Big Bear Pines, and

to development of currently undeveloped areas zoned as residential and commercial. The projected new connections will represent approximately 32 percent of the total system connections (Table 8-1). It is therefore reasonable that fees from these new connections should pay for an absolute minimum of 32 percent of the facilities improvement cost plus a buy-in assessment of 32 percent of the existing main line system elements. These main line elements include wells, reservoirs, treatment facilities, booster pumping stations, and main line pipelines.

The developer of any new development should pay for all new facilities required for that development. The developer should also pay a connection fee (Table 8-1) to help fund facilities that were previously designed and installed that provide the capability for the main line system to convey an adequate supply of water to that new development. Such new facilities must be approved by BBCCSD and satisfy the requirements of the Fire Marshal. Acquired funding in excess of the amount necessary to construct the new facilities required for the development should go into a water facility improvement fund for future repairs and improvements.

8.6 Revenue Bonds (1911 and 1915 Acts)

Revenue bonds are used to finance capital infrastructure that is revenue producing. Revenue bonds are special obligations of the issuing entity with repayment made solely from the revenues produced by the constructed infrastructure. Revenues derived from the constructed facilities must normally also be sufficient to cover the cost of maintaining and operating the facility. Thus bond covenants generally pledge that net revenues will be equal to an amount sufficient to meet all repayment and expense obligations plus an operating margin or coverage that typically varies from about 1.2 to 1.5 times the amount of the debt service. Coverage margins typically reflect the source of the loan as well as the financial characteristics and credit worthiness of the issuing agency. Water system facilities are typically financed with the use of revenue bonds in accordance with the Revenue Bond Law of 1941. In accordance with this act, an election must be held with a majority of the voters at the election approving the revenue bond issue.

Since the passage of Proposition 13 limiting the allowable increases in annual property taxes, the creation of assessment districts, in existence since the early 1900s, has been a popular

alternative method of financing public infrastructure. Approximately one-third of privately owned property in California is included within an assessment district. Assessment districts are created in accordance with either the Improvement Act of 1911 or the Municipal Improvement Act of 1913. The former act can also be used to fund improvement maintenance. These two acts set forth the procedures for implementing an improvement project and for levying the assessment to pay for such work. Assessment bonds to fund capital improvements can be issued by assessment districts in accordance with associated assessment bond acts.

The Improvement Act of 1911 provides for authorization to levy assessments and issue related bonds. The Improvement Act of 1913, however, has no bond procedures, but improvements can be financed through a subsequent bond act known as the Improvement Bond Act of 1915 (solely a bond act). These acts may be used in various combinations. There may be a 1911 act assessment with a 1911 or 1915 act bond, or a 1913 act assessment with a 1911 act or 1915 act bond. However, there is no such thing as a 1913 act bond or a 1915 act assessment.

An assessment district is created by a local sponsoring governmental agency. Property owners typically initiate creation of the assessment district by circulating a petition that must be signed by property owners representing 60 percent of the benefited land area. It is essential that properties within the assessment district, which will bear the burden of tax levies to pay for the bond financings, receive a direct and special benefit (as distinguished from general benefits obtained by the community as a whole). Following the creation of the assessment district, bonds can be approved by the governing board only after the preparation of an Engineer's Report and at the conclusion of a public hearing. In accordance with Proposition 13, the property assessment cannot be based directly on the value of each property, but rather on a mathematical formula that takes into account how much each property will benefit from the constructed infrastructure. Each parcel in the assessment district is obligated for a fixed percentage of the total district debt and will be assessed each year for that portion of the annual debt service.

8.7 State Grants (Propositions 50 and 84)

The State of California has one current grant program available to finance water infrastructure. Proposition 50 (known as the Water Security, Clean Drinking Water, Coastal and Beach Protection Act of 2002) authorized the legislature to appropriate funds for Integrated Regional Water Management (IRWM) projects. This grant program is jointly administered by the DWR and State Water Resources Control Board.

Chapter 8 of the act provides about $380 million for IRWM projects. Grant funding is available to local agencies for projects that are formulated through an integrated regional planning process. Funding is available for implementation of regional projects (Implementation Grant) and to support development of IRWM plans (IRWMPs). Two cycles of funding were being used to select projects for funding. The first cycle submittal deadline is past, leaving only on the order of $200 million available for cycle two funding. To qualify for Implementation Grant funding, the proposed projects need to be the outcome of an IRWMP. Only one Implementation Grant application will be accepted from each region. However, each grant application may request funding for more than one project. Each grant is limited to a maximum of $50 million. This source of grant funding is probably not feasible for BBCCSD's CIP implementation, as projects most likely to be funded should be related to regional rather than local projects.

The second phase of IRWM project funding may not occur, as some of the remaining money could be used to fund cycle one projects not initially selected for funding. Any remaining money would be transferred to the recently passed Proposition 84 program. Limited funds may be available through the Proposition 84 program for BBCCSD's CIP; however, the program details have not yet been fully established.

8.8 Certificates of Participation

In lieu of funding capital improvement projects on a pay-as-you-go basis or by assessment proceedings, BBCCSD may use a financing vehicle referred to as certificates of participation (COPs). COPs were first developed in the late 1970s in response to Proposition 13, which limited an agency's ability to finance services and capital expenditures through property tax

increases and general obligation debt. Proposition 13 prohibits agencies from incurring bonded indebtedness without voter approval. However, agencies can enter into long-term lease obligations in lieu of bonded debt. The COP structure is an extension of a long-term lease sold as a tax exempt investment in the capital markets.

Agencies typically use two types of COP structures:

- Under the first alternative, BBCCSD would issue a COP that directly finances certain capital improvements and would enter into a long-term lease purchase agreement with a nonprofit lessor. The agreement would require BBCCSD to lease back the improvements at an annual cost equal to the annual debt service on the COP. Upon redemption of the COP, BBCCSD would receive title of the financed capital improvements.

- The second alternative involves the mortgaging or refinancing of publicly owned land and improvements. The proceeds from the mortgage can fund projects including land acquisition, new facilities construction, capital improvements, and equipment purchases. This COP structure, known as the Asset Transfer Program (ATP), would enable BBCCSD to pledge its equity in existing real property for financing new projects. Under this method, BBCCSD would sell an existing facility to a nonprofit corporation (typically comprised of members from the governing body) and would enter into a long-term lease purchase agreement at an annual cost equal to the debt service on the COP that initially financed the purchase of the facility. Upon redemption of the COP, title to the refinanced facility would revert to BBCCSD at no additional cost

The primary security for repayment of a COP is BBCCSD's pledge to establish rates and charges that will produce revenues, on an annual basis, in an amount sufficient to support its annual lease obligation to the nonprofit corporation.

The ATP can provide at least four notable advantages over more conventional forms of tax exempt financing and a direct COP:

1. By borrowing against the appreciated value of existing property and improvements, BBCCSD would provide investors with greater security, in the form of real property and existing improvements, as compared to a simple pledge of proposed capital projects.

2. By refinancing existing facilities, BBCCSD would eliminate the need to capitalize interest during construction, thereby reducing the total size of the issue and substantially decreasing BBCCSD's overall cost of borrowing.

3. By eliminating the need for a conditional or provisional rating to be assigned, the marketability of the COP would improve.

4. Water enterprise funds spent on capital improvements without the availability of ATP proceeds would be invested without any arbitrage restrictions.

A COP can provide an agency with a flexible, cost-effective vehicle to finance capital improvement programs or leverage existing facilities. Additionally, BBCCSD may choose to finance capital projects with a COP in lieu of enterprise funds and invest a like amount of money without any arbitrage restrictions. Each of these structures can satisfy BBCCSD's goal of providing capital improvements at the lowest cost to its customers.

8.9 Statement of Existing Rate Structure

An extensive water rate review was conducted by Bartle Wells Associates in 1995 and is provided as Appendix L. The review concluded that the rate structure in place in 1995 was equitable and that district revenues were adequate to fund ongoing operating and capital expenses. The review recommended an increase in standby charges on undeveloped parcels and standardization of tier breakpoints to ensure that all district customers would be charged on the same basis.

BBCCSD produces a water service operating budget (WSOB) annually. The WSOB for fiscal year 2008-2009 is included in Appendix K. This WSOB summarizes revenue and reserve

transfers, operations and maintenance appropriations, debt service, and a 5-year capital improvement program. Revenue is generated according to the following rate structure:

- The minimum base rate for residential water service, irrespective of meter size or consumption, is a bimonthly fee of $40.04. The base rate increases per meter size are indicated in Table 8-2.

- Bimonthly quantity rates are based on a tiered structure that accounts for meter size. All quantity rates are based on a unit cost of dollars per 100 cubic feet (Table 8-3).

- Owners of improved parcels receiving water service pay an additional annual fee of $30 that is included on their property tax bill.

- Owners of unimproved parcels where water service is available pay an annual fee of $40 that is included on their property tax bill.

8.10 Recommendations

BBCCSD should conduct a water rate structure analysis and financing plan study before pursuing the construction of recommended improvements. The rate analysis and financing plan study can assist BBCCSD in determining the most cost-effective and feasible funding approach for construction projects. The pay-as-you-go method, along with special bond financing, remains the most feasible method.

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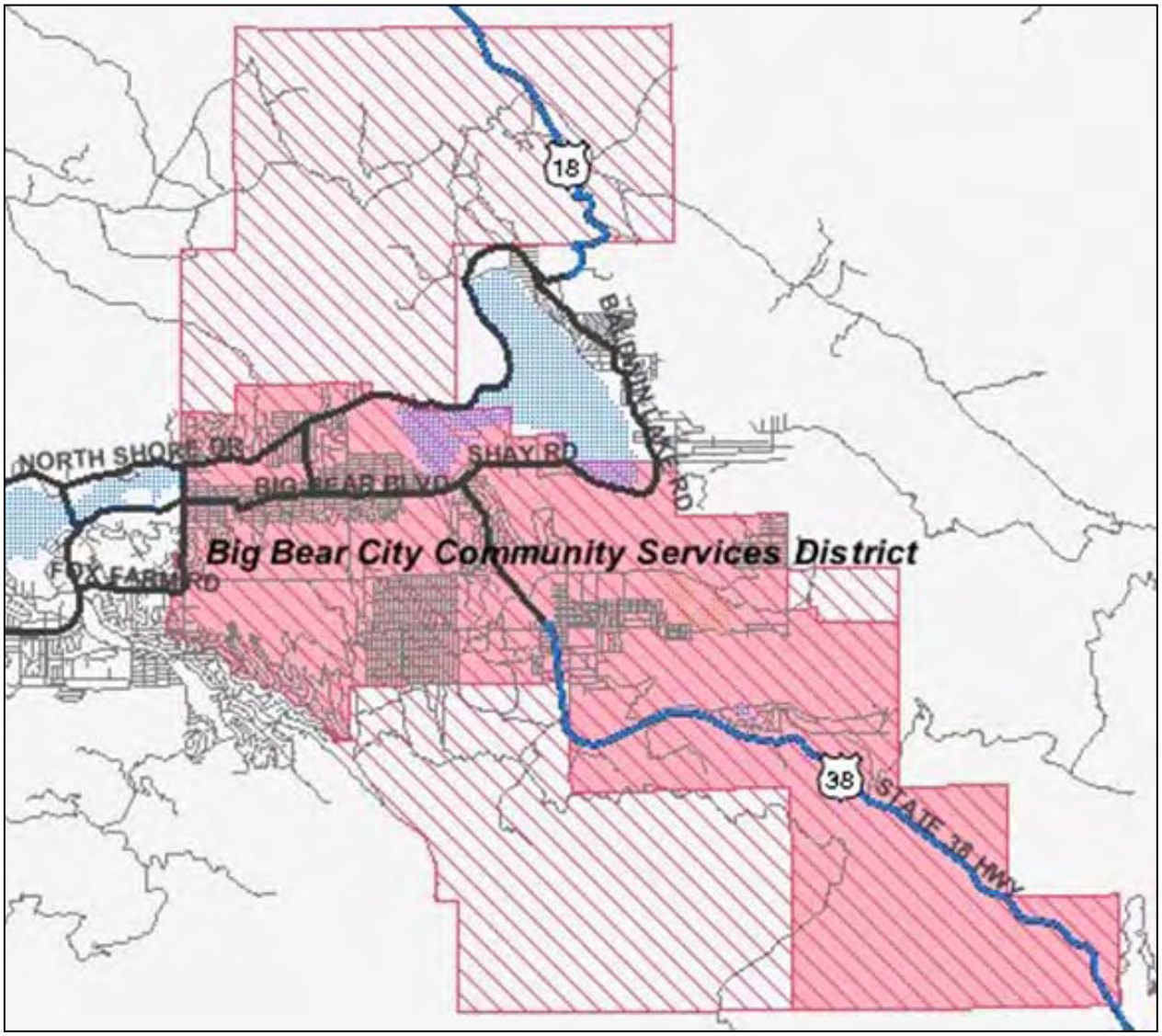
Figures



	Big Bear City Community Services District
	Big Bear City Community Services District Sphere

Source: San Bernardino Local Agency Formation Commission

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BIG BEAR CITY COMMUNITY SERVICES DISTRICT

Sphere of Influence Map

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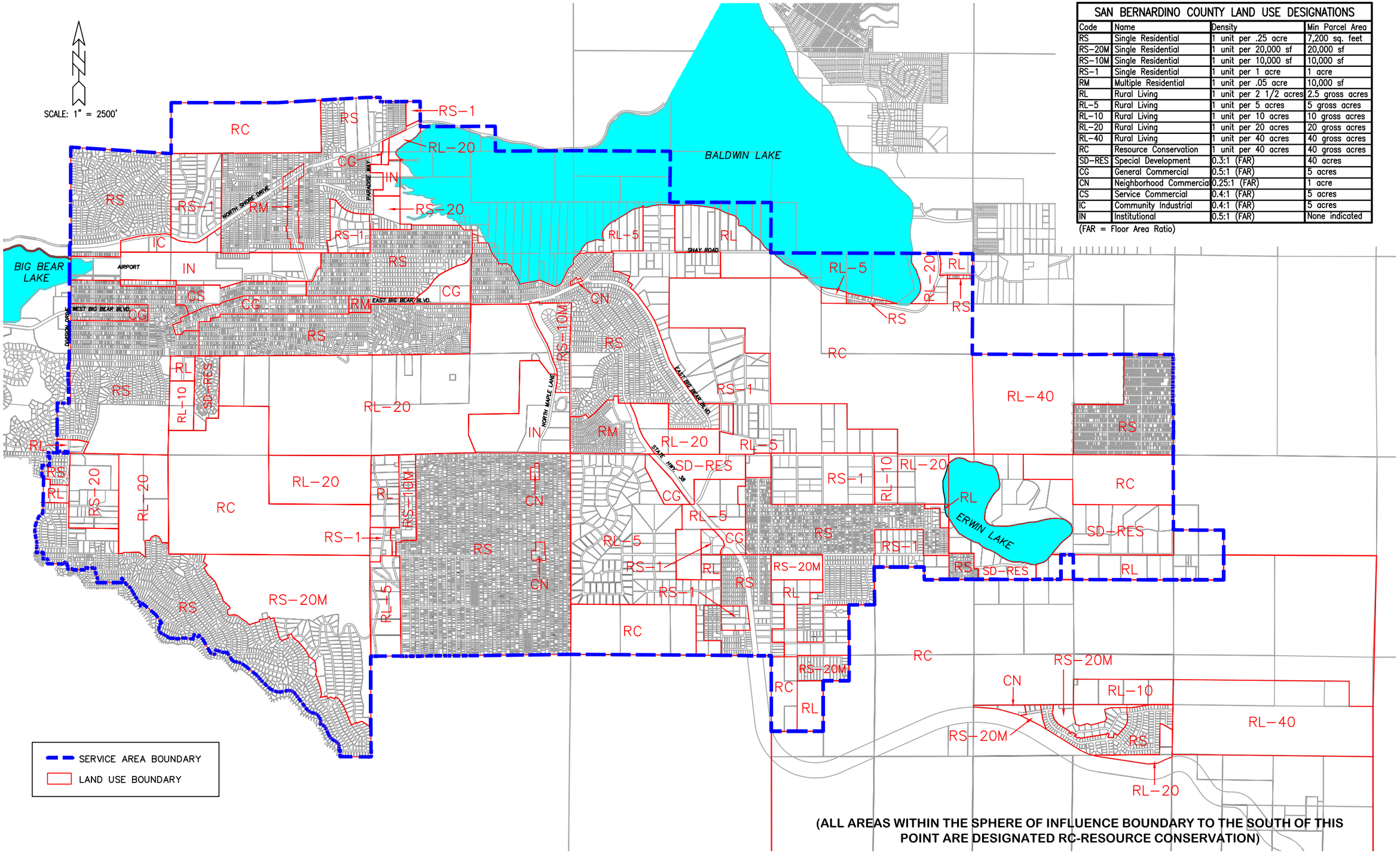
Figure 2-1



(ALL AREAS WITHIN THE SPHERE OF INFLUENCE BOUNDARY TO THE SOUTH OF THIS POINT ARE DESIGNATED RC-RESOURCE CONSERVATION)

SAN BERNARDINO COUNTY LAND USE DESIGNATIONS

Code	Name	Density	Min Parcel Area
RS	Single Residential	1 unit per .25 acre	7,200 sq. feet
RS-20M	Single Residential	1 unit per 20,000 sf	20,000 sf
RS-10M	Single Residential	1 unit per 10,000 sf	10,000 sf
RS-1	Single Residential	1 unit per 1 acre	1 acre
RM	Multiple Residential	1 unit per .05 acre	10,000 sf
RL	Rural Living	1 unit per 2 1/2 acres	2.5 gross acres
RL-5	Rural Living	1 unit per 5 acres	5 gross acres
RL-10	Rural Living	1 unit per 10 acres	10 gross acres
RL-20	Rural Living	1 unit per 20 acres	20 gross acres
RL-40	Rural Living	1 unit per 40 acres	40 gross acres
RC	Resource Conservation	1 unit per 40 acres	40 gross acres
SD-RES	Special Development	0.3:1 (FAR)	40 acres
CG	General Commercial	0.5:1 (FAR)	5 acres
CN	Neighborhood Commercial	0.25:1 (FAR)	1 acre
CS	Service Commercial	0.4:1 (FAR)	5 acres
IC	Community Industrial	0.4:1 (FAR)	5 acres
IN	Institutional	0.5:1 (FAR)	None indicated

(FAR = Floor Area Ratio)

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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

SAN BERNARDINO COUNTY LAND USE DESIGNATIONS

FIGURE
2-2

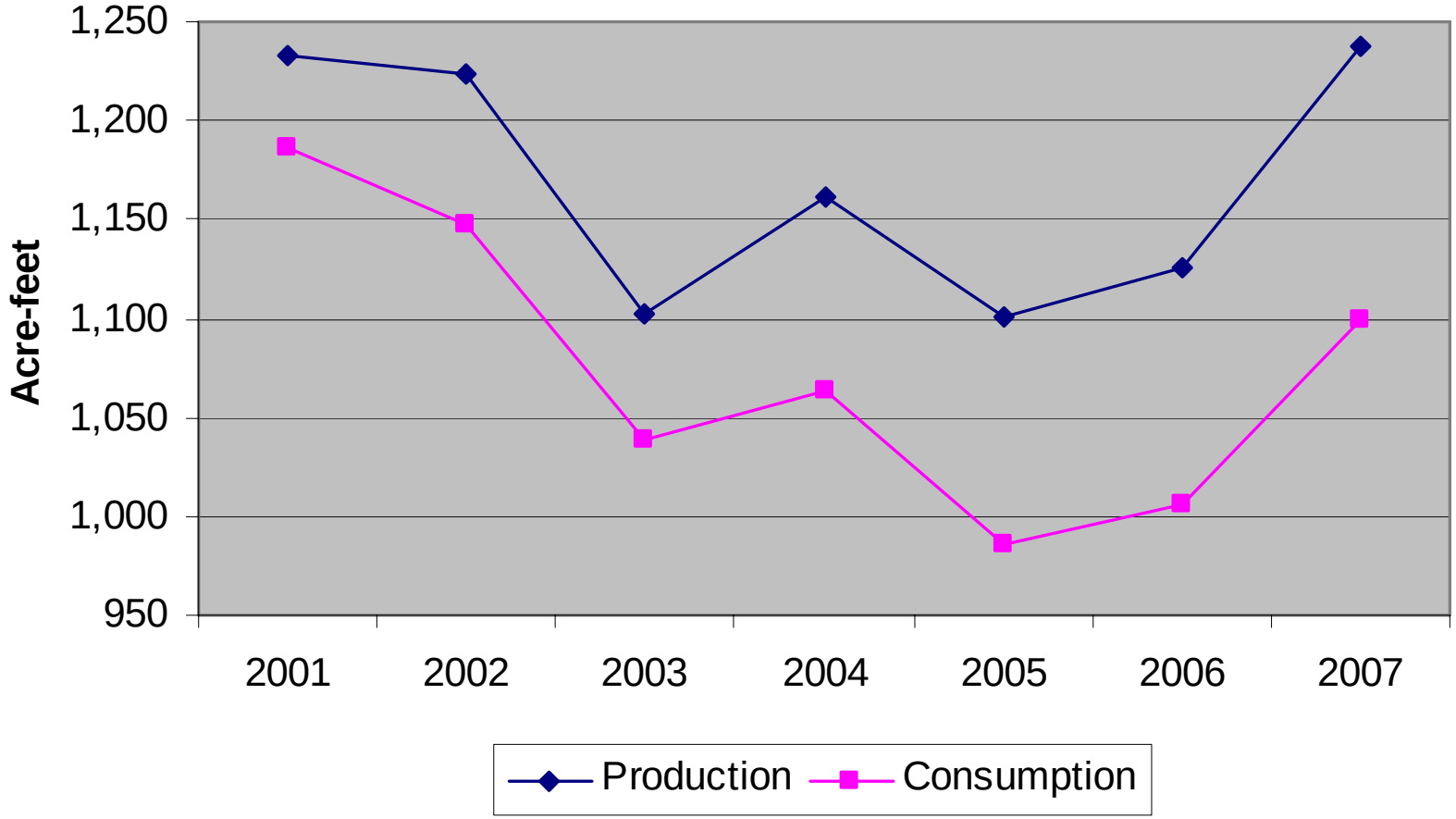

Figure 2-3

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Annual Production and Consumption

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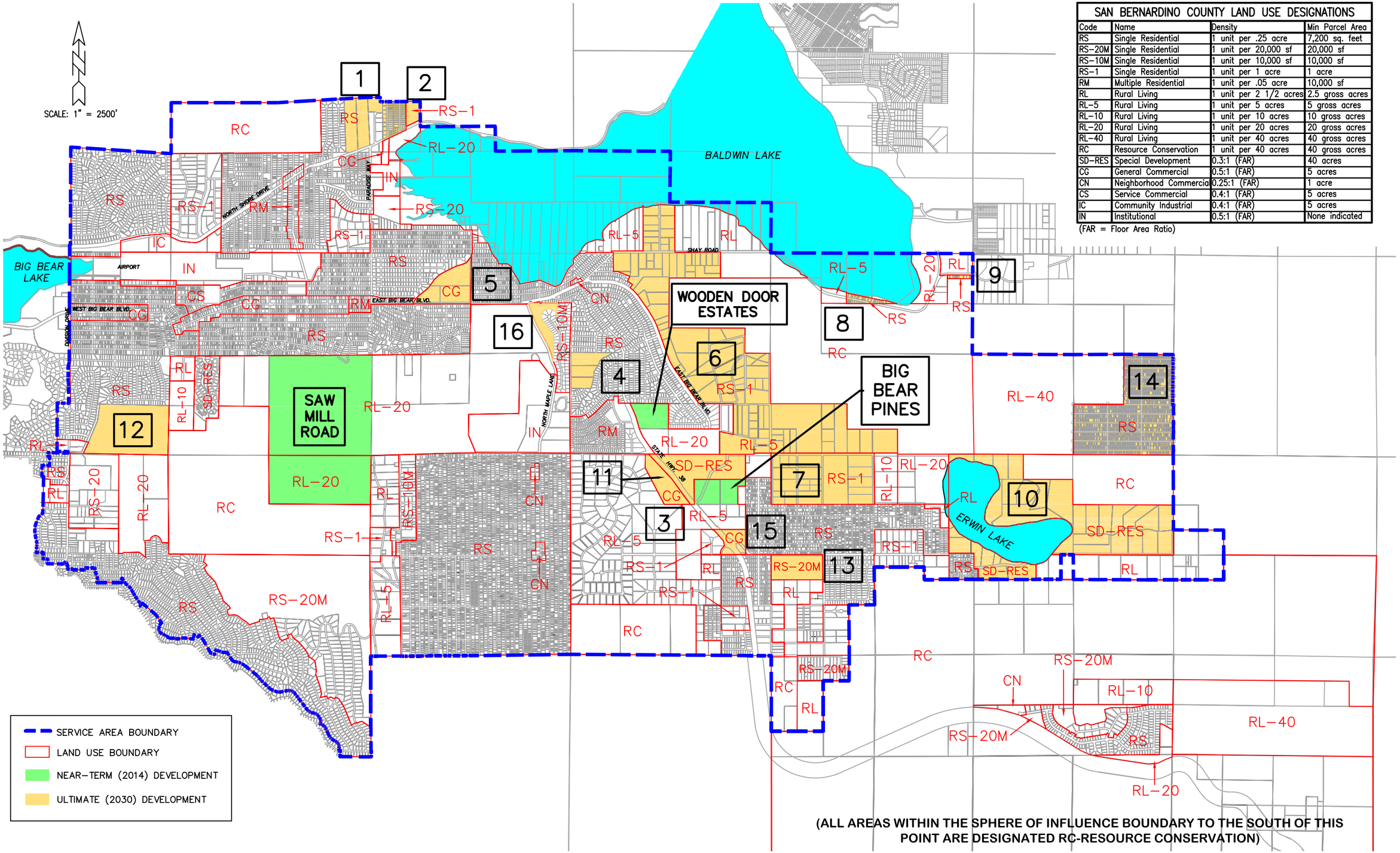

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RM	Multiple Residential	1 unit per .05 acre	10,000 sf
RL	Rural Living	1 unit per 2 1/2 acres	2.5 gross acres
RL-5	Rural Living	1 unit per 5 acres	5 gross acres
RL-10	Rural Living	1 unit per 10 acres	10 gross acres
RL-20	Rural Living	1 unit per 20 acres	20 gross acres
RL-40	Rural Living	1 unit per 40 acres	40 gross acres
RC	Resource Conservation	1 unit per 40 acres	40 gross acres
SD-RES	Special Development	0.3:1 (FAR)	40 acres
CG	General Commercial	0.5:1 (FAR)	5 acres
CN	Neighborhood Commercial	0.25:1 (FAR)	1 acre
CS	Service Commercial	0.4:1 (FAR)	5 acres
IC	Community Industrial	0.4:1 (FAR)	5 acres
IN	Institutional	0.5:1 (FAR)	None indicated

(FAR = Floor Area Ratio)

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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECTED NEAR TERM (2014) AND ULTIMATE (2030) DEVELOPEMENT

FIGURE 2-4

Source: Topographic base adopted from Big Bear Lake, California and San Bernardino, California USGS quadrangle map, 1982

S:/PROJECTS/WR08.0064_BBCCSD/GIS/MXDS/REPORT_11-09/FIG3-1_TOPO_BASIN-BOUNDARY.MXD

Daniel B. Stephens & Associates, Inc.
1/8/2010
JN WR08.0064.00

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

Big Bear Valley Topography and Basin Boundary

Figure 3-1

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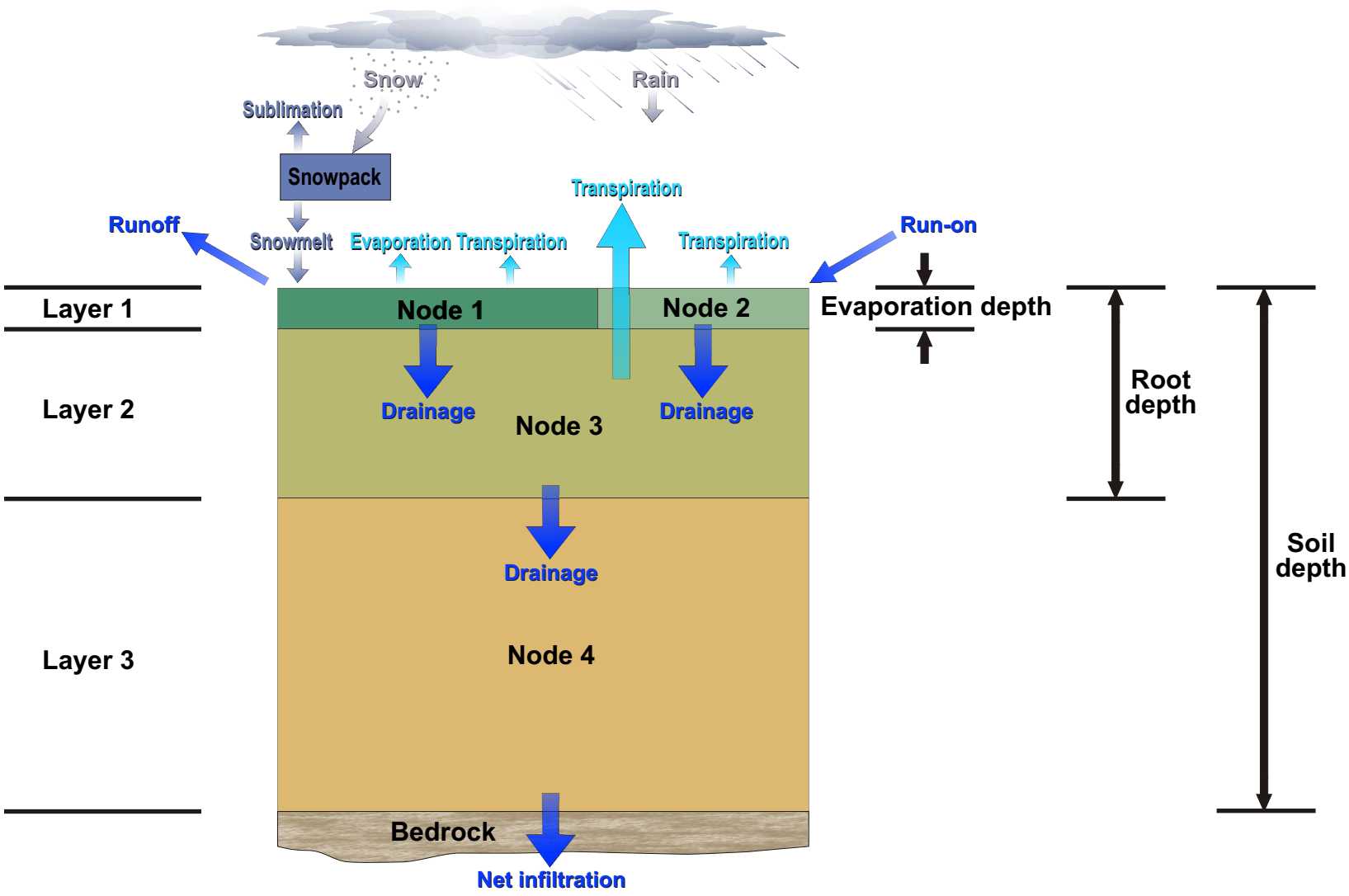

Notes:

Not to scale

Node 1 = fraction exposed and wetted (f_{ew})

Node 2 = fraction covered by vegetation canopy (f_c)

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

Schematic of Water Balance Components and Computational Nodes Present in a Single Model Cell

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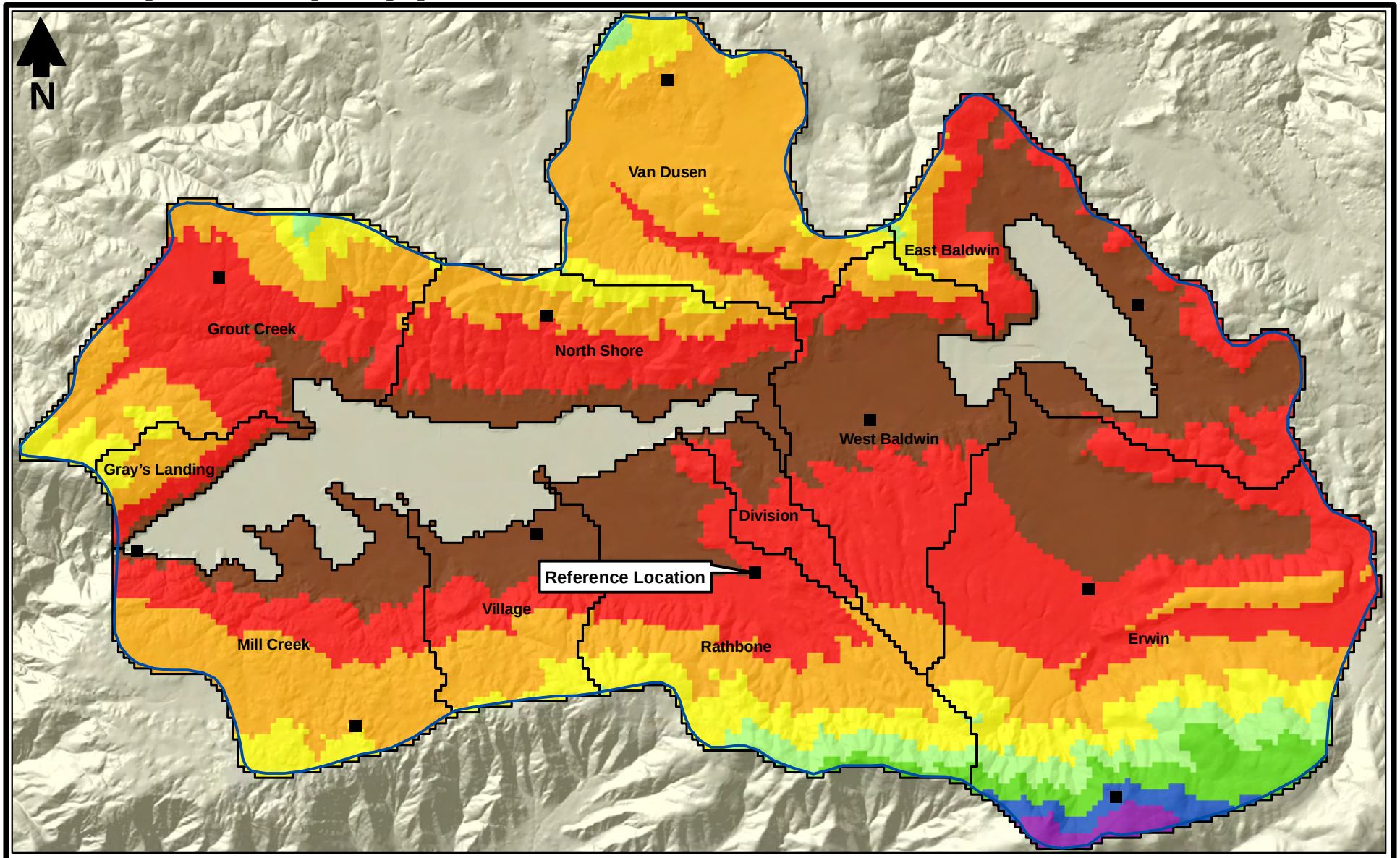
Figure 3-2

S:/PROJECTS/WR08.0064_BBCCSD/GIS/MXDS/REPORT_11-09/FIG3-3_AVG_PRECIP.MXD



0 3750 7500 Feet

Explanation

Precipitation (in/yr)

< 24
24 - 26
26 - 28
28 - 30
30 - 32
32 - 34
34 - 36
> 36

Basin
Sub-basin
DAYMET location

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

DPWM Mean Annual Precipitation

1981 - 2003

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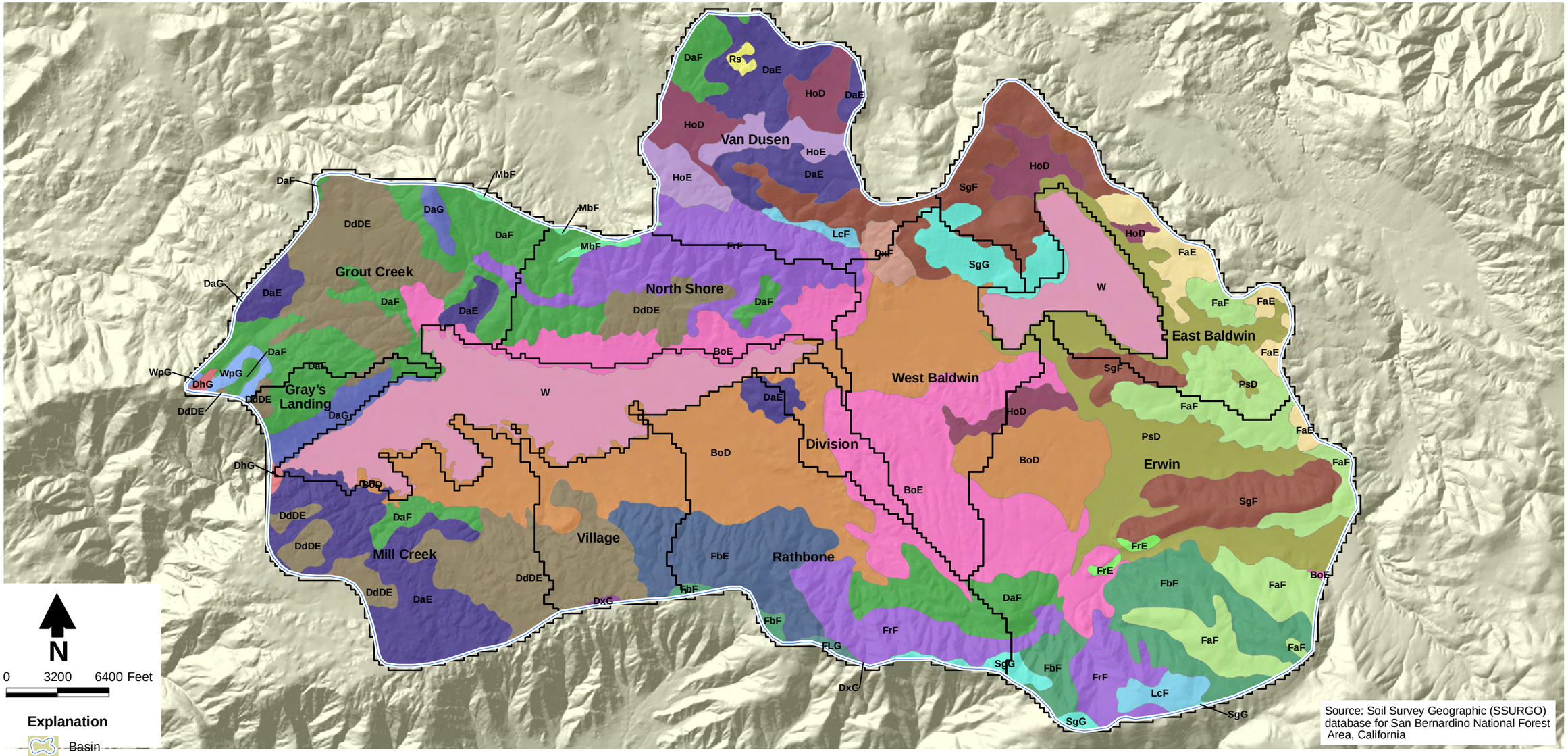
JN WR08.0064

Figure 3-3



Explanation

- Basin
- Sub-basin
- PsD — Avawatz-Oak Glen, dry families association, 2 to 15 percent slopes
- FrF — Lithic Xerorthents, calcareous-Lazzant family association, 30 to 50 percent slopes
- LcF — Lithic Xerorthents, calcareous-Rock outcrop complex, 30 to 50 percent slopes
- DhG — Lithic Xerorthents-Springdale family-Rubble land association, 50 to 100 percent slopes
- FrE — Lizzant family-Lithic Xerorthents, calcareous association, 15 to 30 percent slopes
- FbE — Merkel-Switchback families complex, 15 to 30 percent slopes
- FbF — Merkel-Wapal families complex, 30 to 50 percent slopes
- BoE — Morical, very deep-Hecker families complex, 15 to 30 percent slopes
- BoD — Morical, very deep-Hecker families complex, 2 to 15 percent slopes
- HoE — Morical, very deep-Hodgson families association, 15 to 30 percent slopes
- HoD — Morical, very deep-Hodgson families association, 2 to 15 percent slopes
- MbF — Morical-Wind River families complex, 30 to 50 percent slopes
- FaE — Olete-Goulding families association, 15 to 30 percent slopes
- SgG — Olete-Goulding families-Rubble land association, 50 to 100 percent slopes
- FaF — Olete-Goulding families. association, 30 to 50 percent slopes
- SgF — Olete-Kilburn-Goulding families complex, 30 to 50 percent slopes
- DdDE — Pacifico-Preston families complex, 2 to 30 percent slopes
- DaE — Pacifico-Wapi families complex, 15 to 30 percent slopes
- DaF — Pacifico-Wapi families complex, 30 to 50 percent slopes
- Rs — Rock outcrop, 30 to 100 percent slopes
- FLG — Springdale family-Lithic Xerorthents association,dry, 50 to 75 percent slopes
- WpG — Wapal family-Lithic Xerorthents, cool association, 50 to 75 percent slopes
- DxF — Wapi-Pacifico families, dry-Rock outcrop complex, 30 to 50 percent slopes
- DxG — Wapi-Pacifico families, dry-Rock outcrop complex, 50 to 75 percent slopes
- DaG — Wapi-Pacifico families-Rock outcrop complex, 50 to 75 percent slopes
- W — Water areas

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

Soil Types

Daniel B. Stephens & Associates, Inc.

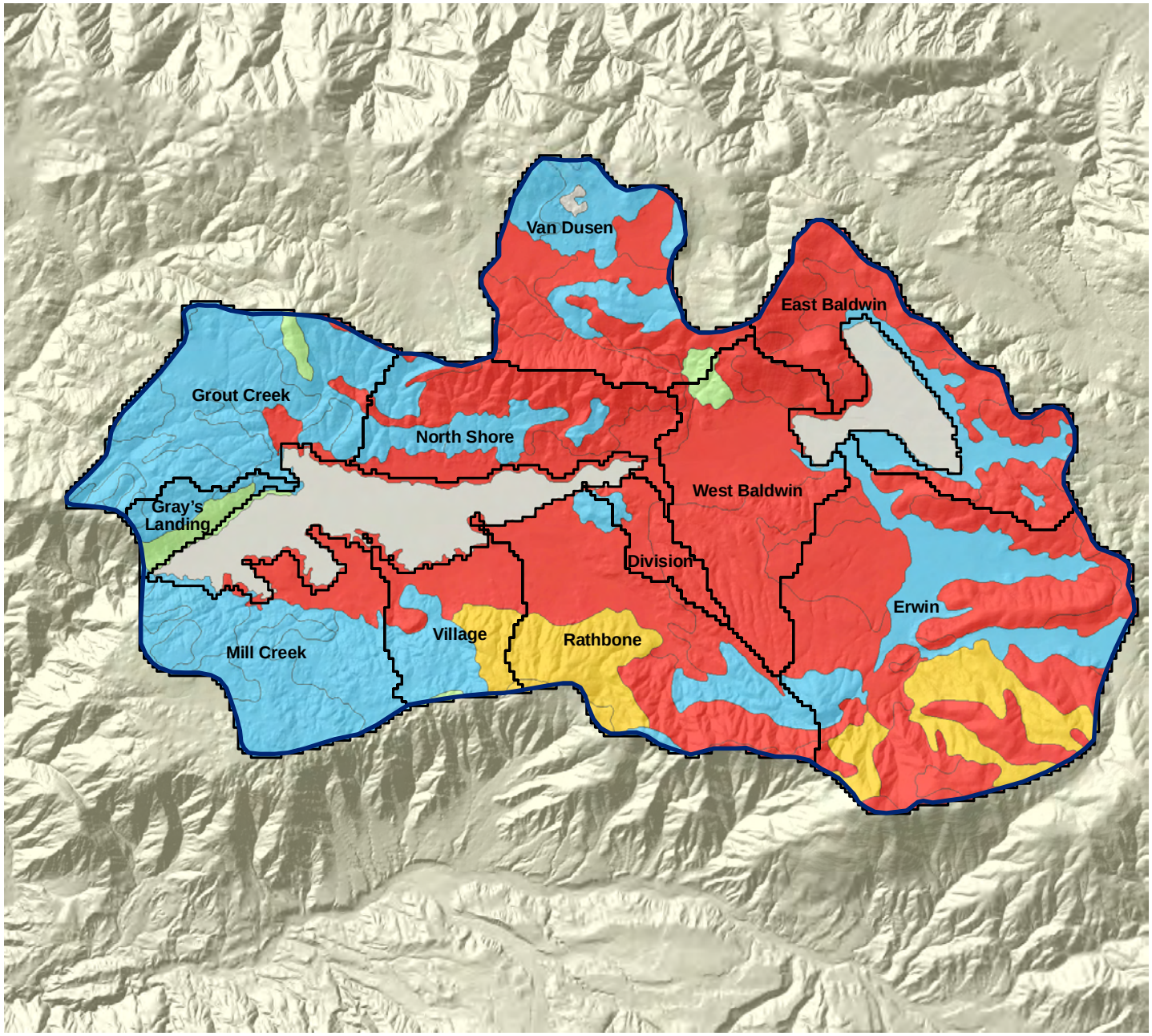
1/8/2010 JN WR08.0064

Figure 3-4

S:\PROJECTS\WR08.0064 BBCCSD\GIS\MXDS\REPORT 11-09\FIG3-4 SSURGO MAPUNITS.MXD 900111

S:/PROJECTS/WR08.0064_BBCCSD/GIS/MXDS/REPORT_11-09/FIG3-5_SSURGO_HYDRAULIC_CONDUCTIVITY.MXD

Source: Soil Survey Geographic (SSURGO) database for San Bernardino National Forest Area, California



Explanation

- Basin
- Sub-basin

Saturated hydraulic conductivity (Ksat)

(in/s x 10^4 [cm/s x 10^4])

- 0 - 4.4 (0 - 11.3)
- 4.4 - 16.0 (11.3 - 40.6)
- 16.0 - 25.6 (40.6 - 65.1)
- 25.6 - 78.4 (65.1 - 199.1)
- 78.4 - 166.5 (199.1 - 423.0)
- Not rated or not available

1 inch = 2.54 cm

N

0 1 2 Miles

Figure 3-5

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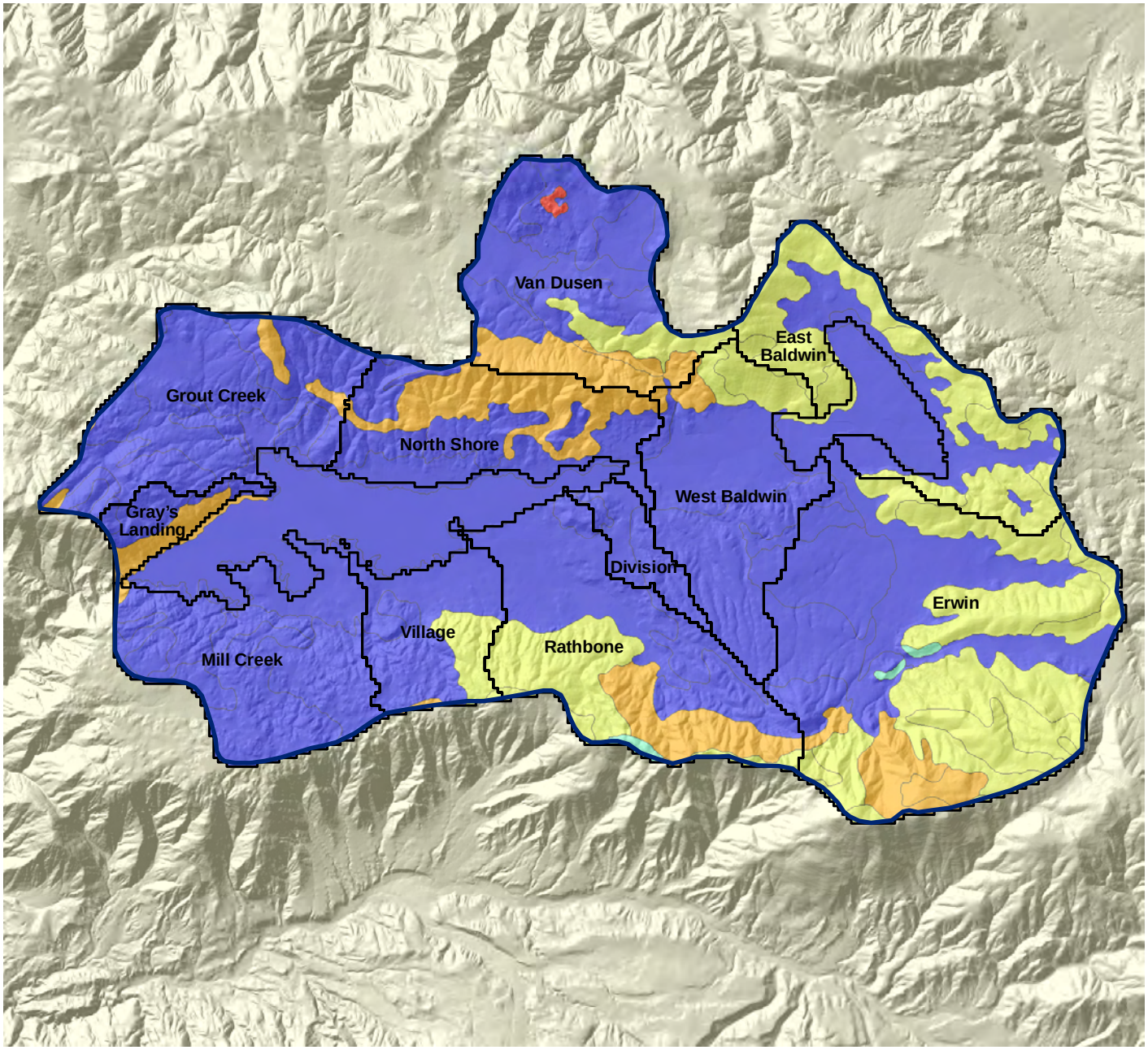
JN WR08.0064

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

Saturated Hydraulic Conductivity

S:/PROJECTS/WR08.0064_BBCCSD/GIS/MXDS/REPORT_11-09/FIG3-6_SSURGO_SOIL_DEPTH_TO_BEDROCK.MXD

Source: Soil Survey Geographic (SSURGO) database for San Bernardino National Forest Area, California



Explanation

Basin

Sub-basin

Depth to a selected soil restrictive layer (in [cm])

- 0 - 9.8 (0 - 25)
- 9.8 - 19.7 (25 - 50)
- 19.7 - 39.4 (50 - 100)
- 39.4 - 59.1 (100 - 150)
- 59.1 - 78.7 (150 - 200)
- > 78.7 (200)

1 inch = 2.54 cm

N

0 1 2 Miles

Figure 3-6

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

Depth to Lithic Bedrock

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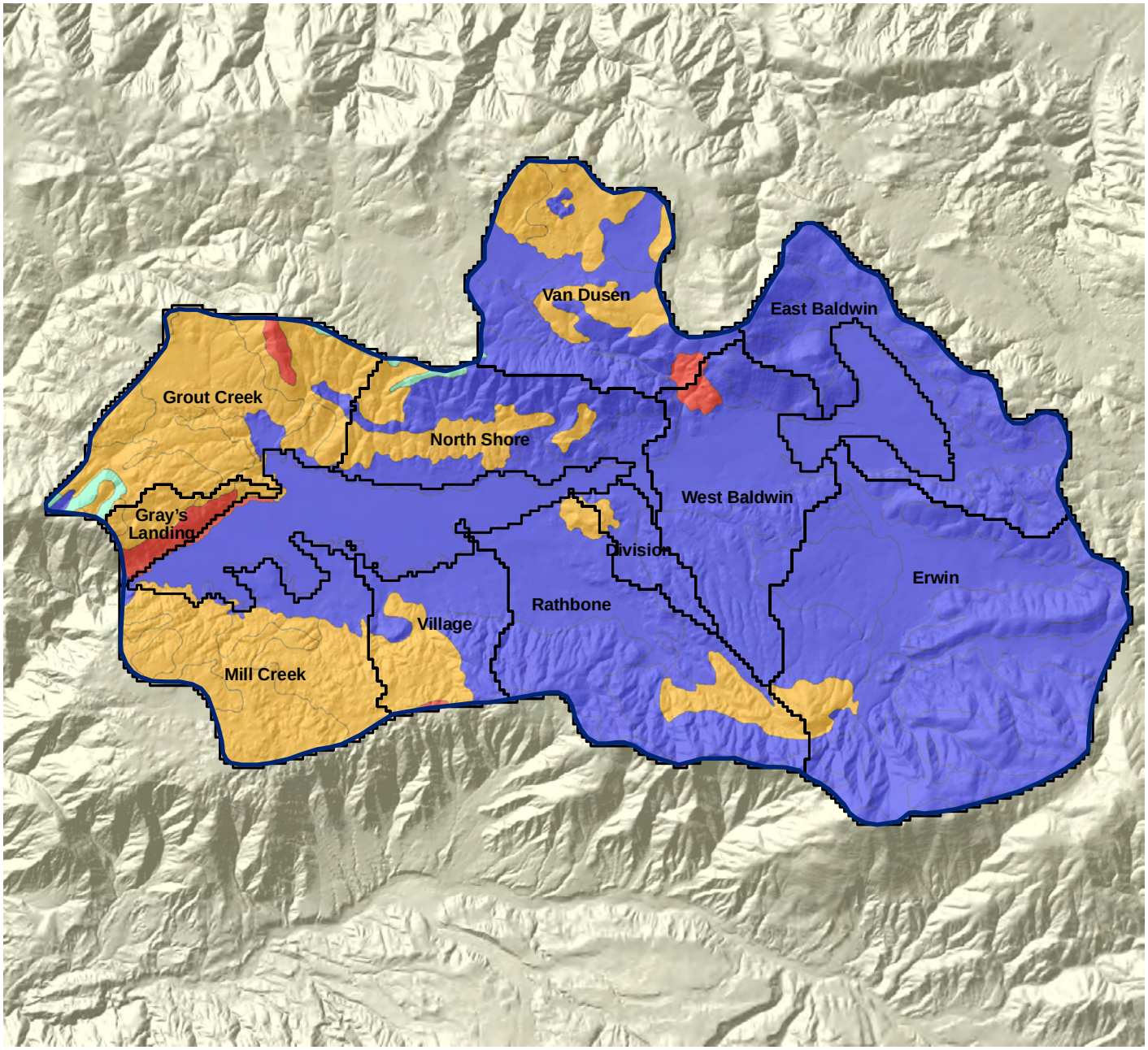
1/8/2010

JN WR08.0064

S:/PROJECTS/WR08.0064_BBCCSD/GIS/MXDS/REPORT_11-09/FIG3-7_SSURGO_SOIL_DEPTH_TO_PARALITHIC_BEDROCK.MXD

Source: Soil Survey Geographic (SSURGO) database for San Bernardino National Forest Area, California



Explanation

Basin

Sub-basin

Depth to a selected soil restrictive layer (in [cm])

- 0 - 9.8 (0 - 25)
- 9.8 - 19.7 (25 - 50)
- 19.7 - 39.4 (50 - 100)
- 39.4 - 59.1 (100 - 150)
- 59.1 - 78.7 (150 - 200)
- > 78.7 (200)

1 inch = 2.54 cm

N

0 1 2 Miles

Figure 3-7

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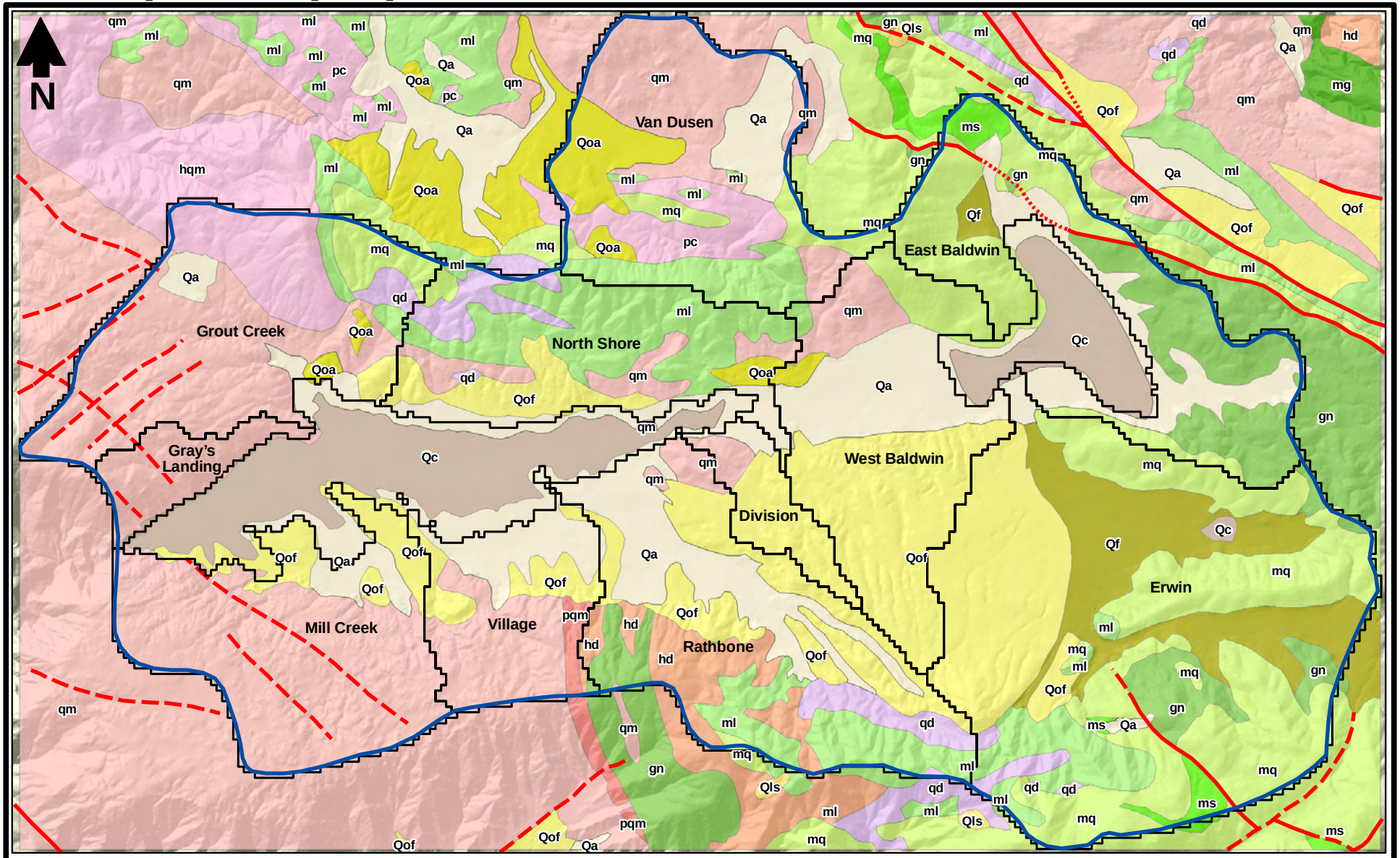
JN WR08.0064

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

Depth to Paralithic Bedrock

S:/PROJECTS/WR08.0064_BBCCSD/GIS/MXDS/REPORT_11-09/FIG3-8_GEOLOGY.MXD



Explanation

0 3750 7500 Feet

Qa	Qc	hd	pqm	mg	ms	Basin
Qoa	Qf	hqm	qd	ml	gn	Sub-basin
Qof	Qls	pc	qm	mq		

Fault (dashed where uncertain, dotted where concealed)

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

Bedrock Geology

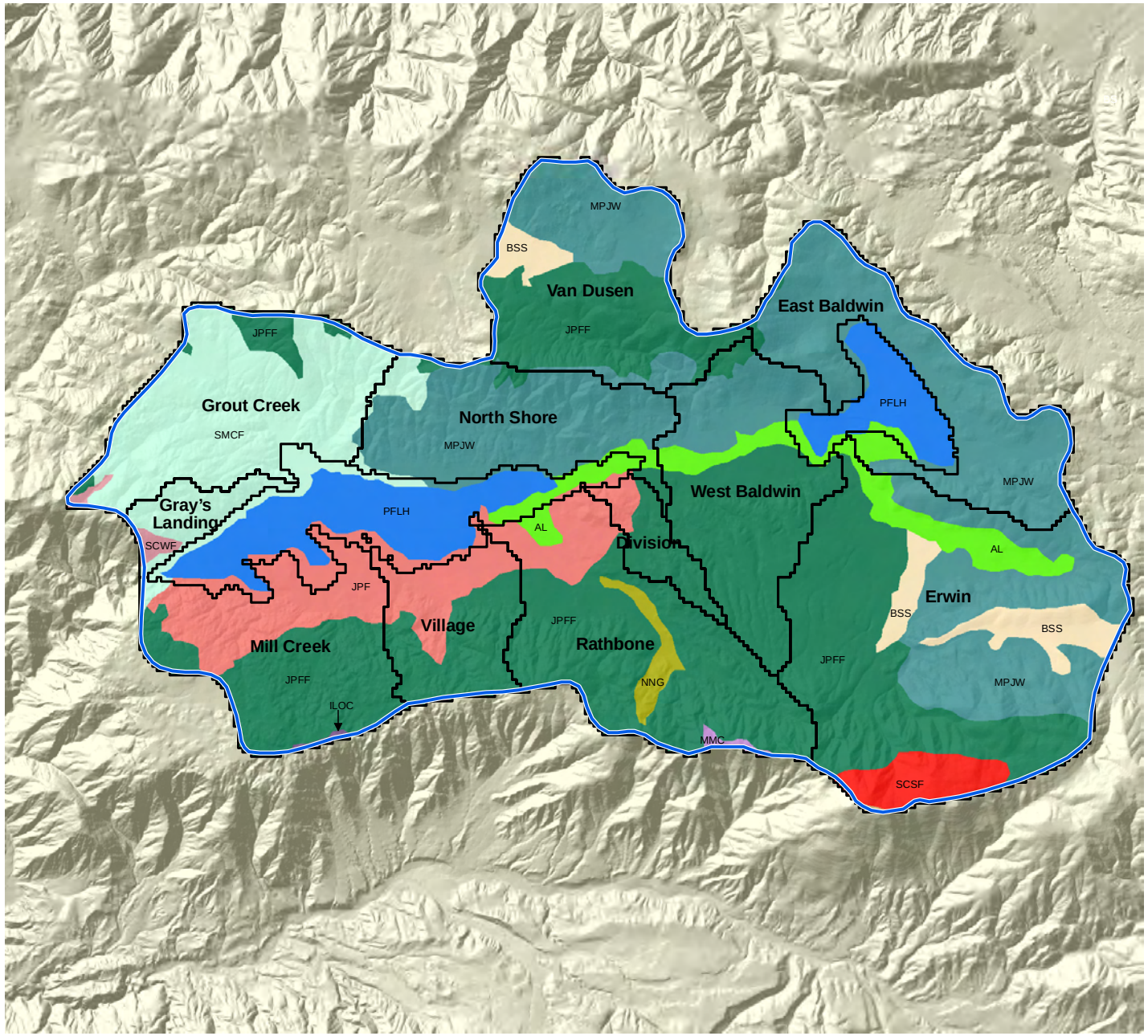
Daniel B. Stephens & Associates, Inc.

1/8/2010

JN WR08.0064

Figure 3-8

S:/PROJECTS/WR08.0064_BBCCSD/GIS/MXDS/REPORT_11-09/FIG3-9_GAP_LANDCOVER.MXD 012//1



Explanation

- Basin
- Sub-basin
- AL — Agricultural land
- BSS — Big sagebrush scrub
- ILOC — Interior live oak chaparral
- JPFF — Jeffrey pine-fir forest
- JPF — Jeffrey pine forest
- MMC — Mixed montane chaparral
- MPJW — Mojavean piñon and juniper woodlands
- NNG — Non-native grassland
- PFLH — Permanently flooded lacustrine habitat
- SOC — Scrub oak chaparral
- SMCF — Sierran mixed coniferous forest
- SCSF — Southern California subalpine forest
- SCWF — Southern California white fir forest

N

0 1 2 Miles

Source: GAP Analysis Program

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

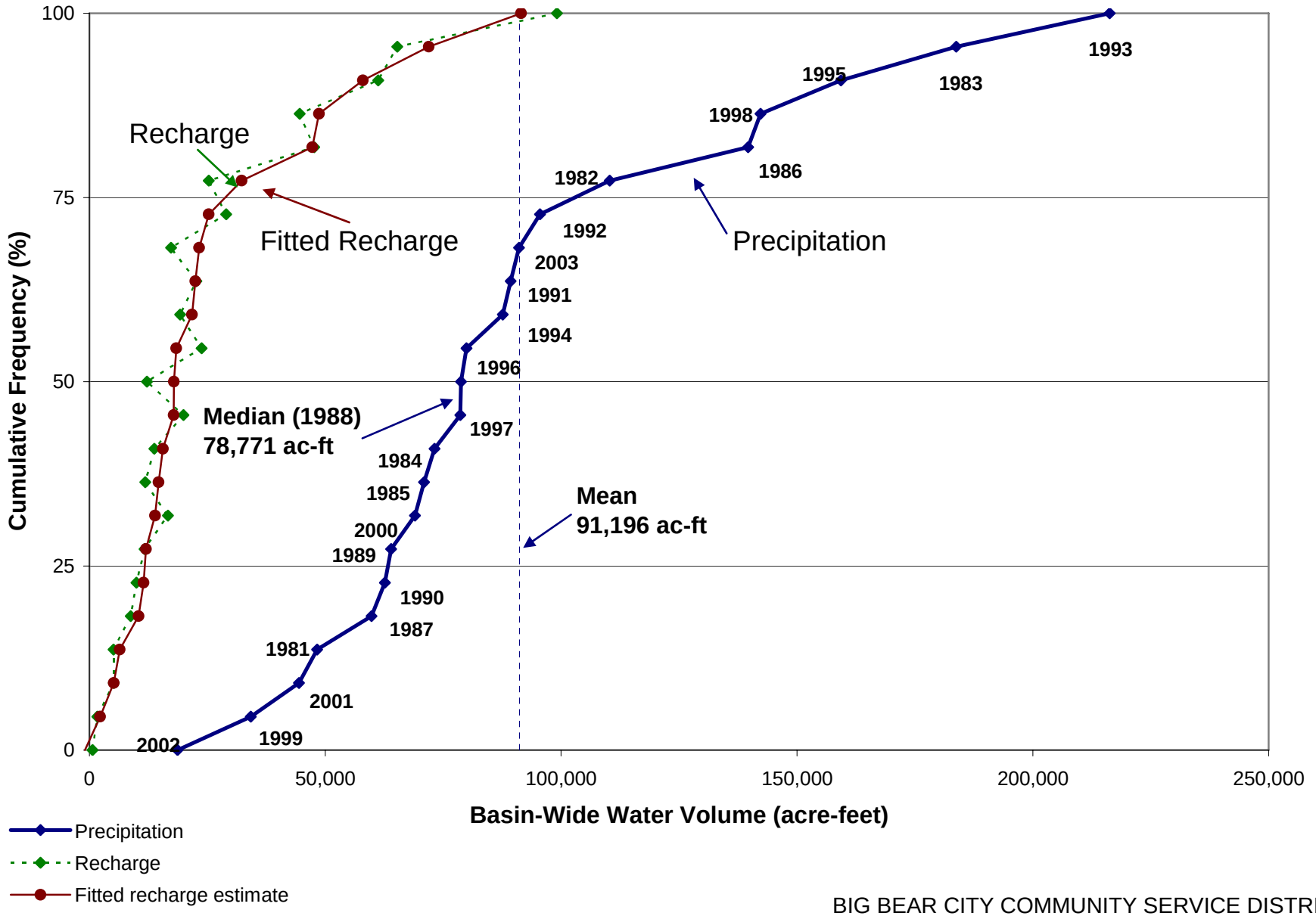
GAP Landcover - Vegetation Types

Daniel B. Stephens & Associates, Inc.

1/8/2010

JN WR08.0064

Figure 3-9



BIG BEAR CITY COMMUNITY SERVICE DISTRICT

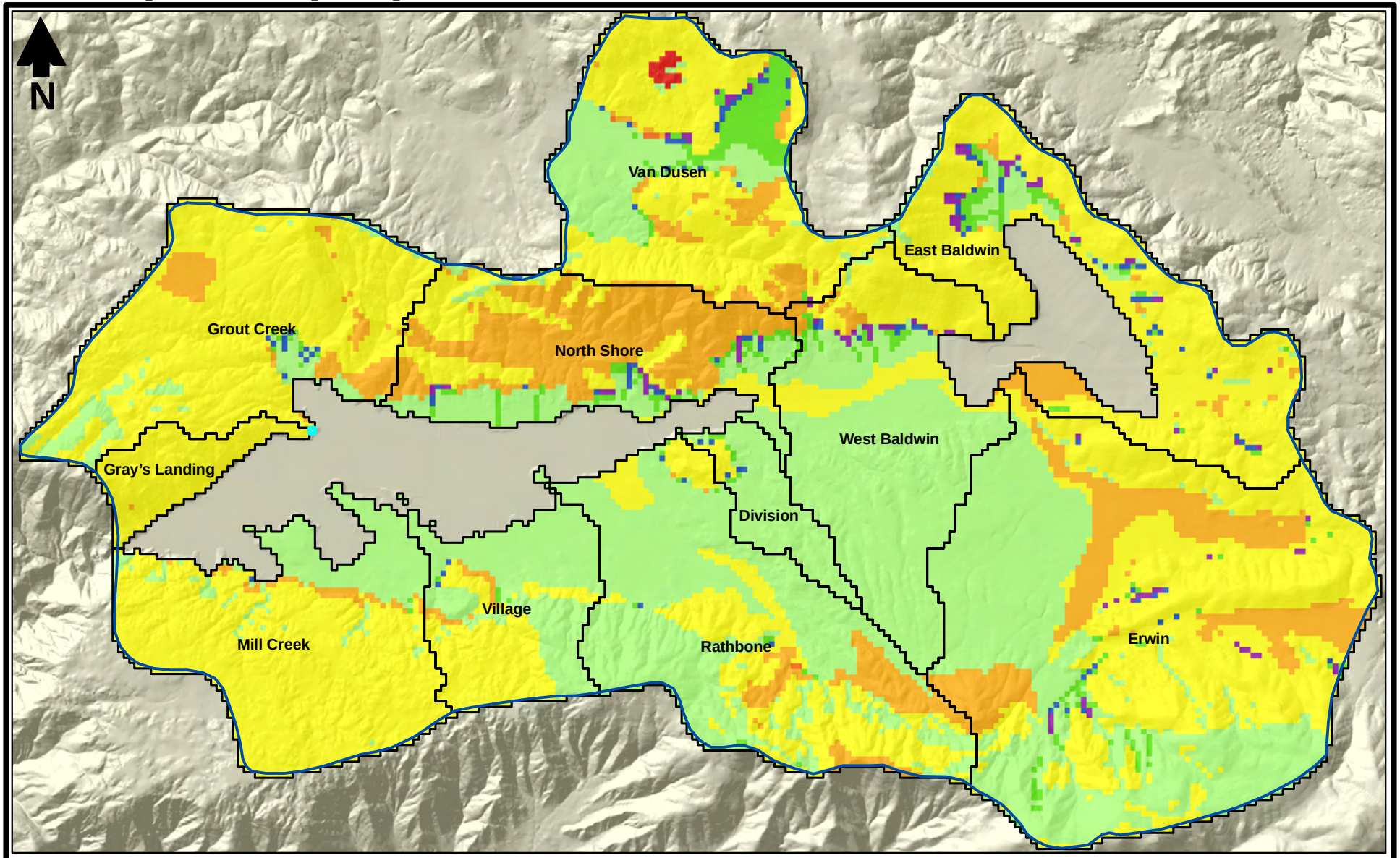
Cumulative Distribution Frequency of Modeled Precipitation 1981–2003

Daniel B. Stephens & Associates, Inc.

1/7/10

Figure 3-10

S:/PROJECTS/WR08.0064_BBCCSD/GIS/MXDS/REPORT_11-09/FIG3-11_ET.MXD 900111



0 3750 7500 Feet

Explanation

Evapotranspiration (in/yr)

< 1	10 - 15	25 - 30	Basin
1 -5	15 - 20	> 30	Sub-basin
5 - 10	20 - 25		

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

DPWM Mean Annual Actual Evapotranspiration

1981 - 2003

Daniel B. Stephens & Associates, Inc.

1/8/2010

JN WR08.0064

Figure 3-11

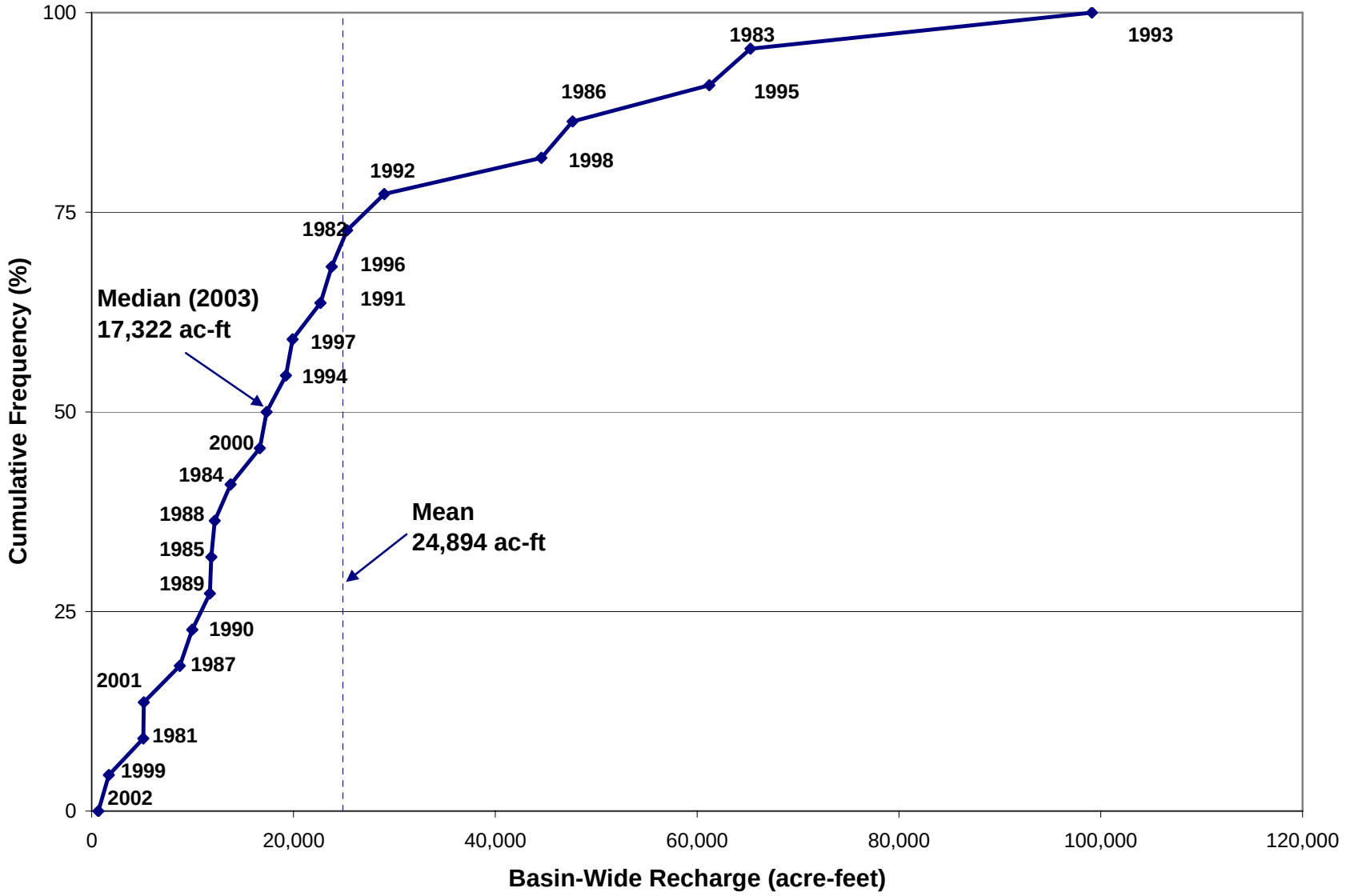


Figure 3-12

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Cumulative Distribution Frequency of Modeled Recharge
1981–2003

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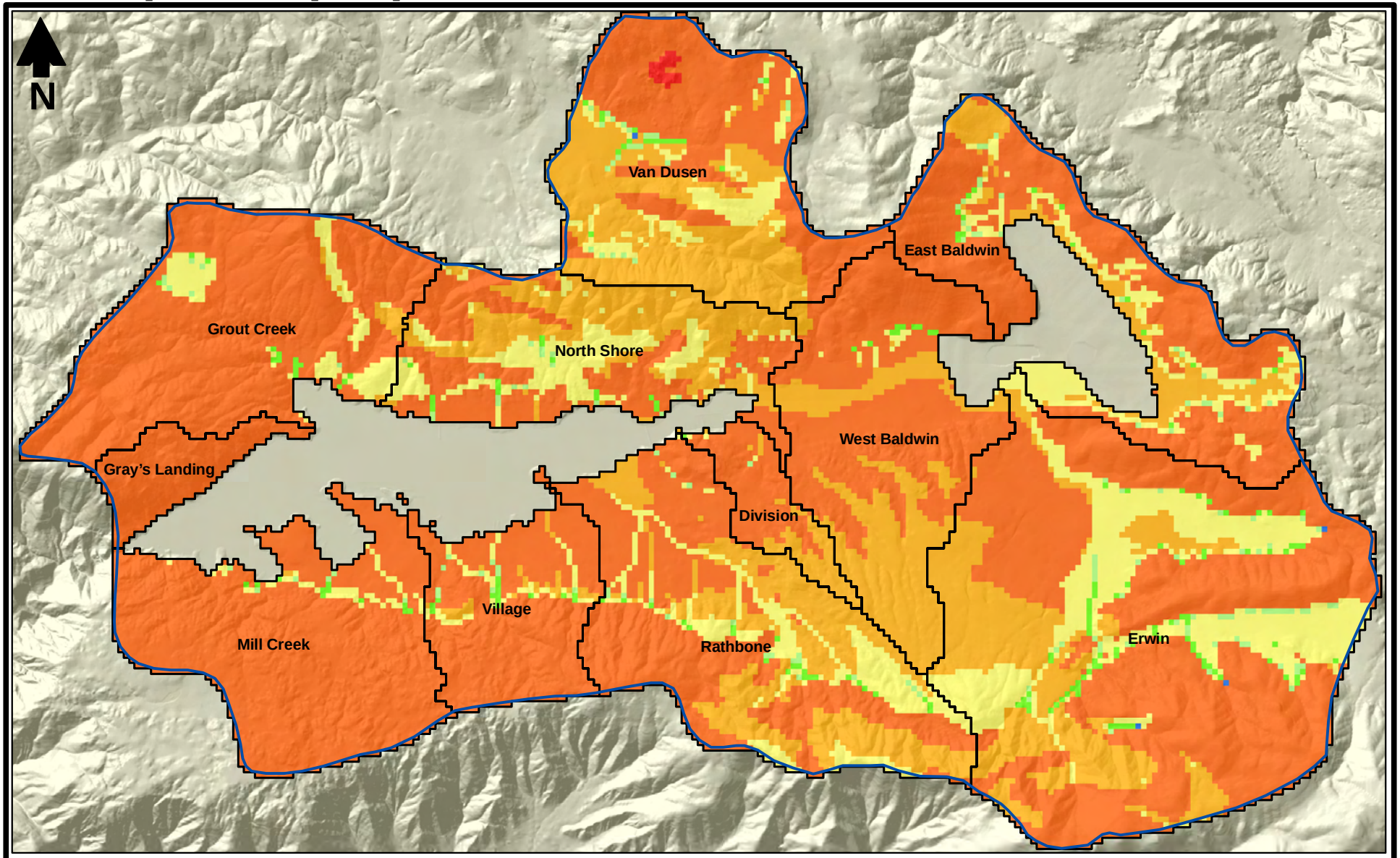
S:/PROJECTS/WR08.0064_BBCCSD/GIS/MXDS/REPORT_11-09/FIG3-13_RECHARGE.MXD 900111



0 3750 7500 Feet

Explanation

Recharge (in/yr)

0	10 - 50	500 - 1,000	Basin
1 - 5	50 - 100	> 1,000	Sub-basin
5 - 10	100 - 500		

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

DPWM Mean Annual Recharge

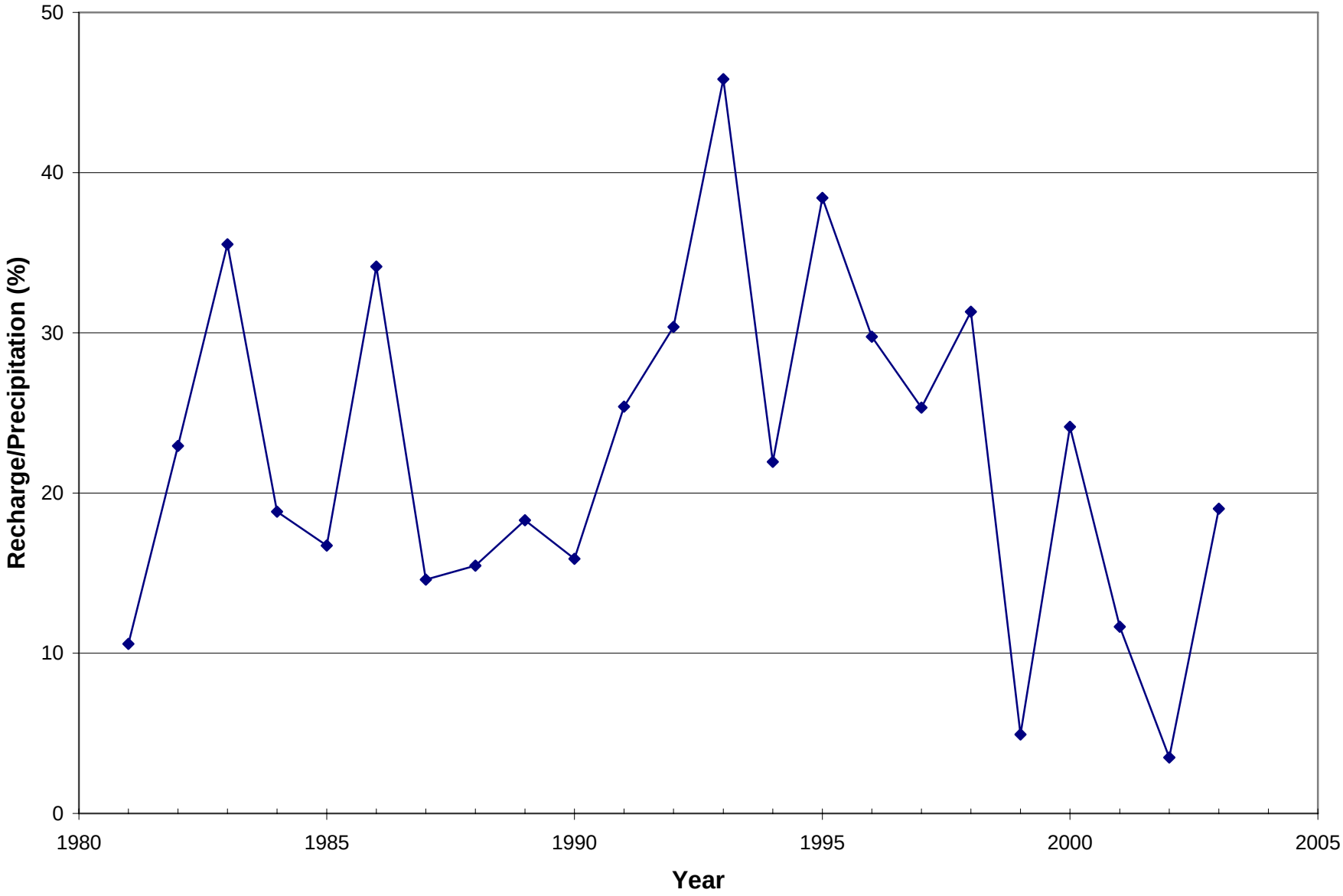
1981 - 2003

Daniel B. Stephens & Associates, Inc.

1/8/2010

JN WR08.0064

Figure 3-13



BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Recharge as Percentage of Precipitation
1981–2003

Figure 3-14

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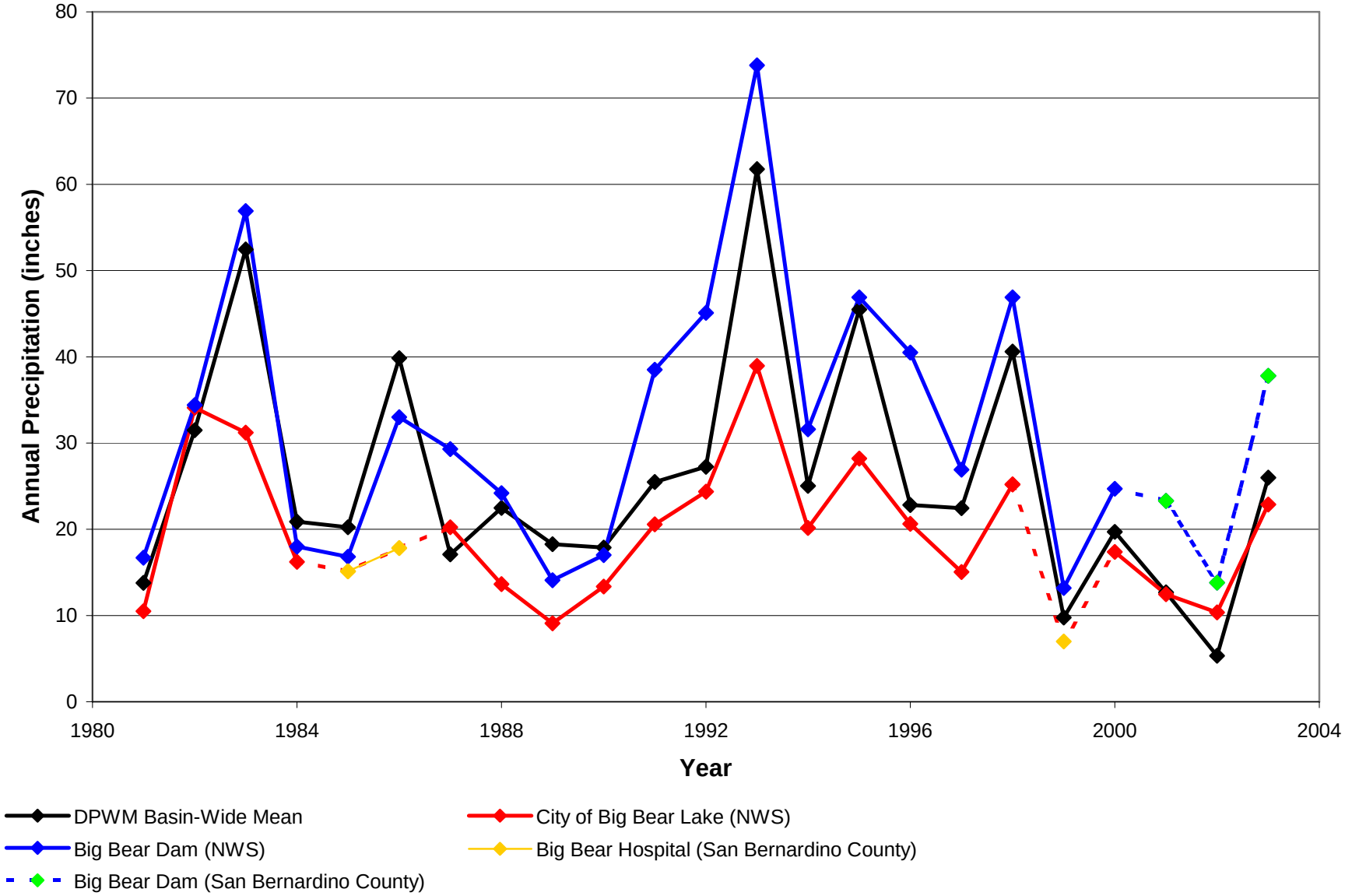

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Comparison of DPWM Mean Annual Precipitation to Weather Station Measured Annual Precipitation, 1981–2003

Daniel B. Stephens & Associates, Inc.

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Figure 3-15

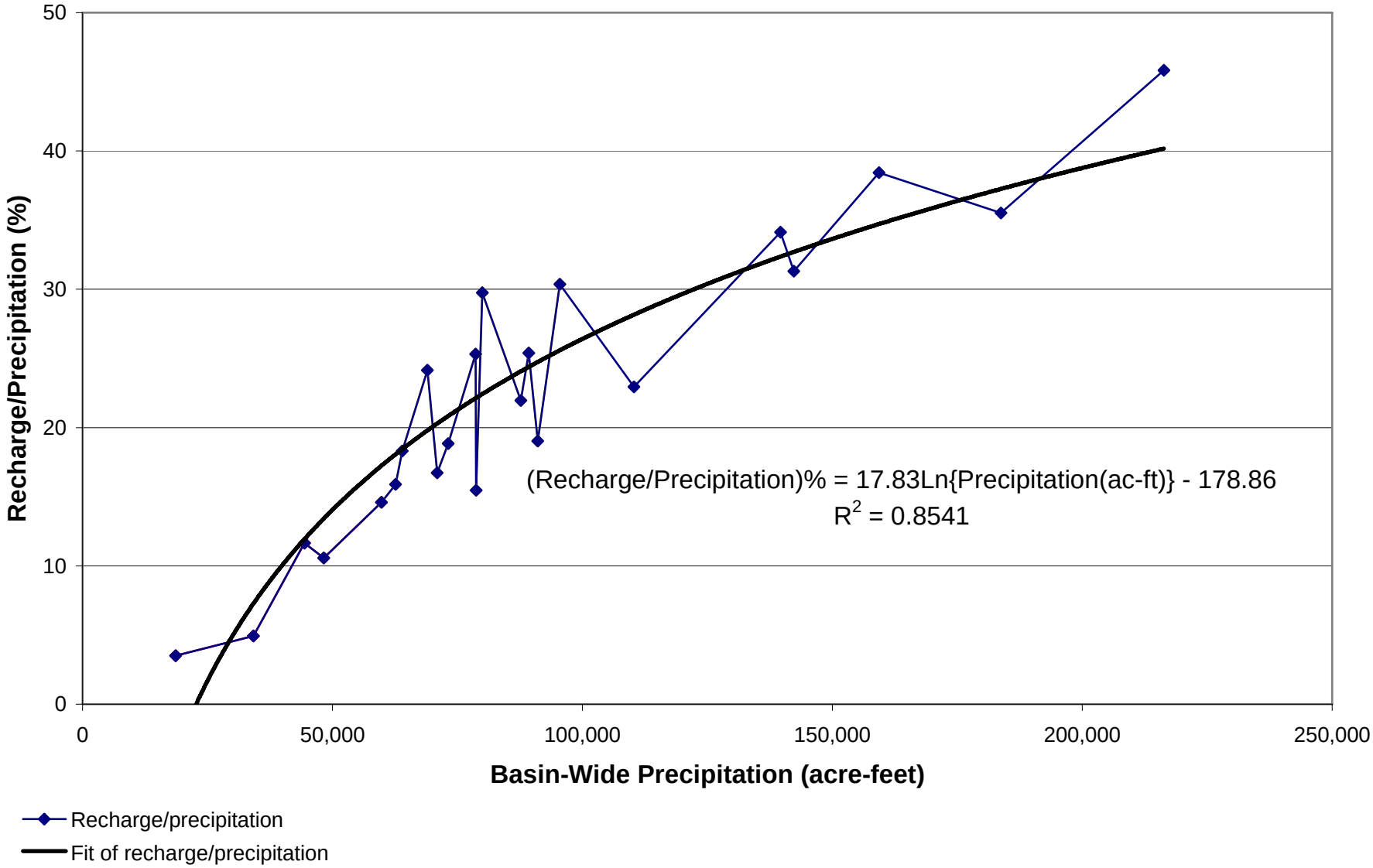

Figure 3-16

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Recharge/Precipitation vs. Precipitation

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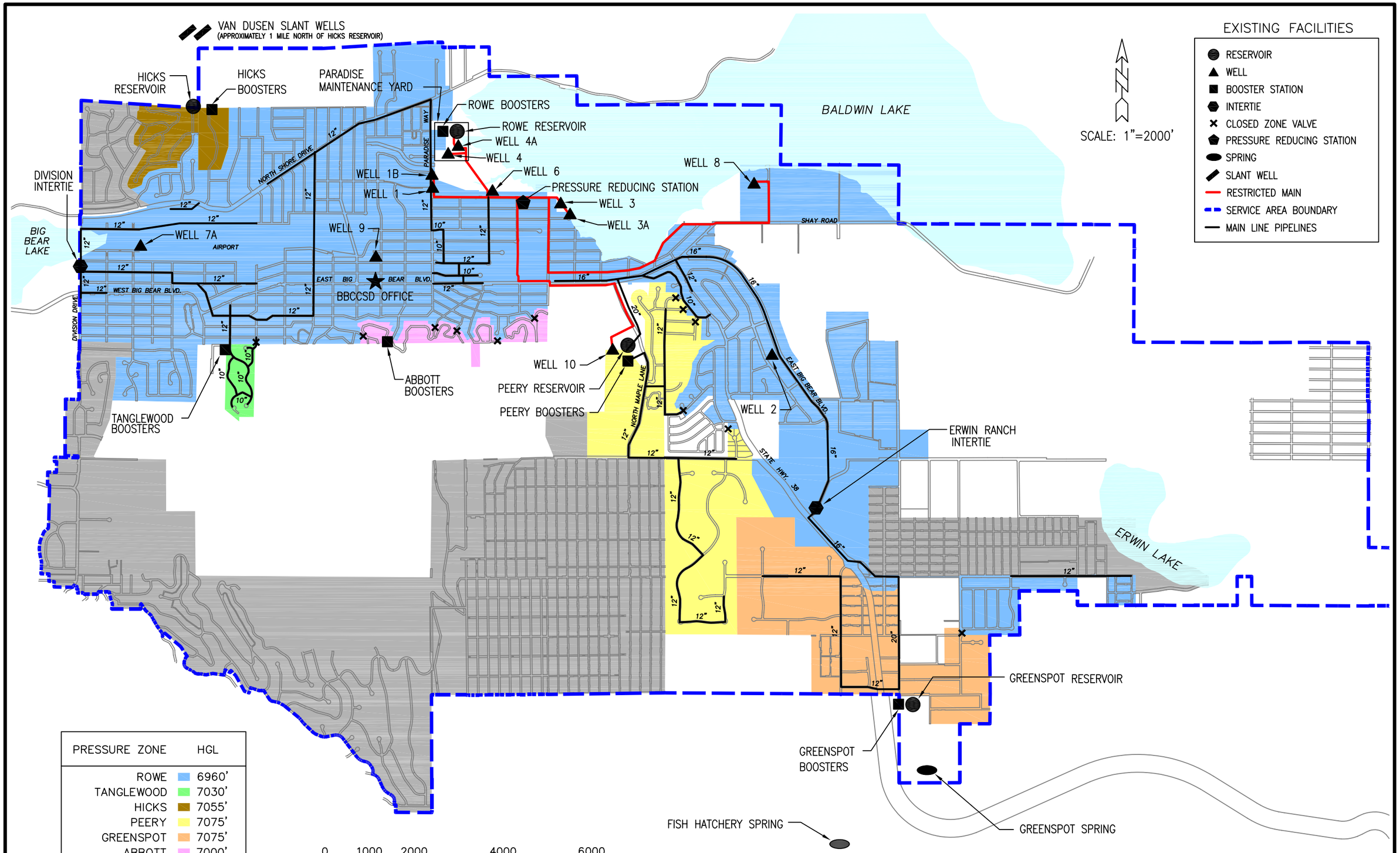

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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

EXISTING MAIN LINES AND PRIMARY FACILITIES

FIGURE
4-1

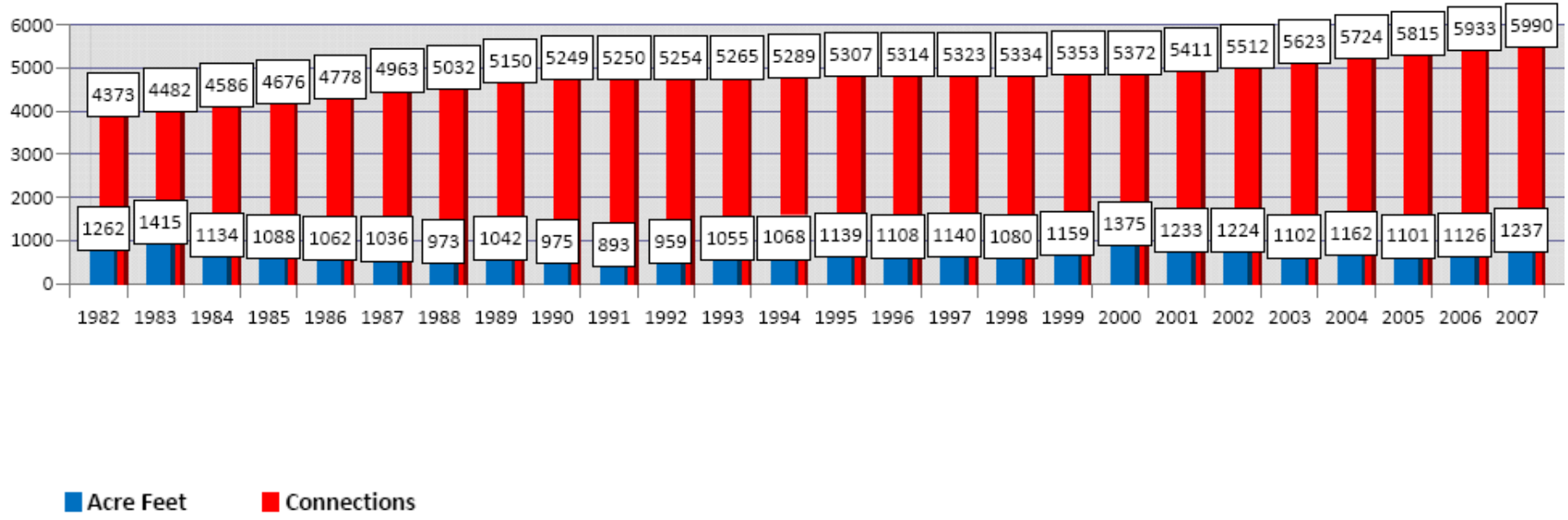

Figure 4-2

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Historical Production and Connection Totals

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1/7/10

Figure 4-3

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Rowe Booster Pumps

Daniel B. Stephens & Associates, Inc.

1/7/10

Figure 4-4

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Peery Fire Pump

Daniel B. Stephens & Associates, Inc.

1/7/10

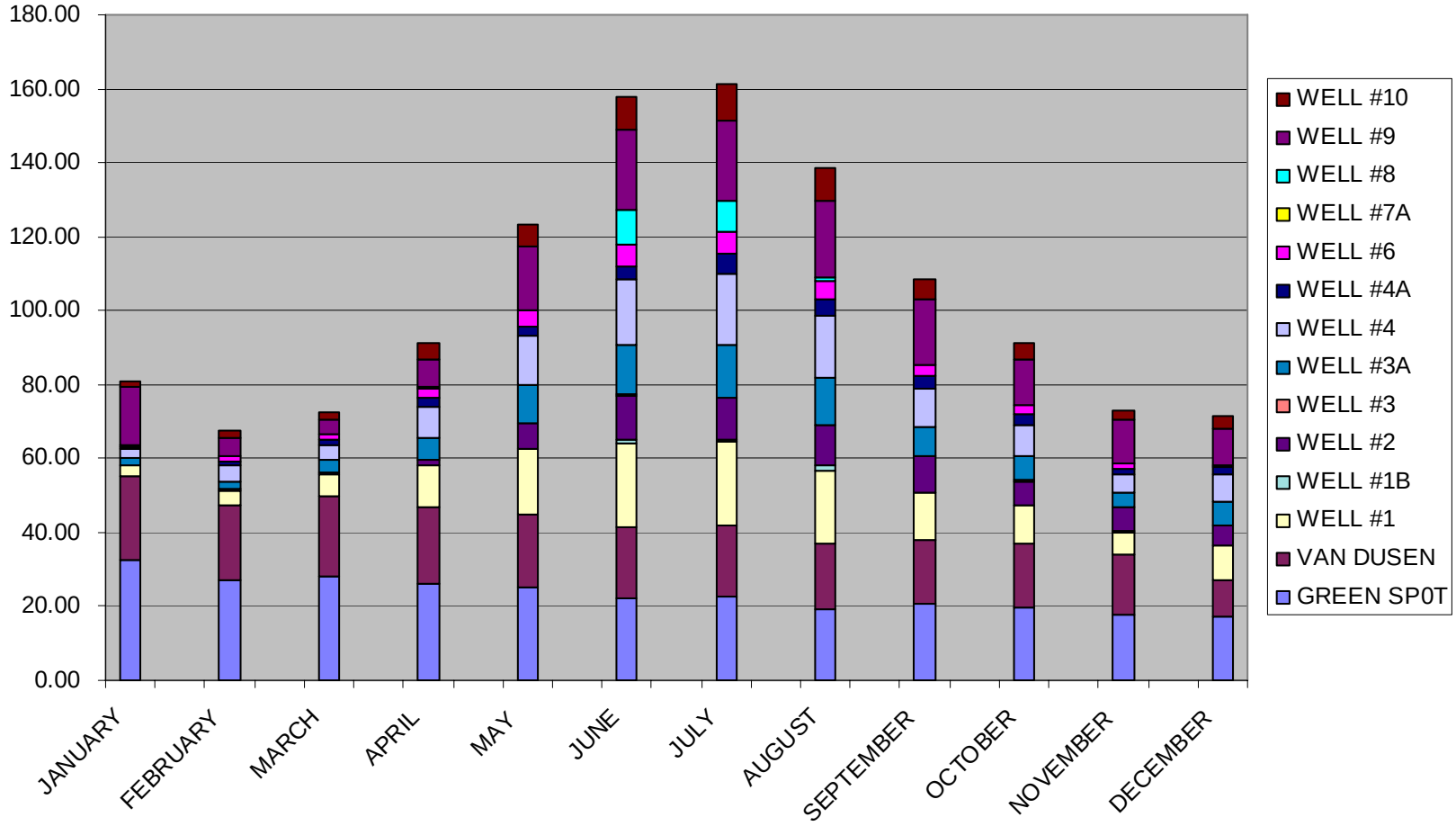

Figure 4-5

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

2007 Water Supplied by BBCCSD

Daniel B. Stephens & Associates, Inc.

1/7/10

Figure 4-6

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Well 10

Daniel B. Stephens & Associates, Inc.

1/7/10

Figure 4-7

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Erwin Ranch Road Intertie Site

Daniel B. Stephens & Associates, Inc.

1/7/10

Figure 4-8

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Greenspot Reservoir

Daniel B. Stephens & Associates, Inc.

1/7/10

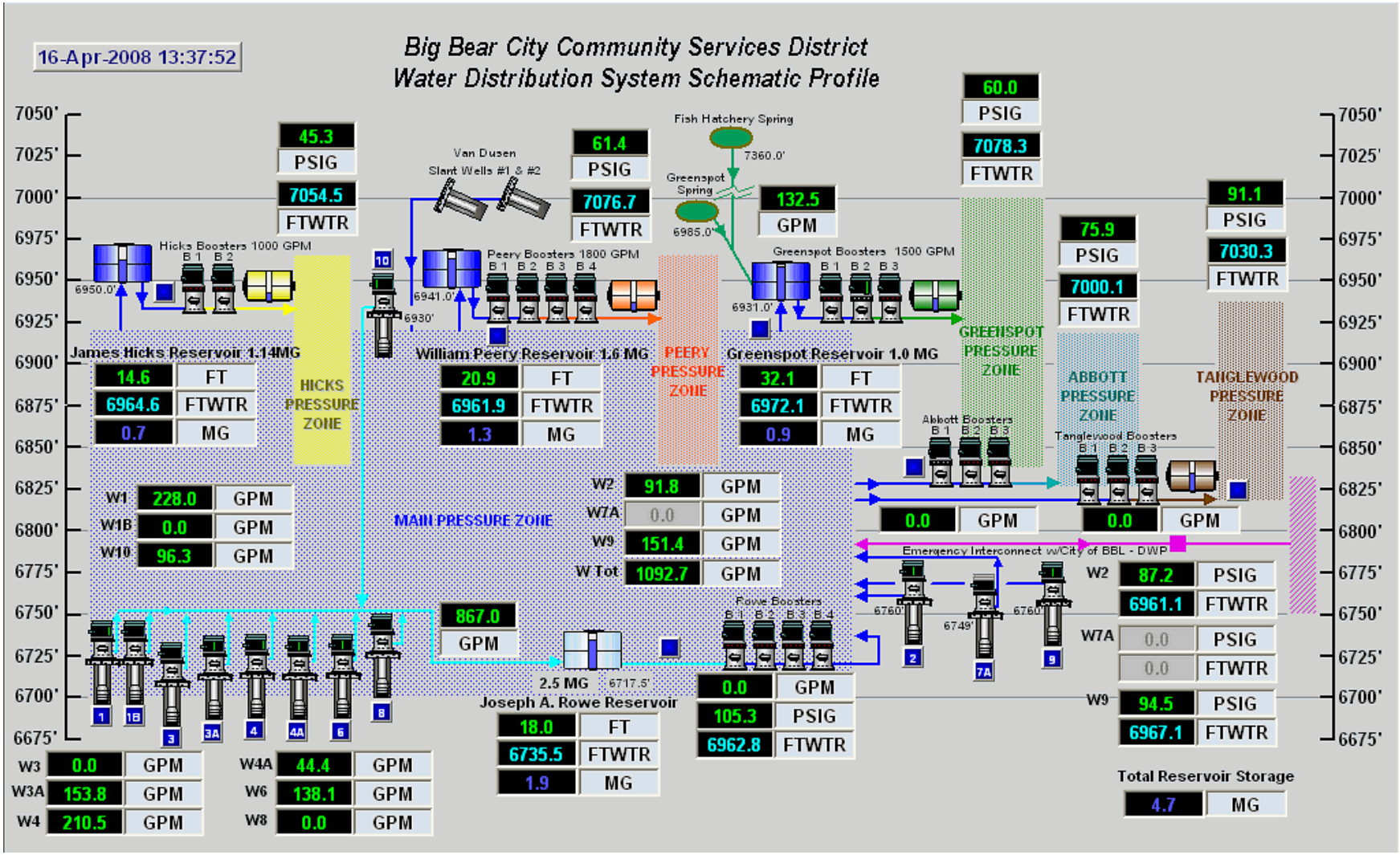

Figure 4-9a

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

SCADA Hydraulic Profile Screenshot

Daniel B. Stephens & Associates, Inc.

1/7/10

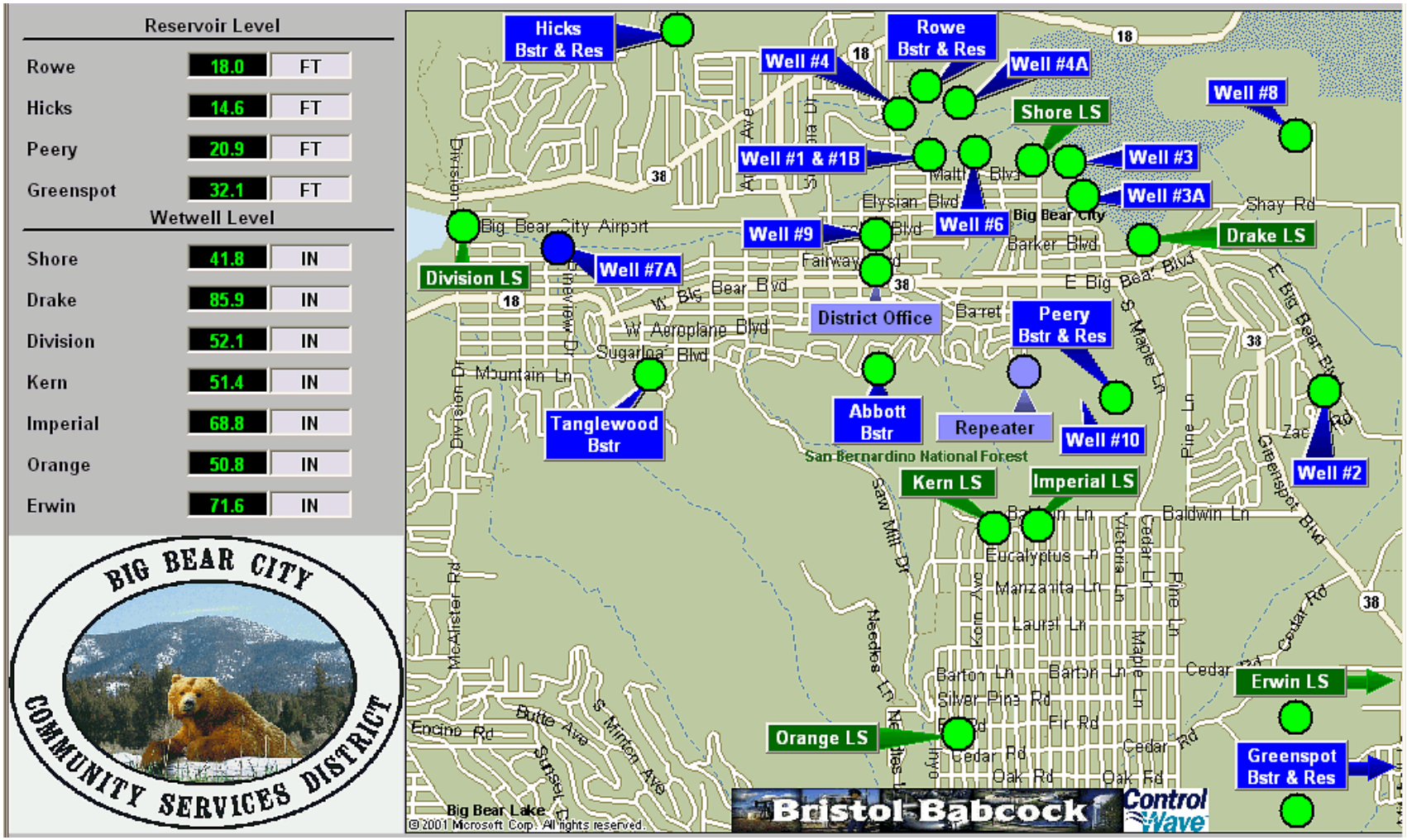

Figure 4-9b

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

SCADA Screenshot Showing Water Levels

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1/7/10

Figure 4-10

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Kinetico Groundwater Treatment System at Well 8

Daniel B. Stephens & Associates, Inc.

1/7/10

Figure 4-11

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Well 2 Chlorine Injection System

Daniel B. Stephens & Associates, Inc.

1/7/10

Figure 4-12

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Greenspot Backup Power Generator

Daniel B. Stephens & Associates, Inc.

1/7/10

Figure 4-13

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Portable Power Generator

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1/7/10

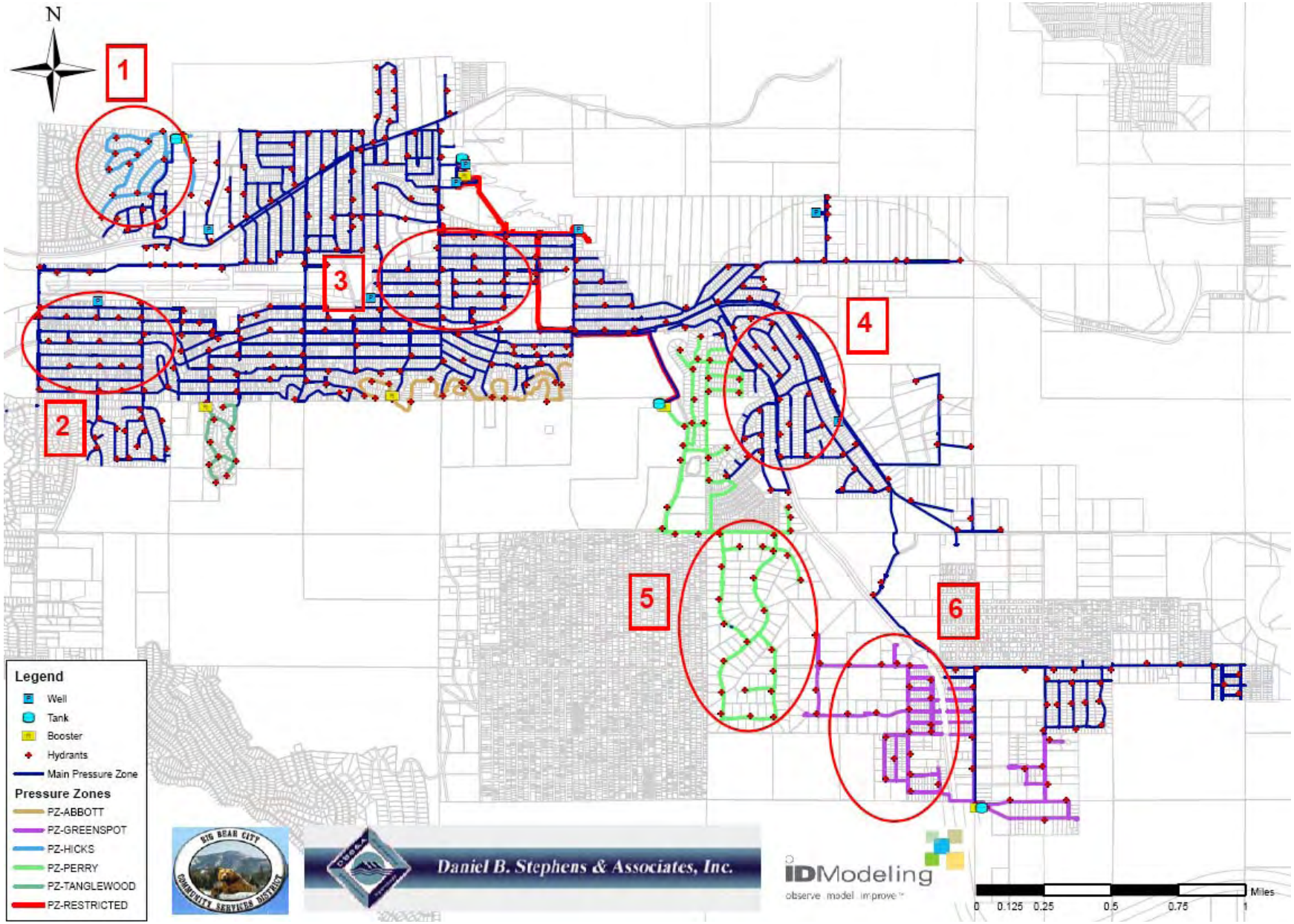

Figure 5-1

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

BBCCSD Hydrant Test Location Map

Daniel B. Stephens & Associates, Inc.

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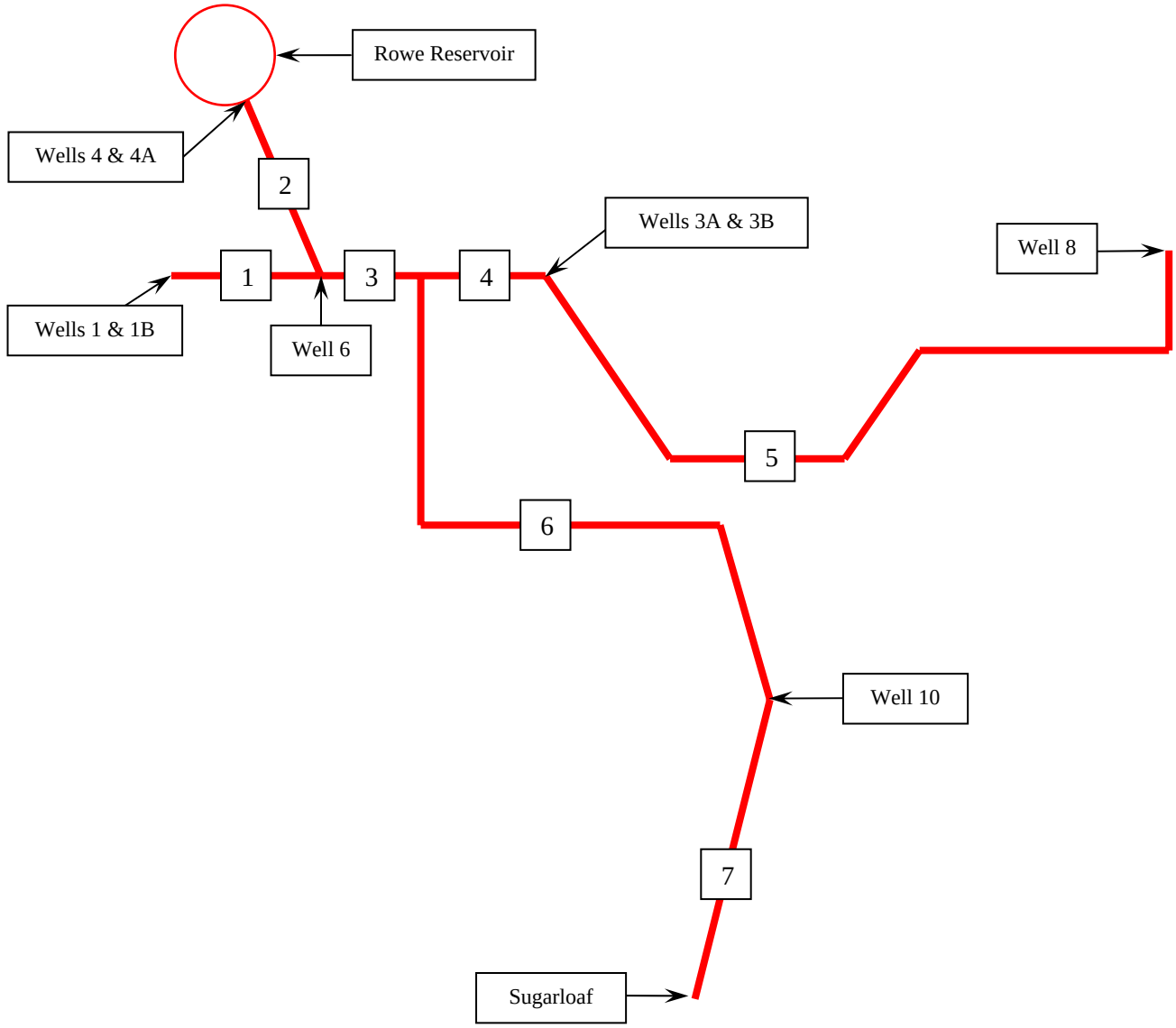

Figure 7-1

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Restricted Main Pipeline Schematic

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1/7/10

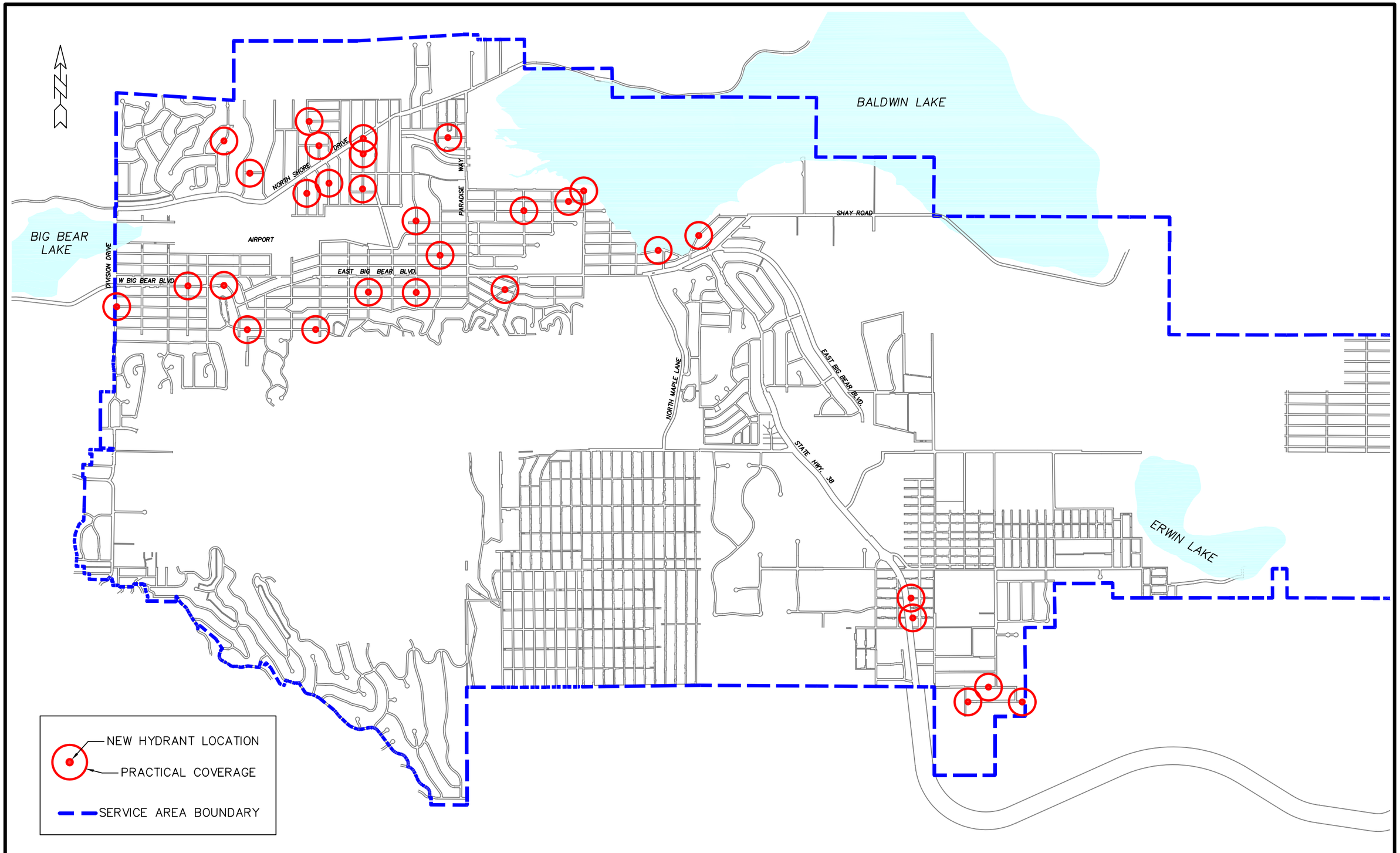
Figure 7-2

BIG BEAR CITY COMMUNITY SERVICE DISTRICT

Well 3B Wellhead

Daniel B. Stephens & Associates, Inc.

1/7/10



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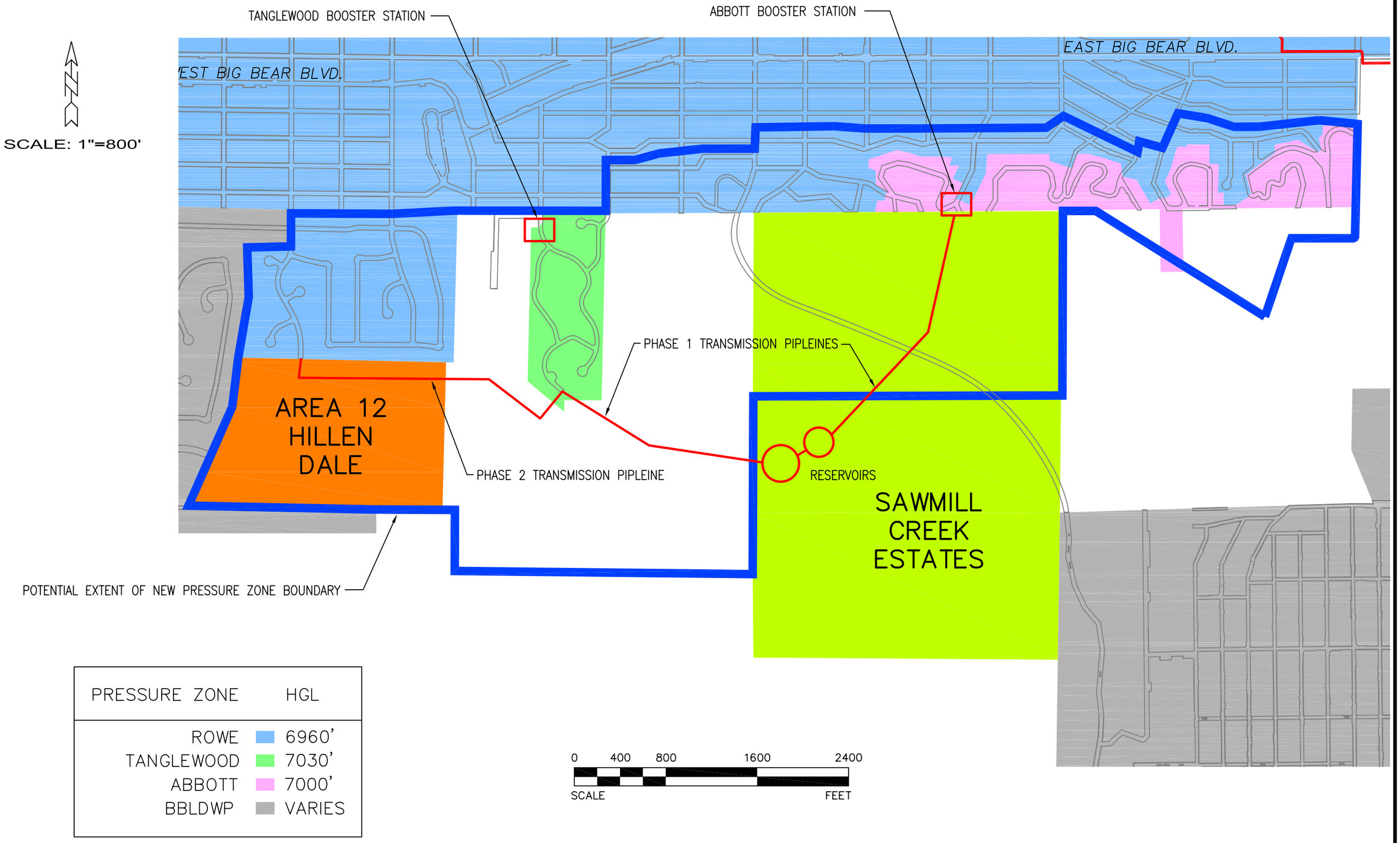
BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

HYDRANT SPACING ANAYSIS
RECOMMENDED NEW HYDRANTS

FIGURE
7-3



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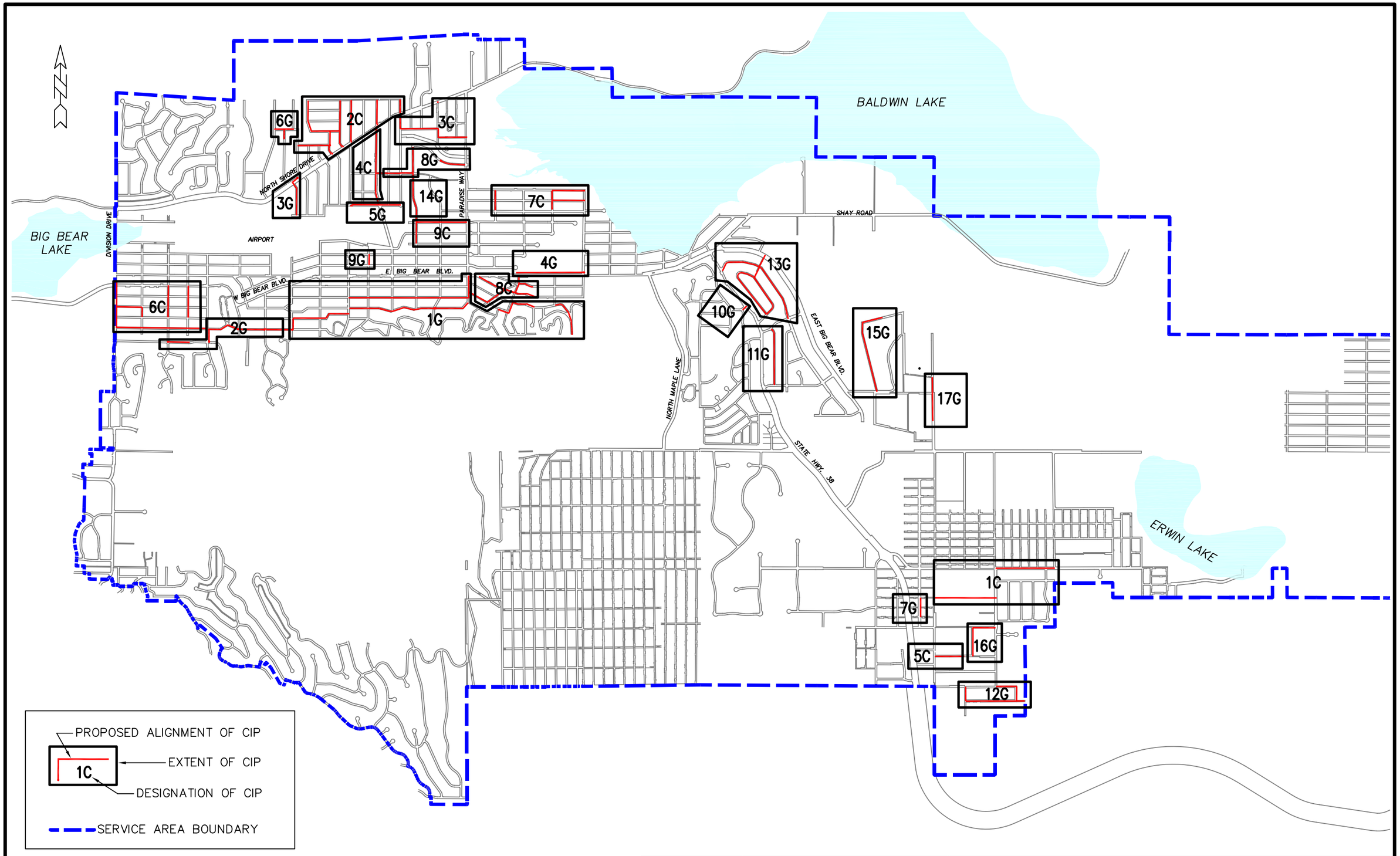
BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

COHILA PRESSURE ZONE CONCEPT

FIGURE
7-4



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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PIPELINE CIP PROJECTS

FIGURE
7-5

Tables

Table 2-1. San Bernardino County Land Use Designations

Code	Name	Density	Minimum Parcel Area	Intended Uses
RS	Single residential	1 unit per 0.25 acre	7,200 ft^2	This district provides sites for single-family residential uses and similar and compatible uses.
RS-20M		1 unit per 20,000 ft^2	20,000 ft^2	
RS-10M		1 unit per 10,000 ft^2	10,000 ft^2	
RS-1		1 unit per 1 acre	1 acre	
RM	Multiple residential	1 unit per 0.05 acre	10,000 ft^2	This district provides sites for multiple residential uses, mixed residential uses, and similar and compatible non-residential uses and activities.
RL	Rural living	1 unit per 2.5 acres	2.5 gross acres	This district provides sites for rural residential uses, incidental agricultural uses, and similar and compatible uses.
RL-5		1 unit per 5 acres	5 gross acres	
RL-10		1 unit per 10 acres	10 gross acres	
RL-20		1 unit per 20 acres	20 gross acres	
RL-40		1 unit per 40 acres	40 gross acres	
RC	Resource conservation	1 unit per 40 acres	40 gross acres	This district provides sites for open space and recreational activities, single-family homes on very large parcels, and similar and compatible uses.
SD-RES	Special development	0.3:1 Floor area ratio (FAR)	40 acres	This district provides sites for a combination of residential, commercial, industrial, agricultural, open space and recreation uses, and similar and compatible uses.
CG	General commercial	0.5:1 (FAR)	5 acres	This district provides sites for retail trade and personal services, lodging services, office and professional services, recreation and entertainment services, wholesaling and warehousing, contract/construction services, transportation services, open lot services, and similar and compatible uses.
CN	Neighborhood commercial	0.25:1 (FAR)	1 acre	This district provides sites for retail trade and personal services, repair services, professional services, recreation and entertainment services, and similar and compatible uses.
CS	Service commercial	0.4:1 (FAR)	5 acres	This district provides sites for a mixture of heavy commercial uses and light industrial uses, including light manufacturing uses, and similar and compatible uses.
IC	Community industrial	0.4:1 (FAR)	5 acres	This district provides sites for light industrial uses such as light manufacturing uses, wholesale/warehouse services, contract/construction services, transportation services, agriculture support services, incidental commercial and accessing residential uses, and similar and compatible uses.
IN	Institutional	0.5:1 (FAR)	None indicated	This district provides sites for public and quasi-public uses facilities and for similar and compatible uses.

Table 2-2. Projected Average Annual Growth Rates for 2000 through 2030

Demographic	Growth Rate (%)
Population	1.8
Households	2.0
Employment	1.6

Table 2-3. BBCCSD Service Area Population Projection

Year	2000	2005	2010	2015	2020	2025	2030
Service population	7,884	8,620	9,424	10,303	11,264	12,315	13,464

Table 2-4. BBCCSD New Service Connection Projection

Year	2007	2010	2015	2020	2025	2030
Connections	5,990	6,035	6,339	6,822	7,342	7,901

Table 2-5. Top 50 Consumers
January 1, 2000 to March 15, 2008
Page 1 of 2

Rank	Account #	Customer	Consumption (ft^3)
1	0160010000	Whispering Pines Est.	16,655,000
2	0180010000	Stickleback	10,785,600
3	0150110000	Big Bear High School	9,892,000
4	0150100000	Chautauqua High School	1,797,000
5	0062211000	Inn Der Bach	1,466,900
6	0101835000	Big Bear Airport District	1,429,000
7	0108000000	Church of Jesus Christ, LDS	881,000
8	0180040000	BBCCSD-Fire Station #292	741,000
9	0087730000	Richard Ollila	647,300
10	0100170000	Thelma's Restaurant	633,900
11	0030025000	Guillermo & Clara A Santiso	628,500
12	0150230000	J L Reed	552,300
13	0073975000	Larry Moore Trust	535,300
14	0105400000	Tom & Michelina Sakai	516,000
15	0180020000	Inn Der Bach	475,500
16	0041220000	BBCCSD - Erwin Lake Park	473,000
17	0115315000	Big Bear Vly Rec & Park	471,000
18	0000003410	Ronald Burkhard	462,000
19	0130030000	Tamio & Michelina Sakai	382,600
20	0061400000	Curt Williamson	367,200
21	0062655000	Bbarwa	365,000
22	0063540000	Cindy Raymond	360,700
23	0041070000	Kathy & Thomas Bradford	359,400
24	0013430000	Cathy's Country Cottages	342,800
25	0105270000	Ken Willis	325,100
26	0113010000	Ann Van Ausdeln	324,800
27	0061450000	John Dykesten	323,200
28	0042410000	Russell Wells	322,700
29	0180050000	BBCCSD-Fire Station #291	310,000
30	0103060000	N E Johnson	309,200
31	0100310000	Morgan Properties	300,200
32	0080990000	Tom & Theresa Spiegel	299,200
33	0023500000	Big Bear B & B	295,000

Note: Yellow shading indicates non-residential customers.
ft^3 = Cubic feet

Table 2-5. Top 50 Consumers
January 1, 2000 to March 15, 2008
Page 2 of 2

Rank	Account #	Customer	Consumption (ft^3)
34	0026790000	Doves	292,300
35	0000003409	Gold Mountain Manor	290,900
36	0075710000	Alex & Lois Dmytriw	276,600
37	0150250000	Bob And Shirley Wood	274,200
38	0075580000	George & Leslie Wilson	272,600
39	0060990000	Kevin Donoho	270,400
40	0170700000	Thomas & Gini Rebber	268,900
41	0105470000	Kim & Kanita Rees	265,800
42	0075480000	Rachel Miller	265,500
43	0075600000	Paul Bedoe	258,000
44	0073675000	Seventh Day Adventist Church	255,800
45	0062560000	Edward E Becker	253,000
46	0107060000	Catrine & Marvin Tadlock	253,000
47	0075180000	John M Stock	251,800
48	0111540000	BBC SawWorks Inc.	249,700
49	0061540000	Kurt Cowell	248,200
50	0052850000	Janet Evans	245,500

Note: Yellow shading indicates non-residential customers.
ft^3 = Cubic feet

Table 2-6. Water Meter Data

Rate Code	Meter Size (inches)	Number of Meters	Total Demand per Rate Code (gpm)	Demand per Meter (gpm)	Percentage of Total Demand (%)
M00	⅝ and ¾	5,898	668.75	0.11	87.1
M01	1	47	14.38	0.31	1.9
M02	1½	3	1.77	0.59	0.2
M03	2	5	7.78	1.56	1.0
M04	3	0	0.00	0.00	0.0
M05	4	1	19.51	19.51	2.5
M06	6	1	35.62	35.62	4.6
M07		20	6.13	0.31	0.8
M08		2	1.42	0.71	0.2
M99		10	11.91	1.19	1.6
CMtr [a]	NA	2	0.12	0.06	0.0
Well [b]	NA	1	0.11	0.11	0.0

[a] Construction meters are temporary by their nature and not necessarily associated with a specific meter size or typical demand.

[b] Private well at 1001 West Paradise Way.

gpm = Gallons per minute

Table 2-7. Water Loss Analysis

	2001	2002	2003	2004	2005	2006	2007
Production (ac-ft)	1,233	1,224	1,102	1,162	1,101	1,126	1,238
Consumption (ac-ft)	1,186	1,147	1,039	1,064	986	1,005	1,099
Loss (ac-ft)	47	77	64	97	115	121	139
Loss (%)	3.8	6.3	5.8	8.4	10.5	10.7	11.2

ac-ft = Acre-feet

Table 2-8. Water Duty Factors

Code	Designation	Water Duty Factor (ac-ft/yr per acre)	Average Meter Demand (ac-ft/yr per unit)
RS	Single-family residential	1.35	0.17
RS-1	Single-family residential	0.47	0.43
RS-10M	Single-family residential	0.56	0.13
RS-20M	Single-family residential	0.52	0.29
RM	Multiple residential	1.53	0.24
RL [a]	Rural living	0.085	0.21
RL-5 [a]	Rural living	0.042	0.21
RL-10 [a]	Rural living	0.021	0.21
RL-20 [a]	Rural living	0.011	0.21
RL-40 [a]	Rural living	0.0053	0.21
SD-RES	Special development	0.88	0.17
CG	General commercial	2.45	0.25
CN [b]	Neighborhood commercial	1.73	0.17
CS	Service commercial	1.42	0.14
IC	Community industrial	1.73	NA
IN	Institutional	0.56	NA
RC	Resource conservation	0.00	NA

Table 2-9. Summary of Build-Out Data

Code [a]	Area (acres)	Density (connections per acre)	Build-Out Connections	Water Duty Factor (ac-ft/yr per acre)	Demand (ac-ft/yr)
RS	1,409	4	5,635	1.35	1,897.8
RS-1	556	1	556	0.47	262.8
RS-10M	60	4	239	0.56	33.7
RS-20M	307	2	614	0.52	158.4
RM	77	16	1,239	1.53	118.3
RL	321	0.025	8	0.04	14.2
RL-5	415	0.2	83	0.042	17.6
RL-10	38	0.1	4	0.021	0.8
RL-20	665	0.05	33	0.011	7.0
RL-40	199	0.025	5	0.0053	1.1
SD-RES	252	3	756	0.88	220.9
RC	1,171	0.025	29	0.0053	6.2
Residential subtotal	5,470		9201		2,739
CG	119	10	1,195	2.45	293.1
CN	1	10	7	1.94	1.4
CS	41	10	409	1.42	58.2
IC	24	10	240	1.94	46.6
IN	218	NA	NA	0.56	122.5
Non-residential subtotal	403		1,851		522
Total	5,873		11,052		3,261

[a] Explanations of land use codes are provided in Table 2-1

ac-ft/yr = Acre-feet per year
NA = Not applicable

Table 2-10. Near-Term Development within BBCCSD Service Area

Development	Units
Wooden Door Estates	37
Big Bear Pines Residential Community	66
Saw Mill Drive	242
Total	345

Table 2-11. Projected Ultimate (2030) Development within BBCCSD Service Area

Area	Projected Development	Area (acres)	Land Use [a]	Density (units/acre)	Total Units	Demand (ac-ft/yr)	Notes
1	North Shore Drive & Live Oak Place	37.0	RS	4	148	29.6	SBC land use density
2	North Shore Drive & Live Oak Place	4.9	RS-1	1	5	1.0	SBC land use density
3	Greenspot Road & Erwin Ranch Road	14.8	CG	1.5	22	4.4	Density similar to adjacent lots
4	Pine Lane & Amber Park Drive	15.0	RS	4	60	12.0	SBC land use density
5	Shore Drive & E. Big Bear Blvd.	24.8	CG	4	99	19.8	Density similar to adjacent lots
6	Northeast of E. Big Bear Blvd.	304.8	RS-1	0.4	122	24.4	Estimated to be 60% built-out
7	Erwin Ranch Road & Cypress Lane	83.8	RS-1	1	84	16.8	SBC land use density
8	Shay Road Tract 1730 PM 7114	4.4	RS	7.9	35	7.0	Density based on number of lots
9	Minnow Lane Tract 1730	1.5	RS	10.5	16	3.2	Density based on number of lots
10	Vicinity of Erwin Lake	186.5	SD-RES	0.2	30	6.0	Density based on number of lots
11	Greenspot and Erwin Lane	38.8	SD-RES	3.5	136	27.2	Adopted density of Tanglewood SD
12	Hillen Dale Lane	59.1	RS	4	237	47.4	SBC land use density
13	State Lane & Willow Lane	20.6	RS-20M	2.178	45	9.0	SBC land use density
14	Section 16 Development	118.5	RS	4	474	94.8	SBC land use density
15	Greenspot Road & Mitchell Lane	10.0	CG	1	10	2.0	Density similar to adjacent lots
16	N. Maple Lane & E. Big Bear Blvd.	9.0	RS-10M	4.356	39	7.8	SBC land use density
	Densification and rural development				706	141.2	
				Total	*1,562*	*312.4*	

[a] Explanations of land use codes are provided in Table 2-1
ac-ft/yr = Acre-feet per year

Table 2-12. Projected Water Requirements

Planning Period	Connections	Average Day Demand (ac-ft/yr)
Existing	5,990	1,238.0
Near-term	6,335	1,307.0
Ultimate	7,901	1,620.2

ac-ft/yr = Acre-feet per year

Table 2-13. Demand Fluctuation Summary

Scenario	Factor	Demand			Period
		ac-ft/d	mgd	gpm	
Average daily demand	1.00	3.39	0.15	767.5	2007
Maximum day demand [a]	1.95	6.63	0.29	1500.2	Thursday, July 19, 2007
Peak hour demand	3.90	13.26	0.58	3,000.4	NA
Minimum day demand [b]	0.43	1.47	0.06	331.8	Saturday, February 17, 2007

[a] The peaking factor for maximum day demand (MDD) was calculated at 1.95, which was then rounded up to 2.0 to simplify certain applications and to give a slightly more conservative estimate. See Appendix A for methodology and calculations.

[b] The peaking factor for minimum day demand was calculated at 0.433, which was then rounded down to 0.4 to simplify certain applications and to give a slightly more conservative estimate. See Appendix A for methodology and calculations.

ac-ft/d = Acre-feet per day
mgd = Million gallons per day
gpm = Gallons per minute
NA = Not applicable

Table 3-1. Sources of Data for Water Balance Model
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Parameter	Published Site-Specific Data	Literature Data	Estimated from Other Properties	Comment
Soils				
Saturated hydraulic conductivity	X			USDA, 2008
Soil texture	X			USDA, 2008
Water retention characteristics			X	Estimated with Rosetta (Schapp et al., 2001)
Soil depth	X			USDA, 2008
Bedrock				
Saturated hydraulic conductivity		X		Brassington, 1988 Maidment, 1993 USDA, 2008 Belcher et al., 2002
Unit map	X			Dibblee, 1971
Vegetation				
Mean maximum root depth		X		Rundel and Gibson, 1996 Schenk and Jackson, 2002
Mean maximum height		X		Rundel and Gibson, 1996 Tirmenstein, 1999
Leaf area index (LAI)		X		DBS&A, 2008 Breuer and Frede, 2003
Phenology		X		Appears to correspond with NTS studies (Rundel and Gibson, 1996)
Vegetation type and distribution	X			GAP study (USGS, 2008a; UCSB, 2008) DBS&A site visits
Stomatal resistance		X		Huxman et al., 1999 Huxman and Smith, 2001 Naumburg et al., 2003 Hamerlynck et al., 2000a Hamerlynck et al., 2000b Hamerlynck et al., 2002 Hamerlynck et al., 2004 Pataki et al., 2000 Smith et al., 1995
Climate				
Precipitation	X			DAYMET, 2007
Air temperature	X			DAYMET, 2007
Precipitation intensity			X	CIMIS, 2009, #199
Climate (cont.)				
Wind speed			X	CIMIS, 2009, #199
Snow sublimation rate		X		Hood et al., 1999

Table 3-1. Sources of Data for Water Balance Model
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Parameter	Published Site-Specific Data	Literature Data	Estimated from Other Properties	Comment
Snowmelt rate			X	Average of values from 12/1/06 through 4/1/07 using HELP model methodology (Schroeder et al., 1994)
Precipitation lapse			X	Estimated from DAYMET, 2007
Temperature lapse			X	Estimated from DAYMET, 2007
Geography				
Ground elevation	X			USGS, 2008b
Basin boundary	X			USGS, 2008b

Table 3-2. Soils Data for the Bear Valley Basin Reported by U.S. Department of Agriculture

MUKEY	MUSYM	Area (acres)	Percent of Basin Area (%)	Depth to Bedrock (cm)	Uppermost Layer K_{sat} (cm/s)	Percent Sand (%)	Percent Silt (%)	Percent Clay (%)	Percent Bare Rock (%)
471613	BoD	6,102	14.58	201	9.0×10^{-5}	39.1	36.9	24.0	0
471614	BoE	4,579	10.94	201	9.0×10^{-5}	39.1	36.9	24.0	0
471622	DaE	3,512	8.39	38	9.2×10^{-4}	84.0	11.0	5.0	0
471623	DaF	4,047	9.67	38	9.2×10^{-4}	84.0	11.0	5.0	0
471624	DaG	479	1.14	25	9.2×10^{-4}	78.5	16.5	5.0	0
471627	DdDE	4,110	9.82	38	9.2×10^{-4}	84.0	11.0	5.0	0
471630	DhG	58	0.14	46	9.2×10^{-4}	78.5	16.5	5.0	0
471637	DxF	211	0.50	25	9.2×10^{-4}	78.5	16.5	5.0	0
471638	DxG	27	0.07	25	9.2×10^{-4}	78.5	16.5	5.0	0
471640	FaE	619	1.48	66	2.8×10^{-4}	65.9	19.1	15.0	0
471641	FaF	2,327	5.56	66	2.8×10^{-4}	65.9	19.1	15.0	0
471642	FbE	1,492	3.56	76	2.8×10^{-4}	44.3	40.7	15.0	0
471643	FbF	1,611	3.85	76	2.8×10^{-4}	44.3	40.7	15.0	0
471645	FLG	49	0.12	114	9.2×10^{-4}	84.0	11.0	5.0	0
471646	FrE	64	0.15	117	9.0×10^{-5}	68.8	16.2	15.0	0
471647	FrF	3,491	8.34	38	9.0×10^{-5}	68.8	16.2	15.0	0
471651	HoD	1,322	3.16	201	9.0×10^{-5}	39.1	36.9	24.0	0
471652	HoE	634	1.51	201	9.0×10^{-5}	39.1	36.9	24.0	0
471656	LcF	323	0.77	38	9.0×10^{-5}	68.8	16.2	15.0	0
471661	MbF	84	0.20	127	9.0×10^{-5}	44.3	40.7	15.0	0
471664	PsD	2,846	6.80	201	9.2×10^{-4}	84.0	11.0	5.0	0
471666	Rs	50	0.12	0	No data	No data	No data	No data	100
471670	SgF	2,810	6.71	66	2.8×10^{-4}	65.9	19.1	15.0	0
471671	SgG	867	2.07%	66	2.8×10^{-4}	65.9	19.1	15.0	0
471676	WpG	137	0.33%	102	9.2×10^{-4}	69.1	23.9	7.0	0

MUKEY = Map unit key (SSURGO database)
MUSYM = Map unit symbol (SSURGO database)
cm = Centimeter(s
K_{sat} = Saturated hydraulic conductivity
cm/s = Centimeters per second

Table 3-3. Soil Properties Predicted by Rosetta for Bear Valley Basin Soils

USDA Map Symbol	MUSYM	Residual Water Content	Saturated Water Content	van Genuchten		Saturated Hydraulic Conductivity (cm/s)
				α (1/cm)	n	
471613	BoD	0.0693	0.4140	0.0113	1.4594	8.67×10^{-5}
471614	BoE	0.0693	0.4140	0.0113	1.4594	8.67×10^{-5}
471622	DaE	0.0434	0.3833	0.0391	1.927	1.67×10^{-3}
471623	DaF	0.0434	0.3833	0.0391	1.927	1.67×10^{-3}
471624	DaG	0.0378	0.3874	0.0424	1.6353	1.02×10^{-3}
471627	DdDE	0.0434	0.3833	0.0391	1.927	1.67×10^{-3}
471630	DhG	0.0378	0.3874	0.0424	1.6353	1.02×10^{-3}
471637	DxF	0.0378	0.3874	0.0424	1.6353	1.02×10^{-3}
471638	DxG	0.0378	0.3874	0.0424	1.6353	1.02×10^{-3}
471640	FaE	0.0510	0.3835	0.0301	1.3816	3.51×10^{-4}
471641	FaF	0.0510	0.3835	0.0301	1.3816	3.51×10^{-4}
471642	FbE	0.0528	0.3963	0.0101	1.4996	1.64×10^{-4}
471643	FbF	0.0528	0.3963	0.0101	1.4996	1.64×10^{-4}
471645	FLG	0.0434	0.3833	0.0391	1.927	1.67×10^{-3}
471646	FrE	0.0514	0.3815	0.0315	1.3899	3.79×10^{-4}
471647	FrF	0.0514	0.3815	0.0315	1.3899	3.79×10^{-4}
471651	HoD	0.0693	0.4140	0.0113	1.4594	8.67×10^{-5}
471652	HoE	0.0693	0.4140	0.0113	1.4594	8.67×10^{-5}
471656	LcF	0.0514	0.3815	0.0315	1.3899	3.79×10^{-4}
471661	MbF	0.0528	0.3963	0.0101	1.4996	1.64×10^{-4}
471664	PsD	0.0434	0.3833	0.0391	1.927	1.67×10^{-3}
471666	Rs	NA	NA	NA	NA	NA
471670	SgF	0.0510	0.3835	0.0301	1.3816	3.51×10^{-4}
471671	SgG	0.0510	0.3835	0.0301	1.3816	3.51×10^{-4}
471676	WpG	0.0366	0.3883	0.0362	1.4335	6.18×10^{-4}

USDA = U.S. Department of Agriculture
MUSYM = Map unit symbol (SSURGO database)
cm = Centimeter(s)
cm/s = Centimeters per second

Table 3-4. Saturated Hydraulic Conductivity Values for Bedrock Hydrogeologic Units

Formation	Abbreviation	Description	Saturated Hydraulic Conductivity		
			cm/s	m/s	ft/d
Precambrian gneissic rocks	gn	Gneiss	1.0×10^{-6}	1.0×10^{-8}	2.6×10^{-3}
Mesozoic plutonic rocks and Paleozoic quartzite	hd	Hornblende diorite	2.0×10^{-8}	2.0×10^{-10}	5.7×10^{-5}
	hqm	Hornblende quartz monzonite	2.0×10^{-8}	2.0×10^{-10}	5.7×10^{-5}
	mq	Quartzite	2.0×10^{-8}	2.0×10^{-10}	5.7×10^{-5}
	pqm	Porphyritic quartz monzonite	2.0×10^{-8}	2.0×10^{-10}	5.7×10^{-5}
	qd	Quartzite diorite and granodiorite	2.0×10^{-8}	2.0×10^{-10}	5.7×10^{-5}
	qm	Quartz monzonite	2.0×10^{-8}	2.0×10^{-10}	5.7×10^{-5}
Paleozoic metamorphic rocks and Mesozoic porhpyry complex	ml	Limestone and dolomite marble	3.66×10^{-8}	3.66×10^{-8}	1.0×10^{-2}
	ms	Schist and phyllite	3.66×10^{-6}	3.66×10^{-8}	1.0×10^{-2}
	pc	Porphyritic andesite to quartz latite	3.66×10^{-6}	3.66×10^{-8}	1.0×10^{-2}
Quaternary clay	Qc	Playa clay	9.0×10^{-6}	9.0×10^{-8}	2.6×10^{-2}
Quaternary coarse grain deposits	Qa	Alluvium	9.0×10^{-4}	9.0×10^{-6}	2.6
	Qf	Alluvial fan gravel	9.0×10^{-4}	9.0×10^{-6}	2.6
	Qoa	Older alluvium	9.0×10^{-4}	9.0×10^{-6}	2.6
	Qof	Older alluvial fan gravel	9.0×10^{-4}	9.0×10^{-6}	2.6

cm/s = Centimeters per second
m/s = Meters per second
ft/d = Feet per day

Table 3-5. Gap Analysis Project Codes and Ecological Systems for Big Bear Valley Watershed Vegetation Types

GAP Code	Percent Area of Basin (%)	Vegetation Association	GAP Ecological System
AL	3.22	Bromus–graminoid (grassland)	Agricultural land
BSS	2.97	Artemisia (sagebrush)	Big sagebrush scrub
ILOC	0.15	Quercus (scrub oak)	Interior live oak chaparral
JPF	7.62	Pinus (pine)	Jeffrey pine forest
JPFF	40.71	Pinus (pine) / Abies (fir)	Jeffrey pine-fir forest
MMC	0.19	Artemisia (sagebrush)	Mixed montane chaparral
MPJW	30.63	Piñon–juniper	Mojavean piñon and juniper woodlands
NNG	0.75	Bromus–graminoid (grassland)	Non-native grassland
SOC	0.08	Quercus (scrub oak)	Scrub oak chaparral
SMCF	11.54	Pinus (pine) / Abies (fir)	Sierran mixed coniferous forest
SCSF	1.75	Pinus (pine)	Southern California subalpine forest
SCWF	0.39	Abies (fir)	Southern California white fir forest

GAP = Gap Analysis Project (U.S. Geological Survey)

Table 3-6. Rooting Depths of Vegetation Associations

Vegetation Association	USGS Map Code	Percent Area of Basin (%)	Rooting Depth (meters)	Source
Agricultural land Bromus–graminoid (grassland)	AGRI	3.22	1.04	Geometric mean maximum rooting depth for grasses in water-limited environments (Schenk and Jackson, 2002)
Big sagebrush scrub Artemisia (sagebrush)	BSG	2.97	2.14	Geometric mean maximum rooting depth for shrubs in water-limited environments (Schenk and Jackson, 2002)
Interior live oak chaparral Quercus (scrub oak)	ILO	0.15	2.14	Geometric mean maximum rooting depth for shrubs in water-limited environments (Schenk and Jackson, 2002)
Jeffrey pine forest Abies (fir)	JFP	7.62	1.50	Rundel and Gibson, 1996, pp. 98-99
Jeffrey pine-fir forest Abies (fir)	JPNF	40.71	1.50	Rundel and Gibson, 1996, pp. 98-99
Mixed montane chaparral Artemisia (sagebrush)	MXMT	0.19	2.14	Geometric mean maximum rooting depth for shrubs in water-limited environments (Schenk and Jackson, 2002)
Mojavean piñon and juniper woodlands Piñon–juniper	PJP	30.63	3.27	Geometric mean maximum rooting depth for trees in water limited environments (Schenk and Jackson, 2002)
Non-native grassland Bromus–graminoid (grassland)	GRS	0.75	1.04	Geometric mean maximum rooting depth for grasses in water-limited environments (Schenk and Jackson, 2002)
Scrub oak chaparral Quercus (scrub oak)	SOC	0.08	2.14	Geometric mean maximum rooting depth for shrubs in water-limited environments (Schenk and Jackson, 2002)
Sierran mixed coniferous forest Abies (fir)	SMC	11.54	1.50	Rundel and Gibson, 1996, pp. 98-99
Southern California subalpine forest Abies (fir)	SCS	1.75	1.50	Rundel and Gibson, 1996, pp. 98-99
Southern California white fir forest Abies (fir)	SCWF	0.39	1.50	Rundel and Gibson, 1996, pp. 98-99

USGS = U.S. Geological Survey

Table 3-7. Mean Leaf Stomatal Resistance

Vegetation Association	USGS Map Code	Percent Area of Basin (%)	Mean Leaf Stomatal Resistance (s/m)			Source
			Initial	Mid-Season	Late	
Agricultural land Bromus–graminoid (grassland)	AL	3.22	352	123	352	Average for Bromus
Big sagebrush scrub Artemisia (sagebrush)	BSS	2.97	390	233	695	Average of values for drought deciduous and evergreen species
Interior live oak chaparral Quercus (scrub oak)	ILOC	0.15	100	100	100	Assume FAO-56 reference value
Jeffrey pine forest Pinus (pine)	JPF	7.62	100	100	100	Assume FAO-56 reference value
Jeffrey pine-fir forest Pinus (pine)/Abies (fir)	JPFF	40.71	100	100	100	Assume FAO-56 reference value
Mixed montane chaparral Artemisia (sagebrush)	MMC	0.19	390	233	695	Average of values for drought deciduous and evergreen species
Mojavean piñon and juniper woodlands Piñon - juniper	MPJW	30.63	100	100	100	Assume FAO-56 reference value
Non-native grassland Bromus - graminoid (grassland)	NNG	0.75	352	123	352	Average for Bromus
Scrub oak chaparral Quercus (scrub oak)	SOC	0.08	100	100	100	Assume FAO-56 reference value
Sierran mixed coniferous forest Pinus (pine)/Abies (fir)	SMCF	11.54	100	100	100	Assume FAO-56 reference value
Southern California subalpine Pinus (pine)	SCSF	1.75	100	100	100	Assume FAO-56 reference value
Southern California white fir forest Abies (fir)	SCWF	0.39	100	100	100	Assume FAO-56 reference value

USGS = U.S. Geological Survey
s/m = Seconds per meter
NA = Not applicable

Table 3-8. Mean Maximum Plant Height

Vegetation Association	USGS Map Code	Percent Area of Basin (%)	Mean Maximum Plant Height (meters)	Source
Agricultural land Bromus–graminoid (grassland)	AL	3.22	0.3	Rundel and Gibson, 1996
Big sagebrush scrub Artemisia (sagebrush)	BSS	2.97	0.6	Rundel and Gibson, 1996
Interior live oak chaparral Quercus (scrub oak)	ILOC	0.15	6.1	UCSB, 1998
Jeffrey pine forest Abies (fir)	JPF	7.62	60	Holland, 1986
Jeffrey pine-fir forest Abies (fir)	JPFF	40.71	60	Holland, 1986
Mixed montane chaparral Artemisia (sagebrush)	MMC	0.19	0.6	Rundel and Gibson, 1996
Mojavean piñon and juniper woodlands Piñon–juniper	MPJW	30.63	5	Assumed maximum
Non-native grassland Bromus–graminoid (grassland)	NNG	0.75	0.3	Rundel and Gibson, 1996
Scrub oak chaparral Quercus (scrub oak)	SOC	0.08	6.1	Tirmenstein, 1999
Sierran mixed coniferous forest Abies (fir)	SMCF	11.54	60	Holland, 1986
Southern California subalpine forest Abies (fir)	SCSF	1.75	10	Holland, 1986
Southern California white fir forest Abies (fir)	SCWF	0.39	60	Holland, 1986

USGS = U.S. Geological Survey

Table 3-9. Leaf Area Index and Phenology

Vegetation Association	USGS Map Code	Percent Area of Basin (%)	Leaf Area Index				Start Development	Start Mid-Season	Start Late Season	Start Dormancy
			Maximum Peak	Average Peak	Minimum Peak	Average Dormant				
Agricultural land Bromus - graminoid (grassland)	AL	3.22	4.10	0.91	0.40	0.35	Mar	Apr	Oct	Nov
Big sagebrush scrub Artemisia (sagebrush)	BSS	2.97	2.20	0.93	0.60	0.47	Mar	Apr	Oct	Nov
Interior live oak chaparral Quercus (scrub oak)	ILOC	0.15	2.60	1.00	0.60	0.44	Oct	Jan	Jun	Jul
Jeffrey pine forest Abies (fir)	JPF	7.62	13.26	13.26	13.26	13.26	Mar	Jun	Aug	Nov
Jeffrey pine-fir forest Abies (fir)	JPFF	40.71	14.43	14.43	14.43	14.43	Mar	Jun	Aug	Nov
Mixed montane chaparral Artemisia (sagebrush)	MMC	0.19	2.20	0.93	0.60	0.47	Mar	Apr	Oct	Nov
Mojavean piñon and juniper woodlands (Piñon - juniper)	MPJW	30.63	2.60	0.92	0.60	0.45	Oct	Jan	Jul	Sep
Non-native grassland Bromus - graminoid (grassland)	NNG	0.75	4.10	0.91	0.40	0.35	Mar	Apr	Oct	Nov
Scrub oak chaparral Quercus (scrub oak)	SOC	0.08	2.60	1.00	0.60	0.44	Oct	Jan	Jun	Jul
Sierran mixed coniferous forest Abies (fir)	SMCF	11.54	14.43	14.43	14.43	14.43	Mar	Jun	Jul	Nov
Southern California subalpine forest Abies (fir)	SCSF	1.75	14.40	14.40	14.40	14.40	Mar	Jun	Jul	Nov
Southern California white fir forest Abies (fir)	SCWF	0.39	15.50	15.50	15.50	15.50	Mar	Jun	Aug	Nov

Sources: DBS&A, 2008; Breuer and Frede, 2003

USGS = U.S. Geological Survey

Table 3-10. General Model Parameters
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Parameter	Variable	Value	Units	Comment
Temperature lapse rate (dry adiabatic lapse rate)	CTcor	0.0094	ºC/m	From linear regression of DAYMET data
Elevation of reference weather station	elevref	6963	ft msl	Elevation of CIMIS station #199 (Big Bear Lake)
Average elevation for entire basin	elevavg	7332	ft msl	Average of USGS DEM cells within basin
Average latitude for entire basin	Latavg	34.253	degrees	Approximate basin midpoint
Precipitation coefficient of elevation change	CPrecipcor	0.003	mm/m	From linear regression of DAYMET data
Adjustment coefficient in Hargreaves' radiation formula	K_{rs}	0.19	ºC-0.5	Standard value near a large body of water
Evaporation layer depth (m)	Z_e	0.1	meters	Depth of the surface soil layer that is subject to drying by way of evaporation. Upper end of range in Allen et al. (1998), p. 144 (ranges 0.10 to 0.15 meters)
Readily evaporable water (mm)	REW	8	mm	Upper end of range for loamy sand (see Table 19 in Allen et al., 1998, p. 144)
Initial capillary head node 1	IC1	61293	cm	Set to wilting point
Initial capillary head node 2	IC2	61293	cm	Set to wilting point
Initial capillary head node 3	IC3	61293	cm	Set to wilting point
Initial capillary head node 4	IC4	340.5167	cm	Set to field capacity
Depletion factor: Average fraction of total available soil water (TAW) that can be depleted from the root zone before moisture stress (reduction in evapotranspiration) occurs (0 to 1).	p	0.5	fraction	Varies 0 to 1 but typically ranges 0.30 for shallow rooted plants at high crop evapotranspiration (ET_c) (>8mm/d) to 0.70 for deep rooted plants at low values of ET_c (<3mm/d) with 0.5 in common use.
Terrain albedo	albedo	0.23	NA	Reference albedo for green vegetation (Allen et al., 1998, p. 43)
Fraction of snowfall that eventually sublimates	Csublime	0.15	NA	Hood et al., 1999, p. 1794
Coefficients describing snow melting with temperature	Csnowmelt	2.6	mm/d/ºC	Average from 12/1/06 through 4/1/07 using Equation 40 in HELP manual (Schroeder et al., 1994)

°C/m = Degrees Celsius per meter
ft msl = Feet above mean sea level
USGS = U.S. Geological Survey
DEM = Digital elevation model
mm/m = Millimeters per meter
mm = Millimeters
cm = Centimeters
NA = Not applicable

Table 3-10. General Model Parameters

Parameter	Variable	Value	Units	Comment
Crop canopy coefficient	fc_switch	–1	NA	–1 = calculate internally by Allen et al., 1998, Equation 76
Minimum transpiration coefficient (Kc) for dry surface soil (upper 0.10 to 0.15 meter) with no vegetation cover	Kc_min	0	NA	0 recommended by Allen et al. (1998) for arid environments
Field capacity pressure head	head_fc	340.5167	cm	⅓bar
Wilting point pressure head	head_wp	61,293	cm	60 bar

°C/m = Degrees Celsius per meter
ft msl = Feet above mean sea level
USGS = U.S. Geological Survey
DEM = Digital elevation model
mm/m = Millimeters per meter
mm = Millimeters
cm = Centimeters
mm/d/°C = Millimeters per day per degree Celsius
NA = Not applicable

Table 3-11. DPWM Subunit Results

Subunit	Result (acre-feet)					
	Precipitation	Actual ET	Recharge	Net Runoff	Change in Storage	Snow Sublimation
Big Bear Lake						
Division	1,530	1,123	296	-15	26	100
Gray's Landing	1,645	791	6	732	0	117
Grout Creek	9,419	4,855	1,394	2,471	29	671
Mill Creek	8,458	4,776	980	2,105	2	596
North Shore	7,145	3,439	2,609	547	60	491
Rathbone	10,951	6,249	3,498	384	21	800
Village	4,241	2,655	1,120	164	10	293
Big Bear Lake totals	43,391	23,888	9,902	6,387	147	3,067
Baldwin Lake						
East Baldwin	8,090	4,486	2,373	568	132	530
Erwin	22,150	11,584	8,205	518	162	1,681
Van Dusen	9,744	5,048	2,762	1,128	85	721
West Baldwin	7,821	5,383	1,652	229	39	517
Baldwin Lake totals	47,805	26,501	14,992	2,444	419	3,449
Totals	91,196	50,389	24,894	8,831	566	6,516

ET = Evapotranspiration

Table 3-12. Comparison of Estimated Recharge with USGS-Estimated Recharge and Geoscience Estimated Maximum Perennial Yield

Watershed	Subunit	Maximum Perennial Yield or Estimated Recharge (ac-ft/yr)					
		Geoscience [a]	USGS BCM [a] (Alluvium)	USGS BCM [a] (Whole Basin)	USGS INFILv3 [a] (Alluvium)	USGS INFILv3 [a] (Whole Basin)	DBS&A DPWM (Whole Basin)
Big Bear Lake	Grout Creek	280	97	1,031	300	512	1,394
	Gray's Landing	0–140	5	145	0	2	6
	Mill Creek	100–175	60	708	452	508	980
	Village	250	91	349	240	283	1,120
	Rathbone	1,100	303	1,998	426	1,238	3,498
	Division	540 [b]	100	227	26	84	296
	North Shore	240 [c]	351	1,172	129	1,287	2,609
Big Bear Lake subtotal		2,510–2,725	1,008	6,630	1,618	3,971	9,902
Baldwin Lake	Erwin	890–900	178	5,439	1,977	3,404	8,205
	Van Dusen	800–900	0	1,541	448	1,125	2,762
	West Baldwin	500–1,000	350	389	277	527	1,652
	East Baldwin	100	58	1,798	449	545	2,373
Baldwin Lake subtotal		2,290–2,900	586	9,166	3,150	5,600	14,992
Combined Big Bear Valley Watershed total		4,800–5,625	1,594	15,796	4,768	9,571	24,894

[a] Geoscience, 2007
[b] Does not include North Shore Tributary Subarea F
[c] Does not include North Shore Tributary Subarea F

ac-ft/yr = Acre-feet per year
USGS = U.S. Geological Survey
BCM = Basin Characterization Model
DBS&A = Daniel B. Stephens & Associates, Inc.
DPWM = Distributed parameter watershed model

Table 4-1. Existing Pipe Breakdown by Diameter per Linear Foot

Material	Diameter (inches)									Total
	2	4	6	8	10	12	14	16	20	
Pipeline length (linear feet)	1,345	1,416	63,203	235,776	27,600	81,990	1,367	14,094	4,746	431,536

Table 4-2. Booster Pump Station Data

Pump Station	Number of Pumps	Total Capacity (gpm)
Abbott	3	1,200
Greenspot	3	1,507
Hicks	2	1,068
Peery	4	4,393
Rowe	3	2,417
Tanglewood	3	2,070

Table 4-3. Booster Pump Data

Booster Pump Name	To Zone	Pump Manufacturer	Type of Pump	Motor Manufacturer	Horse-power	Design Flow (gpm)	Design Head (feet)	Actual Flow (gpm)	Actual Head (feet)	Pump Efficiency (%)	Date Tested	Installation Date
Abbott Booster #1	Abbott	Simflo	TB	US	5	100	100	100	90	46.4	11/13/08	1995
Abbott Booster #2	Abbott	Simflo	TB	US	5	100	100	110	99.5	60.5	11/13/08	1995
Abbott Booster #3	Abbott	Simflo	TB	US	100	1,000	240	995	240	71.3	11/15/08	1995
Greenspot Booster #1	Greenspot	—	TB	Newman	10	—	—	212	123.9	49.1	11/11/08	1981
Greenspot Booster #2	Greenspot	—	TB	Newman	10	—	—	219	115.5	46.6	11/11/08	1981
Greenspot Booster #3	Greenspot	—	TB	—	40	—	—	560	117.7	38.7	11/11/08	1981
Hicks Booster #1	Hicks	Peerless	CF	Marathon Elec.	15	250	—	314	101.7	50.6	5/29/01 [a]	1980
Hicks Booster #2 [b]	Hicks	Peerless	CF	Marathon Elec.	30/25	—	—	754	86.6	45.1	— [a]	1980
Peery Booster #1	Peery	Byron Jackson	TB	US	30	400	190	600	128	72.3	10/9/07	1985
Peery Booster #2	Peery	Byron Jackson	TB	US	30	400	190	600	123.9	71.1	10/9/07	1985
Peery Booster #3	Peery	Byron Jackson	TB	US	75	1,000	190	960	179.3	72.3	10/9/07	1985
Peery Booster #4	Peery	Byron Jackson	TB	—	DFP	3,000	190	2,233	235.6	—	— [a]	1985
Rowe Booster #1	Rowe	Peabody Floway	TB	US	75	750	265	825	236	75.9	10/9/07	1990
Rowe Booster #2	Rowe	Peabody Floway	TB	US	75	750	265	801	246	76.9	10/9/07	1990
Rowe Booster #3	Rowe	Peabody Floway	TB	US	75	750	265	791	246.4	77.1	10/9/07	1990
Tanglewood Booster #1	Tanglewood	—	TB	Newman	20	—	—	390	84	30.5	11/11/08	1979
Tanglewood Booster #2	Tanglewood	—	TB	—	40	—	—	653	87.7	30.3	— [a]	1979
Tanglewood Booster #3	Tanglewood	—	TB	—	60	—	—	1030	90.8	35.4	— [a]	1979

[a] No efficiency test data available
[b] Motor repaired and rewound in 2009

TB = Turbine booster
— = Information not available

CF = Centrifugal
DFP = Diesel fire pump

Table 4-4. Well Drilling Data

Well Name	State Designation	Date Drilled	Depth (feet)	Perforation Depth (feet)	Pump Setting (feet)	Casing Diameter (inches)	Sanitary Seal	Ground Elevation (feet)
Vertical Wells								
Well 1	T02N/R01E-02N	—	111	—	100	12	—	6,722
Well 1B	T02N/R01E-12	2/2/58	332	100–312	170	16	No	6,722
Well 2	T02N/R01E-18L	1/11/58	248	40–218	185	16	Yes	6,760
Well 3	T02N/R01E-12N	3/15/58	324	52–115, 135–240, 304–316	143	16	No	6,708
Well 3A	T02N/R01E-12Q01	11/16/87	203	91–129, 136–166	85	8	Yes	6,712
Well 4	T02N/R01E-2N	12/5/80	112	40–100	90	12	Yes	6,715
Well 4A	T02N/R01E-12M01	11/10/87	150	42–80, 85–106	85	8	Yes	6,713
Well 5 (inactive)	T02N/R01E-11G01	7/1/79	155	75–145	143	8	Yes	6,765
Well 6	T02N/R01E-12L01	10/30/81	150	80–140	90	10	Yes	6,715
Well 7A	T02N/R01E-15B01	7/16/87	142	48–68, 118–132	105	8	Yes	6,749
Well 8	State Well #3610008-014	06/25/03	380	90-175, 195-245, 260-360	360	12	Yes	6,710
Well 9	State Well #3610008-015	07/14/03	536	200-362, 470-516	440	12	Yes	6,760
Well 10	State Well # 3610008-016	06/08/04	640	195-295, 545-620	315	26	Yes	6,930
Van Dusen Slant Wells (gravity)								
Van Dusen 1	T02N/R01E-02N	6/26/70	395	NA	NA	NA	Yes	7,000
Van Dusen 2		8/7/73	400	NA	NA	NA	Yes	
Greenspot Springs (gravity)								
Greenspot	T02N/R01E-29D	NA	NA	NA	NA	NA	Surface sealed	6,995
Fish Hatchery	T02N/R01E-3001	NA	NA	NA	NA	NA	Surface sealed	7,360

NA = Not applicable — = Information not available

Table 4-5. Vertical Well Pump Data

Well Name	Ground Elevation (feet)	Pump Manufacturer	Motor Manufacturer	Horse power	Design Flow (gpm)	Design Head (feet)	Actual Flow (gpm)	Actual Head (feet)	Pump Efficiency (%)	Date Tested
Well 1	6,722	Goulds	Franklin	20	200	—	232	72.8	21.1	10/17/2007
Well 1B	6,722	Crown 6 H-300	Franklin	20	300	300	364	85.3	31.8	11/14/2008
Well 2	6,760	Goulds	Franklin	15	100	—	87	279	40.11	10/17/2007
Well 3	6,708	Goulds	Franklin	20	350	100	305	75.5	24.4	11/14/2008
Well 3A	6,712	Goulds	Franklin	15	225	280	150	91.7	34.7	10/17/2007
Well 4	6,715	Goulds	U.S. Motors	5	165	—	205	69.4	57.8	10/17/2007
Well 4A	6,713	Grunfos 135S	Franklin	5	47	86	58	51.2	17.3	11/13/2008
Well 5 [a]	6,765	Pioneer	Franklin Elec.	2	20	—	—	—	—	— [b]
Well 6	6,715	J-line	Hitachi	5	120	—	71	61.9	29.1	10/17/2007
Well 7A [a]	6,749	Myers	Myers	75	45	—	—	—	—	— [b]
Well 8	6,710	Goulds	U.S. Motors	75	550	332	552	204	67.1	11/12/2008
Well 9	6,760	Christiansen	U.S. Motors	20	160	375	160	363	68.0	10/17/2007
Well 10	6,930	Grunfos	Franklin	10	100	260	97	286	64.1	11/13/2008

[a] Inactive
[b] No test data available

— = Information not available

Table 4-6. Reservoir Data

Tank Name	Year Constructed	Zone Served by Gravity	Zone Served by Booster Pumps	Total Capacity (million gallons)	Base Elevation (feet)	Maximum Water Surface Elevation (feet)	Depth (feet)	Diameter (feet)
Greenspot	1965	Rowe	Greenspot	1.0	6,931.0	6,967	36	70
Hicks	1970	Rowe	Hicks	1.14	6,950.0	6,974	24	90
Peery	1984	Rowe	Peery	1.6	6,941.0	6,967	26	105
Rowe	1989	None	Entire system	2.5	6,717.5	6,740.5	24	136

Table 4-7. Disinfection Injection Locations

Source	Location
Van Dusen slant wells	Van Dusen Canyon Road 1 mile north of Hale Drive
Greenspot Reservoir	Hatchery Drive and Glencove Drive
Rowe Blending Reservoir	Grenfall Lane and Paradise Way
Well 2	East Big Bear Boulevard near Zaca Road
Well 6	Maltby Boulevard and Pintail Drive
Well 8	Palomino Drive near Shay Road
Well 9	Barker Boulevard and Mount Doble Drive

Table 5-1. Tank Hydraulic Attributes

Model Tank ID	Tank Name	Diameter (feet)	Base Elevation (ft msl)	Maximum Height (feet)
GRN_RES	Greenspot	69	6,931	36
HIC_RES	Hick	90	6,950	24
PER_RES	Peery	102	6,941	26
ROW_RES	Rowe	136	6,718	26

ft msl = Feet above mean sea level

Table 5-2. Pump Hydraulic Attributes

Model Pump ID	Pump Description	Elevation (ft msl)	Design Head (feet)	Design Flow (gpm)
ABB_BP1	Abbott Booster Pump 1	6,830	80.4	128
ABB_BP2	Abbott Booster Pump 2	6,830	84.3	128
BBC_W02	Well Pump 2	6,758	279	87
BBC_W09	Well Pump 9	6,731	236	825
BBC_W3B	Well Pump 3B	6,708	91.7	150
BCC_W05	Well Pump 5	6,760	143	20
BCC_W7A	Well Pump 7A	6,746	105	45
GRN_FP	Greenspot Fire Pump	6,931	83.2	1,091
GRN_P1	Greenspot Booster Pump 1	6,931	129.3	201
GRN_P2	Greenspot Booster Pump 2	6,931	135.1	215
HIC_BP1	Hicks Booster Pump 1	6,950	101.7	314
HIC_BP2	Hicks Booster Pump 2	6,950	101.7	314
HIC_FP	Hicks Fire Pump	6,950	86.6	754
PER_FP	Peery Fire Pump	6,941	235.6	2,233
PER_P1	Peery Booster Pump 1	6,941	128	600
PER_P2	Peery Booster Pump 2	6,941	124	600
PER_P3	Peery Booster Pump 3	6,941	195.2	960
ROW_BP1	Rowe Booster Pump 1	6,718	236	825
ROW_BP2	Rowe Booster Pump 2	6,718	246	801
ROW_BP3	Rowe Booster Pump 3	6,718	264.4	791
TAN_BP1	Tanglewood Booster Pump 1	6,800	73.9	387

ft msl = Feet above mean sea level
gpm = Gallons per minute

Table 5-3. Pressure Zone Break Locations

Pressure Zone Break	Location
Greenspot and Main	Willow Lane, South of Manzanita Lane
Abbott and Main	Beaumont Lane, North of Kean Way
Abbott and Main	Shakespeare Lane
Peery and Main	Lofty View Drive, North of Panorama Drive
Abbott and Main	Rainbow Boulevard, East of Deerhorn Drive
Peery and Main	San Martin Drive, West of St. Cloud Circle
Tanglewood and Main	Meadowbrook Drive, North of Baskin Court
Abbott and Main	Marlowe Drive, South of Barrymore Road
Peery and Main	Kayah Drive, West of Mann Drive
Peery and Main	Pine Lane, West of Mann Drive
Abbott and Main	Bernhardt Lane, South of Garrick Way
Abbott and Main	Adams Drove and Barrymore Road
Restricted and Main	South of Well 4A

Table 5-4. Average Day Demand Allocation by Pressure Zone

Pressure Zone or Top User	Percentage of System Demand	Demand (gpm)
Hicks	2.8	21.3
Peery [a]	6.6	50.9
Abbott	1.4	10.4
Greenspot	6.3	48.5
Tanglewood	0.4	3.4
Whispering Pines	4.6	35.6
Big Bear H.S.	2.5	19.5
Stickleback Pond	1.2	9
Remaining System [b]	74.1	568.9
Total	100	767.5

[a] Peery Pressure Zone, excluding demand from Whispering Pines and Big Bear H.S.
[b] Main Pressure Zone, excluding demand from the Stickleback Fish Environment large user.
gpm = Gallons per minute

Table 5-5. Average Day Demand Point Demand Allocation

Large User	Junction ID	Demand (gpm)
Whispering Pines	426	35.6
Big Bear H.S.	612	19.5
Stickleback Pond	466	9

gpm = Gallons per minute

Table 5-6. Hydrant Test Scenario Demands

Test	Time of Test	Adjusted Demand (gpm)	Demand Factor Adjustment	Fire Flow (gpm)
Test 1	8:38 – 8:47 a.m.	829	0.54	1,250
Test 2	9:20 – 9:34 a.m.	911	0.59	2,250
Test 3	9:58 – 10:07 a.m.	765	0.50	2,495
Test 4	10:44 – 10:54 a.m.	765	0.50	2,870
Test 5	1:23 – 1:38 p.m.	845	0.55	980
Test 6	11:18 – 11:30 a.m.	769	0.50	1,865

gpm = Gallons per minute

Table 5-7. Summary of Total Calibration Results

Calibration Criteria	Static Number	Static %	Dynamic Number	Dynamic %	Overall Number	Overall %	Difference in Pressure Drop	Pressure Drop %
Number of sites	36	—	37	—	73	—	36	—
Within 5 psi	7	75	27	73	54	74	32	89
Between 5 and 10 psi	9	25	10	27	19	26	4	11
Greater than 10 psi	0	0	0	0	0	0	0	0

— = Not applicable
psi = Pounds per square inch

Table 5-8. Master Planning Scenarios

Demand Condition	Model Scenario Name
Existing average day demand	EX_SS_ADP
Existing maximum day demand	EX_SS_MDP
Existing peak hour demand	EX_SS_PHP
Existing maximum day demand + fire flow	EX_SS_MDP+FF
2013 average day demand	2013_SS_ADP
2013 maximum day demand	2013_SS_MDP
2013 peak hour demand	2013_SS_PHP
2013 maximum day demand + fire flow	2013_SS_MDP+FF
2030 average day demand	2030_SS_ADP
2030 maximum day demand	2030_SS_MDP
2030 peak hour demand	2030_SS_PHP
2030 maximum day demand + fire flow	2030_SS_MDP+FF

Table 5-9. Master Plan Total System Demands

Planning Year	Demand (gpm)		
	ADD	MDD	PHD
Existing	767.5	1,535	3,070
2013	810.3	1,620.6	3,241.2
2030	1,116	2,232	4,464

gpm = Gallons per minute
ADD = Average day demand
MDD = Maximum day demand
PHD = Peak hour demand

Table 5-10. Summary of Base Analysis Runs

Scenario	Model Facility Set	Model Simulation Run	System Demands
EX_SS_MDD	EX_FAC	Steady-state	Existing maximum day
EX_SS_MDP+FF	EX_FAC	Steady-state FF analysis	Existing maximum day plus fire flow
2013_SS_MDD	2013_FAC	Steady-state	2013 maximum day
2013_SS_MDP+FF	2013_FAC	Steady-state FF analysis	2013 maximum day plus fire flow
2030_SS_MDD	2030_FAC	Steady-state	2030 maximum day
2030_SS_MDP+FF	2030_FAC	Steady-state FF analysis	2030 maximum day plus fire flow

Table 5-11. Summary of Improved Analysis Runs

Scenario	Model Facility Set	Model Simulation Run	System Demands
EX_SS_MDD	IMP_FAC	Steady-state	Existing maximum day
EX_SS_MDP+FF	IMP_FAC	Steady-state FF analysis	Existing maximum day plus fire flow
2013_SS_MDD	2013_IMP_FAC	Steady-state	2013 maximum day
2013_SS_MDP+FF	2013_IMP_FAC	Steady-state FF analysis	2013 maximum day plus fire flow
2030_SS_MDD	2030_IMP_FAC	Steady-state	2030 maximum day
2030_SS_MDP+FF	2030_IMP_FAC	Steady-state FF analysis	2030 maximum day plus fire flow

Table 5-12. Commercial Zone Hydrants Flowed Simultaneously to Achieve Target Fire Flow Capacity

Target Hydrant	Adjacent Hydrant	Total Simultaneous Fire Flow (gpm)
77	78	5,100
713	28	3,200
341	540	7,800
28	713	3,200
224	225	4,700
193	J-192	3,000
166	165	5,900
538	537	1,455 [a]

[a] Still deficient, needs improvements
gpm = Gallons per minute

Table 5-13. High Elevation Hydrants

Hydrant Junction Model ID	Available Fire Flow (gpm)	Location	Resolution
196	1,495	Hilltop Lane and Sawmill Boulevard	CIP G.1 (Section 7.9)
N-36	1,461	West end of Shady Lane	Superintendent deemed available flow to be adequate
469	954	Monmouth Road	CIP G.15 (Section 7.9)
554	402	Mariposa Lane and Hemlock Lane	CIP G.12 (Section 7.9)

gpm = Gallons per minute

Table 6-1. BBCCSD Design Criteria

Element	Design Criteria
System pressure	• Goal for normal system pressure range: 40 to 120 pounds per square inch (psi). • Goal for minimum pressure during fire: 20 psi. • Goal for maximum pressure during minimum hour: 150 psi or pipeline pressure class, whichever is less. • Daily pressure fluctuations: 20 psi maximum.
Supply	• Combined production capacity of maximum day demand (MDD) with largest single source out of service. • Combined production capacity sufficient to refill emergency and fire storage in two days with all sources operating.
Storage capacity	• Operational: 30 percent of MDD. • Fire flow: based upon land use. • Emergency: 24 hours at MDD.
Booster pumping stations	• If gravity storage is available: capacity equals MDD within each pressure zone with largest single pump out of service. • If gravity storage is not available: capacity equals MDD within each pressure zone plus fire flow or peak hour demand (PHD), whichever is greater, with largest pump out of service.
Pressure reducing stations	• Capacity equals MDD or PHD within the continuous rating of valves. Maximum intermittent flow rating of valves is acceptable for fire flows. Allowance made for low flows.
Pipeline sizes	• Use standard pipe diameters of 6, 8, 12, and 16 inches for distribution.
Transmission mains	• Sized to meet MDD for pumping plant discharge lines. • Sized for MDD plus fire flow or PHD, whichever is greater. • For reservoir inlet-outlet, velocity = 4 to 6 feet per second (fps).
Distribution mains	• Sized to meet MDD plus fire flow or PHD, whichever is greater. • Maximum velocity: 10 fps, except under fire flow conditions.
Fire hydrant spacing	• Single-family residential and duplexes: 600 feet on center • All others, 300 feet on center

Table 7-1. Supply Data

Source Name	Horsepower	Design Flow		Required Flow (mgd)		
		ac-ft/yr	gpm	Existing	Near-Term	Ultimate
Well 1	20	323	200	0.29	0.29	0.29
Well 1B	20	484	300	0.43	0.43	0.43
Well 2	15	161	100	0.14	0.14	0.14
Well 3	20	565	350	0.50	NA	NA
Well 3A	15	242	150	0.22	0.22	0.22
Well 3B	TBD	1,145	416	NA	0.60	1.51
Well 4	5	266	165	0.24	0.24	0.24
Well 4A	5	76	47	0.07	0.07	0.07
Well 6	5	194	120	0.17	0.17	0.17
Well 7A	75	73	45	0.06	0.06	0.06
Well 8	75	887	550	0.79	0.79	0.79
Well 9	20	258	160	0.23	0.23	0.23
Well 10	10	161	100	0.14	0.14	0.14
Sugarloaf well	TBD	484	300	NA	0.43	0.43
Van Dusen slant wells	No pump	223	138	0.20	0.20	0.20
Greenspot Springs	No pump	278	172	0.25	0.25	0.25
			Total	3.74	4.27	5.18
			Total minus largest source	2.95	3.48	3.67

ac-ft/yr = Acre-feet per year
gpm = Gallons per minute
mgd = Million gallons per day
NA = Not applicable
TBD = To be determined

Table 7-2. Analysis of Supply

Planning Period	Maximum Day Demand (MDD) Criterion			Two-Day Criterion		
	Supply Minus Largest Source (mgd)	System MDD [a] (mgd)	Surplus or (Deficit) (mg)	All Supply Sources (mgd)	Emergency + Fire Storage [b] (mg)	2 Days > Refill (days)
Existing	2.95	2.21	0.74	3.74	2.75	0.7
Near-term	3.48	2.34	1.14	4.27	3.06	0.7
Ultimate	3.67	2.90	0.77	5.18	3.77	0.7

[a] Average day demand (ADD) times a peaking factor of 2.0 (Section 2).
[b] Calculations based on data presented in Sections 7.2, 7.7, and 7.10

mgd = Million gallons per day
mg = Million gallons

Table 7-3. Data on Fluoride-Blending Wells

Well ID	Fluoride Concentration (mg/L)		Design Flow (gpm)
	Average	Maximum	
Well 1	0.28	0.40	230
Well 1B	3.64	5.10	350
Well 3B	7.50	7.50	1,050
Well 3A	2.82	3.20	155
Well 4	0.13	0.18	205
Well 4A	1.62	2.30	55
Well 6	0.67	0.77	145
Well 8	0.31	0.32	550
Well 10	0.04	0.32	95

mg/L = Milligrams per liter
gpm = Gallons per minute

Table 7-4. Restricted Main Pipe Capacities

Pipe Segment Number	Length (feet)	Existing Diameter (inches)	Combined Flow (gpm)	Existing Velocity (fps)	Existing Head Loss (feet)	Recommendation
1	1,300	8	500	3.2	5.1	
2	2,000	12	2,770	7.9	25.0	Add parallel 10-inch pipe
3	700	8	2,150	13.7	41.2	Add parallel 12-inch pipe
4	1,200	8	1,750	11.2	48.3	Add parallel 10-inch pipe
5	8,000	12	550	1.6	5.2	
6	6,000	10	400	1.6	5.3	

gpm = Gallons per minute
fps = Feet per second

Table 7-5. Well Pump Efficiency Analysis

Well Name	Horse Power	Design Flow (gpm)	Design Head (feet)	Actual Flow (gpm)	Actual Head (feet)	Pump Efficiency (%)	Date Tested
Well 4A	5	47	86	58	51.2	17.3	11/13/2008
Well 1	20	200	—	232	72.8	21.1	10/17/2007
Well 6	5	120	—	71	61.9	29.1	10/17/2007
Well 1B	20	300	300	364	85.3	31.8	11/14/2008
Well 3A	15	225	280	150	91.7	34.7	10/17/2007
Well 2	15	100	—	87	279	40.11	10/17/2007
Well 4	5	165	—	205	69.4	57.8	10/17/2007
Well 10	10	100	260	97	286	64.1	11/13/2008
Well 7A	75	45	—	—	—	—	—[a]

[a] No test data available

gpm = Gallons per minute
— = Information not available

Table 7-6. Analysis of Storage for Existing Requirements

Zone	Tanks	Total Usable Gravity Storage [a] (mg)	Average Annual Demand [a] (ac-ft/yr)	ADD [c] (mgd)	MDD [d] (mgd)	Fire Storage [e] (gpm)	Fire Storage [e] (hrs)	Fire Storage [e] (mg)	Operational Storage [f] (mg)	Emergency Storage [g] (mg)	Total Required Storage [h] (mg)	Surplus or (Deficit) [i] (mg)	Equivalent ADD (days)
Rowe	Rowe	2.50	1,027	0.917	1.83	3,000	3	0.54	0.55	1.83	2.92	(0.42)	2.73
Hicks	Hicks	1.14	32	0.029	0.06	1,500	2	0.18	0.02	0.06	0.25	0.89	39.90
Tanglewood	0	0.00	4	0.004	0.01	1,500	2	0.18	0.00	0.01	0.19	(0.19)	0.00
Abbot	0	0.00	15	0.013	0.03	1,500	2	0.18	0.01	0.03	0.21	(0.21)	0.00
Peery	Peery	1.60	82	0.073	0.15	3,000	3	0.54	0.04	0.15	0.73	0.87	21.85
Greenspot	Greenspot	1.00	78	0.070	0.14	1,500	2	0.18	0.04	0.14	0.36	0.64	14.36
	Total	6.24	1,238	1.11	2.21			1.80	0.66	2.21	3.41	1.57	5.65

[a] Capacity of existing tanks at established high water lines
[b] Demand by pressure zone (Section 4)
[c] Average day demand, conversion of average annual demand from ac-ft/yr to mgd
[d] Maximum day demand, ADD times a peaking factor of 2.0 (Section 2)
[e] Flow times duration of fire flow as required by the Fire Marshal (Section 4)
[f] 30% of MDD (Section 6)
[g] One day of MDD (Section 6)
[h] Sum of fire, operational, and emergency storage
[i] Total usable gravity storage minus total required storage
[j] Total usable gravity storage divided by ADD; must be greater than 3 days

ADD = Average day demand
MDD = Maximum day demand

mg = Million gallons
ac-ft/yr = Acre-feet per year
mgd = Million gallons per day
gpm = Gallons per minute
hrs = Hours

Table 7-7. Existing Booster Pumps

Booster Pump Name	Data Source	Status	Flow (gpm)
Abbott Booster 1	Design flow	Online	100
Abbott Booster 2	Design flow	Online	100
Abbott Booster 3	Design flow	Online	1,000
Greenspot Booster 1	Actual flow	Online	212
Greenspot Booster 2	Actual flow	Online	219
Greenspot Booster 3	Actual flow	Online	560
Hicks Booster 1	Design flow	Online	250
Hicks Booster 2	Design flow	Online	754
Peery Booster 1	Design flow	Online	400
Peery Booster 2	Design flow	Online	400
Peery Booster 3	Design flow	Online	1,000
Peery Booster 4	Design flow	Online	3,000
Rowe Booster 1	Design flow	Online	750
Rowe Booster 2	Design flow	Online	750
Rowe Booster 3	Design flow	Online	750
Rowe Booster 4	Empty bay	Empty bay	0
Tanglewood Booster 1	Actual flow	Online	390
Tanglewood Booster 2	Actual flow	Online	653
Tanglewood Booster 3	Actual flow	Online	1,030

gpm = Gallons per minute

Table 7-8. Analysis of Booster Pumps for Existing Requirements

Zone	Booster Pump Numbers	Maximum Flow per Zone minus Largest Pump (gpm)	Required Capacity (gpm)			Surplus or (Deficit) [a] (gpm)
			Gravity Storage: MDD	No Gravity Storage, the greater of:		
				MDD+FF	PHD	
Rowe	2, 3	1,500	1,274	NA	NA	226
Hicks	2	754	NA	1,540	77	(786)
Tanglewood	1, 3	1,420	NA	1,505	10	(85)
Abbott	1, 3	1,100	NA	1,519	36	(419)
Peery	1, 2, 4	3,800	NA	3,102	198	698
Greenspot	1, 3	772	NA	1,597	189	(825)

[a] Maximum flow minus required capacity

gpm = Gallons per minute
MDD = Maximum day demand
MDD+FF = Maximum day demand plus fire flow
PHD = Peak hour demand
NA = Not applicable

Table 7-9. Booster Pump Efficiency Analysis

Booster Pump Name	To Zone	Type of Pump	Horse-power	Design Flow (gpm)	Design Head (feet)	Actual Flow (gpm)	Actual Head (feet)	Pump Efficiency (%)	Date Tested
Tanglewood Booster 2	Tanglewood	TB	40	—	—	653	87.7	30.3	—[a]
Tanglewood Booster 1	Tanglewood	TB	20	—	—	390	84	30.5	11/11/2008
Tanglewood Booster 3	Tanglewood	TB	60	—	—	1,030	90.8	35.4	—[a]
Greenspot Booster 3	Greenspot	TB	40	—	—	560	117.7	38.7	11/11/2008
Abbott Booster 1	Abbott	TB	5	100	100	100	90	46.4	11/13/2008
Greenspot Booster 2	Greenspot	TB	10	—	—	219	115.5	46.6	11/11/2008
Greenspot Booster 1	Greenspot	TB	10	—	—	212	123.9	49.1	11/11/2008
Hicks Booster 1	Hicks	CF	15	250	—	314	101.7	50.6	5/29/2001
Abbott Booster 2	Abbott	TB	5	100	100	110	99.5	60.5	11/13/2008
Peery Booster 4	Peery	TB	DFP	3,000	190	2,233	235.6	Unknown	—[a]

[a] Data taken from 1991 Water Master Plan

TB = Turbine booster
— = Information not available
CF = Centrifugal
DFP = Diesel fire pump

Table 7-10. 2-inch Steel Pipe Replacement Data

Alignment Location		Length (feet)	Diameter (inches)	Additional Information
1	Nana Ave., between Meadow Ln. and North Shore Dr.	870	8	Install hydrant at mid-block
2	Peter Ave., between Meadow Ln. and North Shore Dr.	920	8	Install hydrant at mid-block
3	Myrtle Ave., between Meadow Ln. and North Shore Dr.	1,630	8	Install hydrant at mid-block
4	Fairway Blvd., between Greenway Dr. and Paradise Way	1,640	8	Install hydrant at Gold Mountain Dr.
5	Barranca Blvd., between Country Club Blvd. and Shay Rd.	1,700	8	Install hydrant at 1208 Barranca Blvd.
6	Paradise Way, between Tiger Lily Ave. and North Shore Dr.	860	8	Install hydrant at back fence line
7	Pan Springs Ln., between Tiger Lily Ave. and Cinderella Dr.	540	8	Install hydrant at back fence line
8	North Shore Drive, between Greenway Dr. and Sierra Ave.	370	8	
9	North Shore Drive, between Michael Ave. and Wendy Ave.	1,650	8	
10	North Shore Dr., between Sequoia Dr. and Gold Mountain Dr.	480	8	
11	W. Cinderella Dr., between Peter Ave. and Michael Ave.	230	8	
12	Mullins Dr., between Nana Ave. and Michael Ave.	730	8	
13	Curly Ave., between Nana Ave. and Michael Ave.	730	8	
14	Myrtle Ave., between Pioneer Ln. and North Shore Dr.	840	8	
15	Sierra Ave., between Pioneer Ln. and North Shore Dr.	450	8	
16	Nana Ave., between Tiger Lily Dr. and North Shore Dr.	540	8	
17	Tinkerbell Ave., between Tiger Lily Dr. and North Shore Dr.	380	8	
18	Tinkerbell Ave., between Meadow Ln. and North Shore Dr.	1,000	8	
19	Michael Ave., between Meadow Ln. and North Shore Dr.	1,070	8	
20	Coy Ln., between Anita Ave. and Greenway Dr.	560	8	
21	Cinderella Dr., between Hugo Ln. and Paradise Way	620	8	
22	Aeroplane Blvd., between Division Dr. and Pineview Dr.	2,050	8	
23	Fairway Blvd., between Division Dr. and Keiner Dr.	2,810	8	
24	Gildhart Dr., between Aeroplane Blvd. and Big Bear Blvd.	230	8	
25	Winding Ln.	1,400	8	
26	Mojave Blvd., between Mt. Doble Dr. and Paradise Way	1,180	8	
27	Mountain View Blvd., west of Mt. Doble Dr.	140	8	
28	Angeles Blvd., between Paradise Way and west of Mt. Doble Dr.	1,260	8	
29	Gold Mountain Dr., north of Mountain View Blvd.	200	8	
30	Elysian Blvd., east of Shore Dr.	460	8	
31	Mt. Doble Dr., between Cinderella Dr. and North Shore Dr.	350	8	
32	Gold Mountain Dr., between Cinderella Dr. and North Shore Dr.	430	8	
33	Bowles Dr., between Aeroplane Blvd. and Valley Blvd.	880	8	

Table 7-11. Cohila Pressure Zone Maximum Day Demand

Area	MDD (gpd)
Tanglewood	7,838
Abbot	29,423
Saw Mill Creek Estates	97,142
Area 12 (Hillen Dale)	95,142
Portions of Rowe Pressure Zone [a]	265,126 [b]
Total	494,671

[a] Approximately 110 acres of single-family residential

[b] See V_{Rowe} calculation below.

$$V_{Rowe} = PF \times A \times WDF = (2.0)(110\ acres)\left(\frac{1.35\ AFY}{acre}\right)\left(\frac{43{,}560\ ft^2}{acre}\right)\left(\frac{7.48\ gal.}{ft^3}\right)\left(\frac{year}{365\ days}\right) = 265{,}126\ gallons$$

where V_{Rowe} = the required volume of the Rowe Reservoir
PF = the peaking factor for maximum day demand (see Section 6.1)
A = area in acres
WDF = single family residential water duty factor (see Section 2.6)

Table 7-12. Analysis of Storage for Near-Term Requirements

Zone	Tanks	Total Usable Gravity Storage (mg)	Average Annual Demand (ac-ft/yr)	ADD (mgd)	MDD (mgd)	Fire Storage (gpm)	Fire Storage (hrs)	Fire Storage (mg)	Operational Storage [a] (mg)	Emergency Storage [b] (mg)	Total Required Storage [c] (mg)	Surplus or (Deficit) [d] (mg)	Equivalent. ADD [e] (days)
Rowe	Rowe	2.50	969	0.865	1.73	3,000	3	0.54	0.52	1.73	2.79	(0.29)	2.89
Hicks	Hicks	1.14	32	0.029	0.06	1,500	2	0.18	0.02	0.06	0.25	0.89	39.90
Peery	Peery	1.60	82	0.073	0.15	3,000	3	0.54	0.04	0.15	0.73	0.87	21.85
Greenspot	Greenspot	1.00	78	0.070	0.14	1,500	2	0.18	0.04	0.14	0.36	0.64	14.36
Cohila	Cohila 1	0.60	148	0.132	0.26	1,500	2	0.18	0.08	0.26	0.52	0.08	4.54
	Total	6.84	1,309.10	1.17	2.34			1.62	0.70	2.34	4.66	2.18	5.85

[a] 30% of the MDD.
[b] 100% of the MDD.
[c] Fire storage plus operational storage plus emergency storage
[d] Total usable gravity storage minus total required storage
[e] Total usable gravity storage divided by ADD ; must be greater than 3 days

ADD = Average day demand
MDD = Maximum day demand

mg = Million gallons
ac-ft/yr = Acre-feet per year
mgd = Million gallons per day
gpm = gallons per minute
hrs = Hours

Table 7-13. Analysis of Booster Pumps for Near-Term Requirements

Zone	Booster Pump Numbers	Maximum Flow per Zone minus Largest Pump (gpm)	Required Capacity (gpm)			Surplus or (Deficit) [a] (gpm)
			Gravity Storage:	Non-Gravity Storage, the greater of:		
				MDD+ FF	PHD	
Rowe	2, 3	1,500	1,202	NA	NA	298
Hicks	2, 3	1,550	NA	1,540	77	10
Peery	1, 2, 4	3,800	NA	3,102	198	698
Greenspot	1, 3	1,712	NA	1,597	189	115
Cohila	1	200	184	NA	NA	16

[a] Maximum flow minus required capacity

gpm = Gallons per minute
MDD = Maximum day demand
MDD+FF = Maximum day demand plus fire flow
PHD = Peak hour demand
NA = Not applicable

Table 7-14. Analysis of Storage for Ultimate Requirements

Zone	Tanks	Total Usable Gravity Storage (mg)	Average Annual Demand (ac-ft/yr)	ADD (mgd)	MDD (mgd)	Fire Storage (gpm)	Fire Storage (hrs)	Fire Storage (mg)	Operational Storage [a] (mg)	Emergency Storage [b] (mg)	Total Required Storage [c] (mg)	Surplus or (Deficit) [d] (mg)	Equivalent. ADD [e] must be > 3 days (days)
Rowe	Rowe	2.50	1,015	0.906	1.81	3,000	3	0.54	0.54	1.81	2.90	(0.40)	2.76
Hicks	Hicks	1.14	33	0.029	0.06	1,500	2	0.18	0.02	0.06	0.26	0.88	38.69
Peery	Peery	1.60	90	0.080	0.16	3,000	3	0.54	0.05	0.16	0.75	0.85	19.91
Greenspot	Greenspot	1.00	85	0.076	0.15	1,500	2	0.18	0.05	0.15	0.38	0.62	13.18
Cohila	Cohila 1, 2	1.00	277	0.247	0.49	1,500	2	0.18	0.15	0.49	0.82	0.18	4.04
Section 16	0	0.00	120	0.107	0.22	1,500	2	0.18	0.07	0.22	0.47	(0.47)	0.00
	Total	7.24	1,620	1.45	2.90			1.80	0.87	2.90	5.57	1.67	5.01

[a] 30% of the MDD.
[b] 100% of the MDD.
[c] Fire storage plus operational storage plus emergency storage
[d] Total usable gravity storage minus total required storage
[e] Total usable gravity storage divided by ADD

ADD = Average day demand
MDD = Maximum day demand

mg = Million gallons
ac-ft/yr = Acre-feet per year
mgd = Million gallons per day
gpm = gallons per minute
hrs = Hours

Table 7-15. Analysis of Booster Pumps for Ultimate Requirements

Zone	Booster Pump Numbers	Maximum Flow per Zone minus Largest Pump (gpm)	Required Capacity (gpm)			Surplus or (Deficit) [a] (gpm)
			Gravity Storage MDD	Non-Gravity Storage, the greater of:		
				MDD+ FF	PHD	
Rowe	2, 3	1,500	1,259	NA	NA	241
Hicks	2, 3	1,550	NA	1,541	80	9
Peery	1, 2, 4	3,800	NA	1,612	218	2,188
Greenspot	1, 3	1,712	NA	1,605	206	107
Cohila	1, 2	400	344	NA	NA	56
Section 16	None	0	156	NA	NA	(156)

[a] Maximum flow minus required capacity

gpm = Gallons per minute
MDD = Maximum day demand
MDD+FF = Maximum day demand plus fire flow
PHD = Peak hour demand
NA = Not applicable

Table 7-16. Existing Capital Improvement Projects

Description	Justification	Capital Cost ($1,000) by Fiscal Year (July to June)																					
		2008 Est.	09-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22	22-23	23-24	24-25	25-26	26-27	27-28	28-29	29-30
A. Supply																							
1. Restricted Main upgrade	Well pump efficiency	698	73	75	78	82	85	88	92	95	99	103											
2. Fluoride treatment facility	Health standards	2,000					243	253	263	274	285	296	308	320	333	346							
3. Well pump rehabilitation	As needed	450	22	23	24	25	26	27	28	29	30	32	33	34	36	37	39	40	42	43	45	47	49
4. Well contamination study	Health standards	20	21																				
5. Equip Well 3B	Increase capacity	300			169	175																	
B. Pump Stations																							
1. Hicks Booster 2A (new 1,300-gpm fire pump)	Improve fire flows	30	31																				
2. Hicks Booster 3A (new 250-gpm booster pump)	Increase redundancy	22		24																			
3. Greenspot Boosters (2 boosters and 1 fire pump)	Improve fire flows	74	26	27	28																		
4. Booster pump rehabilitation	As needed	220	11	11	12	12	13	13	14	14	15	16	16	17	17	18	19	20	20	21	22	23	24
C. Distribution Pipeline Improvements																							
1. State Lane	Improve fire flows	536				157	163	169	176														
2. Michael Avenue	Improve fire flows	1,224								335	348	362	377	392									
3. Cinderella Drive	Improve fire flows	561													467	485							
4. Greenway Drive	Hydrant spacing	458															824						
5. Fern Lane	Improve fire flows	162																303					
6. Division Drive	Improve fire flows	1,133																708	736	765			
7. Maltby Boulevard	Improve fire flows	477																			1,006		
8. Barrett Way	Improve fire flows	660																				724	752
9. Mountain View Boulevard	Improve fire flows	377																				413	430
10. 2-inch steel pipe replacement	As needed	4,602	228	237	247	256	267	277	288	300	312	324	337	351	365	380	395	410	427	444	462	480	499
D. Meter Replacement	Reduce water loss	2,280				267	277	288	300	312	325	337	351	365	380								
	Total	16,284	412	397	558	974	1,074	1,115	1,161	1,359	1,414	1,470	1,422	1,479	1,598	1,266	1,277	1,481	1,225	1,273	1,535	1,687	1,754

Notes: 1. Capital cost = 1.35 x construction cost
2. Costs escalated at 4% per year

Table 7-17. Near-Term and Ultimate Capital Improvement Projects

Description	Justification	Capital Cost ($1,000) by Fiscal Year (July to June)																					
		2008 Est.	09-10	10-11	11-12	12-13	13-14	14-15	15-16	16-17	17-18	18-19	19-20	20-21	21-22	22-23	23-24	24-25	25-26	26-27	27-28	28-29	29-30
E. Reservoirs																							
1. Cohila Reservoir 1 (0.6 mg)	New pressure zone	750					304	316	329														
2. Cohila Reservoir 2 (0.4 mg)	New pressure zone	500								342	356												
3. Section 16 Reservoirs (2 @ 0.25 million gallons)	New pressure zone	625																			439	456	475
F. Pump Stations																							
1. Cohila Boosters, Phase 1 (two 200-gpm boosters)	New pressure zone	44					18	19	19														
2. Cohila Boosters, Phase 2 (one 200-gpm booster)	New pressure zone	22								15	16												
3. Section 16 Boosters (three 75-gpm boosters)	New pressure zone	66																			46	48	50
G. Distribution Pipeline Improvements																							
1. Cohila pipelines, Phase 1	New pressure zone	2,439					989	1,028	1,070														
2. Cohila pipelines, Phase 2	New pressure zone	520								356	370												
3. Wendy Drive	Complete main line system	188										278											
4. Big Bear Boulevard		275										407											
5. Meadow Drive	Improve fire flows	316											486										
6. Hale Drive	Improve continuity	135											208										
7. Woodland Drive	Improve fire flows	104												166									
8. Arbor Lane	Improve fire flows	455												729									
9. Fariway Boulevard	Improve fire flows	67													111								
10. Greenspot Road	Improve fire flows	42													70								
11. Downey Drive	Improve fire flows	251													418								
12. Mariposa Lane	Improve fire flows	505														874							
13. Malabar Way	Improve fire flows	1,086															652	678	705				
14. Mt . Doble Drive	Improve fire flows	236																		477			
15. Monmouth Road	Improve fire flows	345																		699			
16. E Lane	Improve fire flows	261																			550		
17. Lakewood Drive	Improve fire flows	397																				435	452
18. Commercial zone hydrants	Hydrant spacing	54					16	17	18	18													
19. Residential zone hydrants	Hydrant spacing	27					16	17															
	Total	9,681					1,327	1,380	1,436	731	742	685	694	895	599	874	652	678	705	1,176	1,035	939	977

Notes: 1. Capital cost = 1.35 x construction cost
2. Costs escalated at 4% per year

Table 8-1. Impact Fees for New Connections

Projected new connections	2,791 connections
Percentage of total connections	32%
32% of recommended improvements	$8 million
32% of existing main line system	$14 million
Total benefit to new development	$22 million
Connection fee	$7,976 per connection

Table 8-2. Bimonthly Base Rates for Residential Meters

Meter Size (inches)	Rate Code	Bimonthly Base Rate ($)
⅝ and ¾	M00	40.04
1	M01	80.05
1½	M02	160.13
2	M03	256.19
3	M04	480.36
4	M05	800.58
6	M06	1,601.15

Table 8-3. Bimonthly Quantity Rates for Residential Meters

Meter Size (inches)	Rate Code	Tier 1		Tier 2		Tier 3	
		Cost per Unit ($)	Units	Cost per Unit ($)	Units	Cost per Unit ($)	Units
⅝ and ¾	M00	1.48	0-24	1.86	25-40	2.21	41-70
1	M01	1.48	0-48	1.86	49-80	2.21	81-140
1½	M02	1.48	0-96	1.86	97-160	2.21	161-280
2	M03	1.48	0-154	1.86	155-256	2.21	257-560
3	M04	1.48	0-288	1.86	289-480	2.21	481-1,120
4	M05	1.48	0-480	1.86	481-800	2.21	801-2,240
6	M06	1.48	0-960	1.86	961-1,600	2.21	1,601-4,480

Note: A unit is 100 cubic feet of water.

Appendix A

Methodology and Calculations for Maximum Day Demand and Minimum Day Demand

APPENDIX A – Methodology and Calculations for Maximum Day Demand and Minimum Day Demand

Assumptions

Use of production data to estimate demand makes the following assumptions:

- There is a 1-to-1 relationship between production and demand (i.e. no water is exported).
- Meter readings are taken at the same time each day.
- Meters are accurate.
- The meters for some production facilities were not read every day; missing readings were interpolated.
- Variation in storage is negligible.

Methodology and Calculations

The Maximum Day Demand (MDD) peaking factor is the ratio of the MDD to the Average Day Demand (ADD). The Minimum Day Demand (Min Day) peaking factor is the ratio of the Min Day to the ADD.

The ADD was computed as the total volume of water supplied during the calendar year 2007 divided by 365 days:

$$ADD = \frac{1{,}238\ AF}{365\ days}\left(\frac{43{,}560\ ft^2}{acre}\right)\left(\frac{7.48\ gallons}{ft^3}\right)\left(\frac{day}{24 \times 60\ minutes}\right) = 767.5\ gpm$$

The MDD and Min Day were identified as the highest and lowest production volume for a single day respectively. Daily readings are taken by hand at the various well, slant wells and spring sites. The meter readings for July (highest monthly production) and February (lowest monthly production) were tabulated, converted to daily volumes and summed on a daily basis. The highest daily volume produced in July and the lowest daily volume produced in February were determined to represent the MDD and Min Day respectively. Those volumes and peaking factor ratios are as follows:

$$MDD = \frac{6.63\ AF}{day}\left(\frac{43{,}560\ ft^2}{acre}\right)\left(\frac{7.48\ gallons}{ft^3}\right)\left(\frac{day}{24 \times 60\ minutes}\right) = 1{,}500\ gpm$$

$$Peaking\ Factor = \frac{MDD}{ADD} = \frac{1{,}500\ gpm}{767.5\ gpm} = 1.95 \cong 2.0$$

$$MDD = \frac{1.47\ AF}{day}\left(\frac{43{,}560\ ft^2}{acre}\right)\left(\frac{7.48\ gallons}{ft^3}\right)\left(\frac{day}{24 \times 60\ minutes}\right) = 333\ gpm$$

$$Peaking\ Factor = \frac{MDD}{ADD} = \frac{333\ gpm}{767.5\ gpm} = 0.433 \cong 0.4$$

Following are tables containing the tabulated production meter readings and converted daily volumes for MDD and Min Day.

Table A-1 - Tabulated Production Meter Readings for July 2007

	Green Spot	Van Dusen	Well 1	Well 1B	Well 2	Well 3	Well 3A	Well 4	Well 4A	Well 6	Well 8	Well 9	Well 10
Previous Reading	457708	54759	608404	241440	395649	94074	422452	105496	29556	531741	86377	47078	15566
7/1/2007	457965	55038	608697	241440	395772	94074	424242	105727	30208	531741	86377	47314	15696
7/2/2007	458276	55390	609111	241440	395903	94074	426081	105974	30900	531916	86377	47553	15822
7/3/2007	458484	55596	609288	241440	396033	94074	427920	106220	31592	531961	86651	47792	15947
7/4/2007	458769	55847	609529	241440	396164	94074	429925	106409	32127	532035	86784	48050	16048
7/5/2007	459002	56132	609776	241440	396275	94074	430999	106624	32731	532083	87028	48200	16166
7/6/2007	459227	56414	610010	241447	396383	94074	432380	106813	33310	532139	87215	48444	16272
7/7/2007	459528	56708	610275	241447	396534	94078	433944	107019	34442	532201	87402	48673	16376
7/8/2007	459711	57038	610530	241447	396661	94078	435789	107261	34837	532275	87402	49034	16530
7/9/2007	460044	57260	610885	241447	396787	94078	437634	107503	35231	532349	87402	49395	16632
7/10/2007	460309	57496	611096	241447	396900	94078	438801	107659	35674	532396	87693	49507	16720
7/11/2007	460550	57775	611273	241447	397012	94078	439968	107815	36117	532442	87720	49618	16800
7/12/2007	460751	58058	611518	241449	397143	94078	441408	108012	36679	532501	87720	49832	16900
7/13/2007	461024	58272	611750	241449	397273	94078	443023	108235	37313	532540	87764	50068	16997
7/14/2007	461240	58525	612012	241449	397365	94078	444638	108445	37910	532613	87827	50230	17108
7/15/2007	461487	58803	612275	241449	397485	94078	446252	108655	38508	532686	87890	50488	17218
7/16/2007	461770	59122	612537	241449	397645	94078	447867	108865	39105	532759	87953	50745	17329
7/17/2007	461983	59333	612799	241449	397733	94078	448739	108990	39457	532800	88145	50954	17417
7/18/2007	462292	59592	612979	241449	397853	94078	450199	109190	40029	532853	88755	51117	17524
7/19/2007	462516	59876	613205	241460	397982	94078	451623	109363	40519	532900	88896	51840	17607
7/20/2007	462760	60227	613453	241460	398111	94078	452982	109561	41081	533045	89036	51945	17722
7/21/2007	463043	60620	613639	241460	398253	94078	454617	109779	41701	533045	89058	52050	17836
7/22/2007	463224	60876	613826	241460	398395	94078	456252	109998	42320	533120	89079	52123	17951
7/23/2007	502502	60946	614012	241460	398488	94078	457887	110216	42940	533160	89101	52316	18065
7/24/2007	502731	61192	614199	241460	398601	94078	458296	110326	43250	533175	89122	52508	18123
7/25/2007	502995	61456	614385	241464	398724	94078	458705	110326	43250	533190	89144	52701	18123
7/26/2007	503236	61768	614658	241464	398852	94084	460352	110547	43879	533279	89144	52968	18236
7/27/2007	503436	61995	614931	241464	398940	94090	461998	110767	44508	533325	89144	53104	18349
7/28/2007	503681	62290	615115	241464	399061	94090	463628	110986	45133	533384	89144	53397	18473
7/29/2007	503970	62545	615300	241464	399209	94090	465257	111205	45759	533451	89144	53604	18598
7/30/2007	504164	62813	615484	241464	399301	94090	466887	111424	46384	533527	89144	53824	18722
7/31/2007	504430	63079	615846	241464	399432	94090	469346	111727	47245	533619	89144	54083	18842

Table A-2 – Production Meter Readings Differences for July 2007

	Green Spot	Van Dusen	Well 1	Well 1B	Well 2	Well 3	Well 3A	Well 4	Well 4A	Well 6	Well 8	Well 9	Well 10
7/1/2007	257	279	293	0	123	0	1790	231	652	0	0	236	130
7/2/2007	311	352	414	0	131	0	1839	247	692	175	0	239	126
7/3/2007	208	206	177	0	131	0	1839	247	692	45	274	239	126
7/4/2007	285	251	241	0	131	0	2005	189	535	74	133	258	101
7/5/2007	233	285	247	0	111	0	1074	215	604	48	244	150	118
7/6/2007	225	282	234	7	108	0	1381	189	579	56	187	244	106
7/7/2007	301	294	265	0	151	4	1564	206	1132	62	187	229	104
7/8/2007	183	330	255	0	127	0	1845	242	395	74	0	361	154
7/9/2007	333	222	355	0	127	0	1845	242	395	74	0	361	102
7/10/2007	265	236	211	0	113	0	1167	156	443	47	291	112	88
7/11/2007	241	279	177	0	113	0	1167	156	443	47	27	112	80
7/12/2007	201	283	245	2	131	0	1440	197	562	59	0	214	100
7/13/2007	273	214	232	0	131	0	1615	223	634	39	44	236	97
7/14/2007	216	253	262	0	92	0	1615	210	597	73	63	162	111
7/15/2007	247	278	262	0	120	0	1615	210	597	73	63	258	111
7/16/2007	283	319	262	0	160	0	1615	210	597	73	63	258	111
7/17/2007	213	211	262	0	88	0	872	125	352	41	192	209	88
7/18/2007	309	259	180	0	120	0	1460	200	572	53	610	163	107
7/19/2007	224	284	226	11	129	0	1424	173	490	47	141	723	83
7/20/2007	244	351	248	0	129	0	1359	198	562	145	141	105	115
7/21/2007	283	393	186	0	142	0	1635	218	620	0	22	105	115
7/22/2007	181	256	186	0	142	0	1635	218	620	75	22	73	115
7/23/2007	205	70	186	0	93	0	1635	218	620	40	22	193	115
7/24/2007	229	246	186	0	113	0	409	110	310	15	22	193	58
7/25/2007	264	264	186	4	123	0	409	0	0	15	22	193	0
7/26/2007	241	312	273	0	128	6	1647	221	629	89	0	267	113
7/27/2007	200	227	273	0	88	6	1647	221	629	46	0	136	113
7/28/2007	245	295	184	0	121	0	1630	219	625	59	0	293	124
7/29/2007	289	255	184	0	148	0	1630	219	625	67	0	207	124
7/30/2007	194	268	184	0	92	0	1630	219	625	76	0	220	124
7/31/2007	266	266	362	0	131	0	2459	303	861	92	0	259	120

Table A-3 – Daily Production Volumes for July 2007 (acre-feet)

	Green Spot	Van Dusen	Well 1	Well 1B	Well 2	Well 3	Well 3A	Well 4	Well 4A	Well 6	Well 8	Well 9	Well 10	Daily Totals
7/1/2007	0.789	0.640	0.899	0.000	0.377	0.000	0.549	0.709	0.200	0.000	0.000	0.724	0.399	5.290
7/2/2007	0.954	0.808	1.271	0.000	0.401	0.000	0.564	0.757	0.212	0.537	0.000	0.734	0.385	6.620
7/3/2007	0.638	0.473	0.543	0.000	0.401	0.000	0.564	0.757	0.212	0.138	0.841	0.734	0.385	5.690
7/4/2007	0.875	0.576	0.740	0.000	0.402	0.000	0.615	0.580	0.164	0.227	0.408	0.792	0.310	5.690
7/5/2007	0.715	0.654	0.758	0.000	0.341	0.000	0.330	0.660	0.185	0.147	0.749	0.460	0.362	5.360
7/6/2007	0.691	0.647	0.718	0.021	0.331	0.000	0.424	0.580	0.178	0.172	0.574	0.749	0.325	5.410
7/7/2007	0.924	0.675	0.813	0.000	0.463	0.012	0.480	0.632	0.347	0.190	0.574	0.703	0.319	6.130
7/8/2007	0.562	0.758	0.783	0.000	0.388	0.000	0.566	0.743	0.121	0.227	0.000	1.108	0.473	5.730
7/9/2007	1.022	0.510	1.090	0.000	0.388	0.000	0.566	0.743	0.121	0.227	0.000	1.108	0.313	6.090
7/10/2007	0.813	0.542	0.648	0.000	0.345	0.000	0.358	0.479	0.136	0.143	0.893	0.342	0.270	4.970
7/11/2007	0.740	0.640	0.543	0.000	0.345	0.000	0.358	0.479	0.136	0.143	0.083	0.342	0.246	4.050
7/12/2007	0.617	0.650	0.752	0.006	0.401	0.000	0.442	0.605	0.172	0.181	0.000	0.657	0.307	4.790
7/13/2007	0.838	0.491	0.712	0.000	0.401	0.000	0.496	0.684	0.195	0.120	0.135	0.724	0.298	5.090
7/14/2007	0.663	0.581	0.805	0.000	0.282	0.000	0.496	0.645	0.183	0.224	0.193	0.497	0.340	4.910
7/15/2007	0.758	0.638	0.805	0.000	0.368	0.000	0.496	0.645	0.183	0.224	0.193	0.790	0.340	5.440
7/16/2007	0.869	0.732	0.805	0.000	0.491	0.000	0.496	0.645	0.183	0.224	0.193	0.790	0.340	5.770
7/17/2007	0.654	0.484	0.805	0.000	0.270	0.000	0.268	0.384	0.108	0.126	0.589	0.641	0.270	4.600
7/18/2007	0.948	0.595	0.552	0.000	0.368	0.000	0.448	0.614	0.176	0.163	1.872	0.500	0.328	6.560
7/19/2007	0.687	0.652	0.694	0.034	0.396	0.000	0.437	0.531	0.150	0.144	0.431	2.219	0.255	***6.630***
7/20/2007	0.749	0.806	0.761	0.000	0.396	0.000	0.417	0.608	0.172	0.445	0.431	0.322	0.351	5.460
7/21/2007	0.869	0.902	0.572	0.000	0.436	0.000	0.502	0.670	0.190	0.000	0.066	0.322	0.351	4.880
7/22/2007	0.556	0.588	0.572	0.000	0.436	0.000	0.502	0.670	0.190	0.230	0.066	0.224	0.351	4.390
7/23/2007	0.629	0.161	0.572	0.000	0.285	0.000	0.502	0.670	0.190	0.123	0.066	0.591	0.351	4.140
7/24/2007	0.703	0.565	0.572	0.000	0.347	0.000	0.126	0.338	0.095	0.046	0.066	0.591	0.178	3.630
7/25/2007	0.811	0.606	0.572	0.012	0.377	0.000	0.126	0.000	0.000	0.046	0.066	0.591	0.000	3.210
7/26/2007	0.738	0.716	0.838	0.000	0.393	0.018	0.505	0.677	0.193	0.273	0.000	0.819	0.347	5.520
7/27/2007	0.613	0.521	0.838	0.000	0.270	0.018	0.505	0.677	0.193	0.141	0.000	0.417	0.347	4.540
7/28/2007	0.753	0.677	0.566	0.000	0.371	0.000	0.500	0.672	0.192	0.181	0.000	0.899	0.382	5.190
7/29/2007	0.887	0.585	0.566	0.000	0.454	0.000	0.500	0.672	0.192	0.206	0.000	0.635	0.382	5.080
7/30/2007	0.595	0.615	0.566	0.000	0.282	0.000	0.500	0.672	0.192	0.233	0.000	0.675	0.382	4.710
7/31/2007	0.818	0.611	1.111	0.000	0.402	0.000	0.755	0.930	0.264	0.282	0.000	0.795	0.368	6.340

Table A-4 – Tabulated Production Meter Readings for February 2007

	Green Spot	Van Dusen	Well 1	Well 1B	Well 2	Well 3	Well 3A	Well 4	Well 4A	Well 6	Well 8	Well 9	Well 10
Previous Reading	416011	10048	588501	241075	388609	93861	307622	89818	994094	526652	82080	29016	7967
2/1/2007	416334	10438	588520	241078	388609	93861	307845	89818	994141	526661	82080	29331	7971
2/2/2007	416661	10711	588520	241078	388609	93861	307845	89818	994141	526661	82080	29586	7971
2/3/2007	417028	11015	588520	241078	388609	93861	307845	89819	994141	526661	82080	29586	7971
2/4/2007	417356	11377	588601	241078	388609	93861	308660	89879	994308.5	526678.5	82080	29586	8019
2/5/2007	417642	11644	588682	241078	388609	93861	308660	89939	994476	526696	82080	29619	8019
2/6/2007	417963	11918	588787	241083	388609	93861	309376	90021	994591.5	526727	82080	29650	8062
2/7/2007	418313	12258	588793	241088	388609	93871	309437	90024	994707	526729	82080	29650	8068
2/8/2007	418619	12663	588843	241101	388609	93878	309437	90063	994818	526741	82080	29650	8123
2/9/2007	418944	12887	588843	241101	388609	93878	309691	90063	994818	526741	82080	29697	8123
2/10/2007	419275	13242	588894.5	241101	388609	93878	310283	90142	994927.5	526766	82080	29753	8140.5
2/11/2007	419605	13597	588946	241101	388609	93878	310283	90142	995037	526766	82080	29753	8158
2/12/2007	419913	13864	589106	241101	388609	93878	311207	90264	995377	526806	82080	29844	8213
2/13/2007	420226	14205	589106	241101	388609	93878	311207	90264	995377	526806	82080	29844	8213
2/14/2007	420523	14468	589220	241102	388609	93878	311930	90267	995632	526834	82080	29908	8257
2/15/2007	420838	14811	589247	241103	388609	93878	312020	90373	995675	526841	82080	29908	8260
2/16/2007	421143	15091	589305	241103	388609	93878	312353	90416	995798	526856	82080	29958	8281
2/17/2007	421469	15263	589316	241103	388609	93878	312356	90425	995826	526856	82080	29958	8281
2/18/2007	421795	15780	589491	241103	388609	93878	313433	90512	995865	526902	82080	30066	8291
2/19/2007	422105	16140	589592	241103	388609	93878	314019	90636	996410	526927	82080	30123	8382
2/20/2007	422353	16355	589616	241103	388609	93878	314258	90654	996459	526936	82080	30160	8382
2/21/2007	422669	16650	589733	241115	388609	93890	314863	90744	996706	526963	82080	30194	8431
2/22/2007	422966	16986	589736	241122	388609	93894	314874	90750	996727	526964	82080	30194	8435
2/23/2007	423317	17296	589739	241122	388609	93894	314874	90861	997032	527001	82080	30250	8478
2/24/2007	423575	17609	589739	241122	388609	93894	314874	90861	997032	527001	82080	30250	8484
2/25/2007	423884	17909	589739	241122	388609	93894	314874	91067	997602	527076	82080	30335	8580
2/26/2007	424205	18292	589739	241122	388609	93894	314874	91111	997726	527077	82080	30390	8603
2/27/2007	424491	18566	589810	241122	388609	93894	314941	91163	997869	527098	82080	30482	8646
2/28/2007	424789	18840	589881	241122	388609	93894	315008	91215	998012	527119	82080	30574	8654

Table A-5 – Production Meter Readings Differences for February 2007

	Green Spot	Van Dusen	Well 1	Well 1B	Well 2	Well 3	Well 3A	Well 4	Well 4A	Well 6	Well 8	Well 9	Well 10
7/1/2007	323	390	19	3	0	0	223	0	47	9	0	315	4
7/2/2007	327	273	0	0	0	0	0	1	0	0	0	255	0
7/3/2007	367	304	0	0	0	0	0	60	0	0	0	0	0
7/4/2007	328	362	81	0	0	0	815	60	168	18	0	0	48
7/5/2007	286	267	81	0	0	0	0	82	168	18	0	33	0
7/6/2007	321	274	105	5	0	0	716	82	116	31	0	31	43
7/7/2007	350	340	6	5	0	10	61	3	116	2	0	0	6
7/8/2007	306	405	50	13	0	7	0	39	111	12	0	0	55
7/9/2007	325	224	0	0	0	0	254	0	0	0	0	47	0
7/10/2007	331	355	52	0	0	0	592	79	110	25	0	56	18
7/11/2007	330	355	52	0	0	0	0	0	110	0	0	0	18
7/12/2007	308	267	160	0	0	0	924	122	340	40	0	91	55
7/13/2007	313	341	0	0	0	0	0	0	0	0	0	0	0
7/14/2007	297	263	114	1	0	0	723	3	255	28	0	64	44
7/15/2007	315	343	27	1	0	0	90	106	43	7	0	0	3
7/16/2007	305	280	58	0	0	0	333	43	123	15	0	50	21
7/17/2007	326	172	11	0	0	0	3	9	28	0	0	0	0
7/18/2007	326	517	175	0	0	0	1077	87	39	46	0	108	10
7/19/2007	310	360	101	0	0	0	586	124	545	25	0	57	91
7/20/2007	248	215	24	0	0	0	239	18	49	9	0	37	0
7/21/2007	316	295	117	12	0	12	605	90	247	27	0	34	49
7/22/2007	297	336	3	7	0	4	11	6	21	1	0	0	4
7/23/2007	278	310	3	0	0	0	0	111	305	37	0	56	43
7/24/2007	258	313	0	0	0	0	0	0	0	0	0	0	6
7/25/2007	309	300	0	0	0	0	0	206	570	75	0	85	96
7/26/2007	321	383	0	0	0	0	0	44	124	1	0	55	23
7/27/2007	286	274	71	0	0	0	67	52	143	21	0	92	43
7/28/2007	298	274	71	0	0	0	67	52	143	21	0	92	8

Table A-6 – Daily Production Volumes for February 2007 (acre-feet)

	Green Spot	Van Dusen	Well 1	Well 1B	Well 2	Well 3	Well 3A	Well 4	Well 4A	Well 6	Well 8	Well 9	Well 10	Daily Totals
7/1/2007	0.991	0.895	0.058	0.009	0.000	0.000	0.068	0.000	0.014	0.028	0.000	0.967	0.012	3.04
7/2/2007	1.004	0.627	0.000	0.000	0.000	0.000	0.000	0.003	0.000	0.000	0.000	0.783	0.000	2.42
7/3/2007	1.126	0.698	0.000	0.000	0.000	0.000	0.000	0.184	0.000	0.000	0.000	0.000	0.000	2.01
7/4/2007	1.007	0.831	0.249	0.000	0.000	0.000	0.250	0.184	0.051	0.054	0.000	0.000	0.147	2.77
7/5/2007	0.878	0.613	0.249	0.000	0.000	0.000	0.000	0.252	0.051	0.054	0.000	0.101	0.000	2.20
7/6/2007	0.985	0.629	0.322	0.015	0.000	0.000	0.220	0.252	0.035	0.095	0.000	0.095	0.132	2.78
7/7/2007	1.074	0.781	0.018	0.015	0.000	0.031	0.019	0.009	0.035	0.006	0.000	0.000	0.018	2.01
7/8/2007	0.939	0.930	0.153	0.040	0.000	0.021	0.000	0.120	0.034	0.037	0.000	0.000	0.169	2.44
7/9/2007	0.997	0.514	0.000	0.000	0.000	0.000	0.078	0.000	0.000	0.000	0.000	0.144	0.000	1.73
7/10/2007	1.016	0.815	0.158	0.000	0.000	0.000	0.182	0.242	0.034	0.077	0.000	0.172	0.054	2.75
7/11/2007	1.013	0.815	0.158	0.000	0.000	0.000	0.000	0.000	0.034	0.000	0.000	0.000	0.054	2.07
7/12/2007	0.945	0.613	0.491	0.000	0.000	0.000	0.284	0.374	0.104	0.123	0.000	0.279	0.169	3.38
7/13/2007	0.961	0.783	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	1.74
7/14/2007	0.912	0.604	0.350	0.003	0.000	0.000	0.222	0.009	0.078	0.086	0.000	0.196	0.135	2.59
7/15/2007	0.967	0.787	0.083	0.003	0.000	0.000	0.028	0.325	0.013	0.021	0.000	0.000	0.009	2.24
7/16/2007	0.936	0.643	0.178	0.000	0.000	0.000	0.102	0.132	0.038	0.046	0.000	0.153	0.064	2.29
7/17/2007	1.001	0.395	0.034	0.000	0.000	0.000	0.001	0.028	0.009	0.000	0.000	0.000	0.000	***1.47***
7/18/2007	1.001	1.187	0.537	0.000	0.000	0.000	0.331	0.267	0.012	0.141	0.000	0.331	0.031	3.84
7/19/2007	0.951	0.826	0.310	0.000	0.000	0.000	0.180	0.381	0.167	0.077	0.000	0.175	0.279	3.35
7/20/2007	0.761	0.494	0.074	0.000	0.000	0.000	0.073	0.055	0.015	0.028	0.000	0.114	0.000	1.61
7/21/2007	0.970	0.677	0.359	0.037	0.000	0.037	0.186	0.276	0.076	0.083	0.000	0.104	0.150	2.96
7/22/2007	0.912	0.771	0.009	0.021	0.000	0.012	0.003	0.018	0.006	0.003	0.000	0.000	0.012	1.77
7/23/2007	0.852	0.712	0.009	0.000	0.000	0.000	0.000	0.341	0.094	0.114	0.000	0.172	0.132	2.42
7/24/2007	0.792	0.719	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.018	1.53
7/25/2007	0.948	0.689	0.000	0.000	0.000	0.000	0.000	0.632	0.175	0.230	0.000	0.261	0.295	3.23
7/26/2007	0.985	0.879	0.000	0.000	0.000	0.000	0.000	0.135	0.038	0.003	0.000	0.169	0.071	2.28
7/27/2007	0.878	0.629	0.218	0.000	0.000	0.000	0.021	0.160	0.044	0.064	0.000	0.282	0.132	2.43
7/28/2007	0.915	0.629	0.218	0.000	0.000	0.000	0.021	0.160	0.044	0.064	0.000	0.282	0.025	2.36

Appendix B

Distributed Parameter Watershed Model Documentation

Distributed Parameter Watershed Model Documentation

Big Bear City Community Services District

June 10, 2009

Daniel B. Stephens & Associates, Inc.

6020 Academy NE, Suite 100 • Albuquerque, New Mexico 87109

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1 Description of Input File ipm Parameters

1. Introduction

This report has been prepared by Daniel B. Stephens & Associates, Inc. (DBS&A) on behalf of Big Bear City Community Services District (BBCCSD) in support of its water master plan.

The BBCCSD has tasked DBS&A to quantify the natural recharge in the Big Bear Valley watershed. Natural recharge was estimated to range from 3,500 to 10,000 acre-feet per year (ac-ft/yr). The recharge rate was calculated by a physically based water balance model. In the DPWM, recharge is the quantity of water passing below the root zone and is referred to as net infiltration. In DBS&A's original analysis, basin-wide estimates of mean annual precipitation from DAYMET (1981-2003) fell between recorded precipitation levels recorded at the National Weather Service (NWS) gauges at Big Bear Lake Dam and the City of Big Bear Lake (2008).

2. Distributed Parameter Watershed Model Documentation

2.1 Model Description

The DPWM estimates the daily water balance for discrete locations in a watershed. Using climate, soil, vegetation, and bedrock properties distributed throughout the watershed at the cell scale, the water balance components can be computed for each of the individual cells. Rather than a lumped water balance for the entire watershed, the DPWM can map at a relatively fine spatial scale the rates of recharge, runoff, and evapotranspiration. Water balance components include precipitation, runon, runoff, evaporation, transpiration, net infiltration (recharge), soil water storage, snowmelt, snowpack accumulation, and sublimation. The DPWM has the ability to route runoff through the watershed using an elevation-based algorithm. Rather than simply assuming that a percentage of runoff becomes recharge, routing of runoff allows for the water to enter the water balance computations of downstream cells, where it can ultimately become runoff from the basin, net infiltration, change in soil water storage, or evapotranspiration.

The DPWM uses the widely accepted FAO-56 method for computing water uptake by vegetation (Allen et al., 1998). The FAO-56 method uses the FAO Penman-Monteith equation to estimate the reference evapotranspiration for the given latitude, elevation, and climate conditions. The DPWM implements an unpublished modification to the reference evapotranspiration calculation by one of the authors of FAO-56 that accounts for the slope and orientation of the cell (Allen and Trezza, 2006). The reference evapotranspiration is for a well watered grass with an assumed height of 0.12 meter, a fixed surface resistance of 70 seconds per meter (s/m) and an albedo of 0.23. The reference evapotranspiration is translated to another type of vegetation using what is termed a “crop coefficient”. The reference crop coefficient is 1.0; vegetation that transpires less than the reference crop will have a crop coefficient less than 1.0, while vegetation that transpires more than the reference crop will have a crop coefficient greater than 1.0. The crop coefficient typically changes for most vegetation as it enters different growth stages (initial, development, peak or mid-season, late season, and dormancy). Although the FAO-56 method was originally developed for agricultural vegetation, it has been adapted for natural vegetation including desert vegetation. The FAO-56 method provides equations for estimating crop coefficients of natural vegetation based on leaf area

index (LAI) or fraction of vegetation ground cover (Allen et al., 1998). The FAO-56 method provides additional equations for modifying the crop coefficients for natural vegetation based on measurements of stomatal resistance. This can account for the ability of desert vegetation to conserve water by closing their stomata. Also, any wilting point pressure can be used with the FAO-56 method. Typical agricultural vegetation has a wilting point of about 15 bars. The DPWM reduces evaporation and transpiration as the soil dries using evaporation and transpiration reduction coefficients described in the FAO-56 method (Allen et al., 1998).

The water balance calculation in the DPWM generally follows the FAO-56 method except in shallow depth soils where bedrock may restrict net infiltration. The water balance calculation can use either a field capacity approach or the van Genuchten-Mualem (VGM) model. The field capacity approach allows for instantaneous drainage when the water level in the soil is above the level determined from field capacity (typically the water level corresponding to capillary pressures of 1/10 to 1/3 bar) and no drainage when the water level is below field capacity. The VGM water balance allows for continuous drainage but at progressively lower rates as the soil dries. The VGM method assumes a unit gradient with downward flow only.

In both the field capacity and VGM models, drainage from the soil layers in a cell is restricted by the soil saturated hydraulic conductivity. Additionally, drainage or net infiltration from the bottom of the cell is restricted by the bedrock saturated hydraulic conductivity. Runoff can occur when the water content equals the saturated water content of the soil or the infiltration rate exceeds the saturated hydraulic conductivity of the uppermost soil layer.

Climate inputs to DPWM include daily total precipitation, minimum and maximum air temperature, wind speed, and the duration of precipitation for a reference location in the watershed. Precipitation and temperature are adjusted from the reference point in the basin for the elevation of each cell using a simple linear regression. The dewpoint temperature is adjusted downward below the minimum daily temperature as described in FAO-56 for arid environments (Allen et al., 1998).

The DPWM does not have the ability to calculate lateral saturated or unsaturated subsurface flow. It also cannot calculate stream flow other than simple elevation-based ephemeral runoff

routing. The DPWM is not an ecological model in that it does not simulate root growth, migration of vegetation types, plant height growth, etc.

2.1.1 Description of DPWM Model Cell

The computational "core" of the DPWM is the "cell", which describes the soil and water transport in a comparatively small region. The cells model a 90-meter by 90-meter region.

The cell is composed of four nodes (Figure 1). Nodes 1 and 2 comprise the first layer, which is only a few centimeters deep. Node 1 models the bare soil region near the ground surface. In addition to plant transpiration via plant roots, higher levels of short-term evaporation extract water from this region. Node 2 models the ground surface under the plant canopy. Shade from the canopy strongly reduces the evaporation component from this node. The small fraction of evaporation that occurs is lumped into the transpiration component from this node.

The balance of the root zone is modeled by Node 3. In most soils, most of the plant transpiration takes place in this node. Vapor migrations to the ground surface, caused by gradients in vapor pressure, are lumped into the transpiration term.

The balance of the soil region is modeled by Node 4. Neither evaporation nor transpiration takes place with this node. However, if the underlying rock has a sufficiently low permeability, water can accumulate within Node 4 and drain into the rock over a prolonged period of time.

2.1.2 Compiling and Executing DPWM

The DPWM was written in the C/C++ computer language. The code is relatively easy to understand for anyone experienced with computer languages, in that it is simply composed of function calls, if-then statements, arithmetic expressions, and for-loops. Executables have been compiled in release mode with Microsoft Visual C++ version 7.1.6030. Microsoft compilers (available for free at http://www.microsoft.com/express/vc/) have also been used to successfully compile DPWM. The DPWM is executed at the command line. All input and output files have the same root name with different extensions. The DPWM will query the user to enter the root

name for a simulation or the user can use the DOS redirection command to enter the root name automatically from a text file (e.g., DPWM < root.txt).

2.1.3 Input Files

There are seven input files for the DPWM, one of which is optional. All files are standard ASCII text files that can be edited with any text editing software. The nomenclature for the input file extension names is “i” for input followed by a two-letter abbreviation for the input file type (e.g., ipm for the input parameter file).

2.1.3.1 Input Parameter File (.ipm)*

The parameter input file has the soil, vegetation, bedrock, and general model parameter values. The file has two columns: (1) the parameter name identifier and (2) the parameter value. Table 1 describes the input parameters and provides the units. The file is in a space delimited format.

2.1.3.2 Input Climate File (.icl)*

The climate input file has climate data for the reference location in the watershed. Columns are as follows:

- Month
- Day of month [DOM]
- Water year
- Day of water year [DOWY]
- Precipitation in millimeters (mm) [PRECIP]
- Maximum daily temperature in °C [TMAX]
- Minimum daily temperature in °C [TMIN]
- Wind speed in meters per second (m/s) [WIND]
- Duration of precipitation in hours [DURATION]

The file is in a space delimited format.

2.1.3.3 Input Watershed File (.iws)*

The watershed input file has the cell location and elevation along with the types of soil, vegetation and bedrock. Columns are as follows:

- Cell ID [BBCCSD_ID]
- UTM Zone 11 easting in NAD83 meters [POINT_X]
- UTM Zone 11 northing in NAD83 meters [POINT_Y]
- Elevation of cell in meters [ELEV_METER]
- Cell ID of downstream cell that receives runoff [DWNSTRM_ID]
- Slope of cell in degrees [SLOPE_DEG]
- Aspect of cell [ASPECT]
- Soil type index with array origin at 1 [Soil_Index]
- Bedrock type index with array origin at 1 [Rock_Index]
- Vegetation type with array origin at 1 [Veg_Index]
- Area of cell in meters squared [Area2]

This file must be ordered with upstream cell above downstream cell. The file is in a space delimited format.

2.1.3.4 Input Downstream Receptor File (.idn)*

The downstream contributor file instructs the DPWM how to route runoff. Columns are as follows:

- C/C++ array index with 0 origin [RankJ]
- Cell ID [BBCCSD_ID]
- Cell ID of downstream cell that will receive runon [DWNSTREAM_ID]
- C/C++ array index of downstream cell with 0 origin [Dwnstrm_J]

Index values must correspond with positions in the watershed file (*.iws). The second line in this file (first line after header) corresponds with array index 0. The file is in a space delimited format.

2.1.3.5 Input Daily Observation File for Specified Cells (.iob)*

The input file identifies individual cells to monitor daily water balance. The first line is the number of cells to monitor. Subsequent lines have cell IDs. The code currently allows a maximum of 40 monitor cells. Output of monitored cells is in the output file *.ocd.

2.1.3.6 Input Observation File for Specified Times (.iot)*

The input file identifies times to output water balance for entire watershed. The first line is the number of output times. Subsequent lines have water year, month, and day of month to generate output. The file is in a tab delimited format. Output can be found in the *.oct file.

2.1.4 Output Files

Output files provide the mass balance components (precipitation, evaporation, transpiration, net infiltration, runoff, storage, snowpack level, and error) for the water balance calculations at the watershed or cell level and at either daily, annual, or other specified time intervals. Generally the nomenclature for output file extensions is “o” for output, “c” for cell or “w” for watershed, and “d” for daily or “a” for annual (e.g., owd is mass balance components for the watershed on a daily basis). Unless otherwise specified, the units for the output files are in cubic meters. To obtain units of length (e.g., inches of rain on a cell), the volume of water must be divided by the area of the cell found in the input watershed file (*.iws).

2.1.4.1 Output Watershed Daily Mass Balance (.owd)*

This output file contains the daily water balance for the entire watershed on a lumped basis. Columns are as follows:

- Day of run [Day]
- Total precipitation in cubic meters [Precip]
- Change in water stored for watershed in cubic meters [dStorage]
- Change in water stored in the snowpack in cubic meters [dSnow]
- Evapotranspiration for watershed in cubic meters [ET]
- Net infiltration for watershed in cubic meters [Infil]
- Runoff into lakes in cubic meters [RunoffExit]

- Mass balance for model in cubic meters [Masscheck]

2.1.4.2 Output Cell Annual Mass Balance (.oca)*

This output file has the annual water balance for each cell in the watershed. Columns are as follows:

- Cell ID [ID]
- Water year [Year]
- UTM Zone 11 easting in NAD83 meters [UTM83_X]
- UTM Zone 11 northing in NAD83 meters [UTM83_Y]
- Precipitation volume for year on cell in cubic meters [Precip]
- Actual evapotranspiration in cubic meters [AET]
- Net infiltration in cubic meters [Infil]
- Runoff in cubic meters [Runoff]
- Runon in cubic meters [Runon]
- Total change in water stored in soil in cubic meters [dWlevel]
- Total change of water in snowpack in cubic meters [dSnow]
- Area of cell in square meters [Area]

The file is in a tab delimited format. This file can be read by ArcMAP, but ArcMAP sometimes does not display text files correctly. It is better to import the file into Microsoft Access for reading into ArcMAP.

2.1.4.3 Output for Specified Cells Daily Mass Balance (.ocd)*

This file has the daily water balance for individual cells specified in the input file *.iob. Columns are as follows:

- Day of run [Day]
- Cell ID [CellID]
- Change in water stored in soil in cubic meters [dWlevel]
- Daily precipitation in cubic meters [Precip]

- Daily transpiration in cubic meters [Trans]
- Daily evaporation in cubic meters [Evap]
- Daily runon in cubic meters [Runon]
- Daily runoff in cubic meters [Runoff]
- Daily net infiltration in cubic meters [Infil]
- Daily water balance error in cubic meters [Balance]
- Actual evapotranspiration in mm [AET_mm]
- Reference evapotranspiration in mm [RefET_mm]
- Kcb [Kcb]
- Maximum Kc [Kcmax]
- Vegetation canopy cover fraction (fc)
- Water level in Node 1 in cubic meters [WL1]
- Water level in Node 2 in cubic meters [WL2]
- Water level in Node 3 in cubic meters [WL3]
- Water level in Node 4 in cubic meters [WL4]
- Mid-season Kcb [Kcb_mid]
- Full-season Kcb [Kcb_full]
- Minimum daily relative humidity [RH_min]

The file is space delimited.

2.1.4.4 Output Watershed Annual Mass Balance (.owa)*

This file has the water balance for the entire lumped watershed on an annual basis. Columns are as follows:

- Water year [Year]
- Total annual precipitation on watershed in cubic meters [Precip]
- Actual evapotranspiration for entire watershed in cubic meters [AET]
- Total net infiltration (e.g., recharge) for watershed in cubic meters [Infil]
- Runoff from watershed into lakes in cubic meters for year [GapRunoff]
- Change in water storage for year over watershed in cubic meters [dStorage]

- Mass balance error for watershed in percent [MBE%]
- Runoff from mountain block onto alluvium (e.g., mountain front runoff as described in Wilson and Guan [2004]) in cubic meters for year [MF_Runoff]

The file is space delimited.

2.1.4.5 Output All Cells at Specified Times (.oct)*

This file contains the water balance for specified days for the entire watershed. Columns are as follows:

- Cell ID [ID]
- Water year [Year]
- Month [Month]
- Day of month [Day]
- UTM easting for zone 11 and NAD83 in meters [UTM83_Xm]
- UTM northing for zone 11 and NAD83 in meters [UTM83_Ym]
- Area of cell in square meters [Area_m2]
- Water level in Node 1 in mm [WL1mm]
- Water level in Node 2 [WL2mm]
- Water level in Node 3 in mm [WL3mm]
- Water level in Node 4 in mm [WL4mm]
- Depth-averaged volumetric water content in Node 1 [Q1]
- Depth-averaged volumetric water content in Node 2 [Q2]
- Depth-averaged volumetric water content in Node 3 [Q3]
- Depth-averaged volumetric water content in Node 4 [Q4]
- Unsaturated hydraulic conductivity in Node 1 in millimeters per day (mm/d) [K(Q1)mm•d-1]
- Unsaturated hydraulic conductivity in Node 2 in mm/d [K(Q2)mm•d-1]
- Unsaturated hydraulic conductivity in Node 3 in mm/d [K(Q3)mm•d-1]
- Unsaturated hydraulic conductivity in Node 4 in mm/d [K(Q4)mm•d-1]
- Precipitation at cell in mm [Precip_mm]

- Net infiltration at cell in mm [Infil_mm]
- Runoff at cell in mm [Runoff_mm]
- Transpiration in cell in mm [Trans_mm]
- Evaporation in cell in mm [Evap_mm]
- Snow stored at cell in mm [Snow]
- Total change in water level at cell in mm [dWlevel]
- Reference evapotranspiration in mm [refETmm]
- Minimum relative humidity in percent [RHmin]
- Kcb [Kcb]
- Capillary pressure head in centimeters (cm) at Node 1 [h_cp1_cm]
- Capillary pressure head in cm at Node 2 [h_cp2_cm]
- Capillary pressure head in cm at Node 3 [h_cp3_cm]
- Capillary pressure head in cm at Node 4 [h_cp4_cm]

–9.999 values indicate that a node does not exist.

2.1.4.6 Echo of Input and Output of Calculated Input Values (check.txt or .chk)

This file echoes input data and outputs calculated input values. The file will flag errors in the runoff routing input file (idn) and shows the annual watershed water balances in acre-feet.

2.1.5 Main Program Routine

The main program routine reads the input files, opens the output files, calculates initial properties, and computes the water balance components for each cell and for each day in the simulation. Functions given below are described in more detail in the sections following this section. The main program begins by reading the input files and opening the output files for writing. Next the initial properties for the cells are computed from the input files. Input file units are converted to units of mm and days. The cdepth_fcn is called for each cell to calculate the thicknesses of the nodes from the total soil depth, as follows:

$$Thick_{1-4} = cdepth_fcn(Depth, Ze, Zr_{veg}) \quad (1)$$

where $Thick_{1\text{-}4}$ = the thickness of Nodes 1 through 4 (mm)

Depth = the soil depth specified for the soil type of the cell converted to mm

Ze = the evaporation layer thickness (mm)

Zr = the rooting depth of the vegetation at the cell (mm)

If the vegetation index indicates that the cell is bare rock, the depth is set to the evaporation depth (Ze) to allow for surface storage and evaporation and the soil hydraulic conductivity is set to the bedrock hydraulic conductivity. For cells that are washes, the hydraulic properties of the soil are set to those specified for washes.

The maximum water level in each cell node is set based on the saturated water content and node thickness, as follows:

$$\theta s_level_{1-4} = \theta s \cdot Thick_{1-4} \tag{2}$$

where $\theta s_level_{1\text{-}4}$ = the water level equivalent to saturation in the node (mm)

θs = the saturated water content from the soil type at the cell

The water contents associated with the field capacity and wilting point capillary pressure heads are computed, as follows:

$$\begin{aligned} \theta_{FC} &= vg_head_to_wc(\theta r, \theta s, \alpha, n, h_{FC}) \\ \theta_{WP} &= vg_head_to_wc(\theta r, \theta s, \alpha, n, h_{WP}) \end{aligned} \tag{3}$$

where θ_{FC} = the field capacity water content

θ_{WP} = the wilting point water content, θr is the residual water content

θs = the saturated water content

α and n = the van Genuchten curve fitting parameters

h_{FC} = the field capacity capillary pressure head (cm) specified by the user

h_{WP} = the wilting point capillary pressure head (cm) specified by the user

The water levels equivalent to the field capacity and wilting point water contents are calculated as follows:

$$\begin{aligned} FC_{1-4} &= Thick_{1-4} \cdot \theta_{FC} \\ WP_{1-4} &= Thick_{1-4} \cdot \theta_{WP} \end{aligned} \tag{4}$$

where FC_{1-4} = the water level equivalent to the field capacity water content for Nodes 1 through 4 (mm)

WP_{1-4} = the water level equivalent to the wilting point water content for Nodes 1 through 4 (mm)

The FAO-56 parameters for total evaporable water (TEW) and total available water (TAW) are computed for each cell based on the equations in Allen et al. (1998), as follows:

$$\begin{aligned} TEW &= \left(\theta_{FC} - 0.5 \cdot \theta_{WP}\right) \cdot Thick_1 \\ TAW &= \left(\theta_{FC} - \theta_{WP}\right) \cdot \left(Thick_1 + Thick_3\right) \end{aligned} \tag{5}$$

where θ_{FC} = the field capacity water content

θ_{WP} = the wilting point water content

$Thick_1$ = the thickness of Layer 1 from Node 1 (Nodes 1 and 2 in Layer 1 have the same thickness and either could have been used here)

$Thick_3$ = the thickness of Layer 2 from Node 3

The initial water levels in each node of each cell are set based on the user specified capillary pressure heads in each node, as follows:

$$\begin{aligned} \theta_{1-4} &= vg_head_to_wc\left(\theta r, \theta s, \alpha, n, h_{i,1-4}\right) \\ Wlevel_{1-4} &= \theta_{1-4} \cdot Thick_{1-4} \end{aligned} \tag{6}$$

where θ_{1-4} = the water content in Nodes 1 through 4 in each cell

θr = the residual water content

θs = the saturated water content

α and n = the curve fitting parameters

$hi_{,1\text{-}4}$ = the initial capillary pressure head for Nodes 1 through 4 specified by the user

Typically, the initial water levels are set to the wilting point in Nodes 1 through 3 and to field capacity in Node 4. The water in Node 4 is stagnant (will not drain or evapotranspire) when set at or below the field capacity.

After the initial properties have been calculated, they are printed to the output file check.txt or *.chk for verification. The next step in the main program routine is the daily water balance calculation for each cell and for each day. For each day of the simulation, the program loops through all of the cells as ordered in the watershed file. The cells in the watershed file must be ordered so that no cell is below a cell that is downstream (the program checks that the order is correct and if not correctly ordered will stop execution). Before the cell calculations, the program calculates the dewpoint offset (Koffset), as follows:

$$Koffset = KdewOffset_fcn(DOY, A, L, V, C) \tag{7}$$

where DOY = the calendar day of the year starting January 1

A, L, V, and C = the fitting parameters of the harmonic function

Then the calculation for individual cells begins with correction of the reference temperature to the cell elevation using the following function that returns the average, maximum, minimum, and dewpoint temperatures for the cell:

$$T_elev_cor_fcn(Tref_{\max}, Tref_{\min}, Elev_{cell}, Elev_{ref}, C_{temp}, Koffset) \tag{8}$$

where $Tref_{max}$ = the maximum daily temperature at the reference location

$Tref_{min}$ = the minimum daily temperature for the reference location

$Elev_{cell}$ = the elevation of the cell

$Elev_{ref}$ = the elevation of the reference location

C_{temp} = the correlation of temperature with elevation specified by the user

Koffset = the dewpoint temperature offset

Similarly, the average, maximum, minimum and dewpoint temperatures for the average elevation of the watershed are computed as follows:

$$T_elev_cor_fcn(Tref_{\max}, Tref_{\min}, Elev_{average}, Elev_{ref}, C_{temp}, Koffset) \quad (9)$$

where $Elev_{average}$ = the average elevation for all the cells in the watershed

The correction to the daily total precipitation for elevation is computed for the cell from the reference location, as follows:

$$P = precip_elev_cor_fcn\left(P_{ref}, Elev_{cell}, Elev_{ref}, Cprecip\right) \quad (10)$$

where P = the daily total precipitation at the cell

P_{ref} = the reference daily total precipitation

$Elev_{cell}$ = the elevation of the cell (m)

$Elev_{ref}$ = the elevation of the reference location (m)

Cprecip = the correlation between precipitation and elevation specified by the user

Next, the crop and vegetation canopy coefficients are calculated. The atmospheric pressure (Patm), psychrometric constant (γ), slope of the saturation vapor pressure curve (Δ), and the minimum relative humidity are calculated as follows:

$$\begin{aligned} Patm &= CellP_fcn(Elev_{cell}) \\ \gamma &= Psych_fcn(Patm) \\ \Delta &= slope_es_fcn(Tavg_{cell}) \\ RH_{\min} &= RH_\min_fcn(Tdew_{cell}, T\max_{cell}) \end{aligned} \quad (11)$$

The Kcb for full vegetation cover is estimated as follows:

$$Kcb_{full} = KcbFull_fcn(wind, h_{plant}, RH_{\min}) \tag{12}$$

where wind = the wind speed at the reference location (m/s)

h_{plant} = the peak season plant height (m)

RH_{min} = the minimum daily relative humidity

The stomatal resistance adjustments (Fr) are estimated for the initial, peak mid-season, and the late season, as follows:

$$\begin{aligned} Fr_{ini} &= Fr_fcn(\Delta, \gamma, wind, rl_{ini}) \\ Fr_{mid} &= Fr_fcn(\Delta, \gamma, wind, rl_{mid}) \\ Fr_{end} &= Fr_fcn(\Delta, \gamma, wind, rl_{end}) \end{aligned} \tag{13}$$

where rl_{ini}, rl_{mid}, and rl_{end} are the stomatal resistance values specified by the user for the vegetation for the initial growth season, peak mid-season, and the late season, respectively.

The Kcb values for the initial (Kcb_{ini}), peak mid-season (Kcb_{mid}), and late season (Kcb_{end}) are calculated as follows:

$$\begin{aligned} Kcb_{ini} &= KcbLAI_fcn(Kcb_{full}, LAI_{ini}, Fr_{ini}, Kc_{\min}) \\ Kcb_{mid} &= KcbLAI_fcn(Kcb_{full}, LAI_{mid}, Fr_{mid}, Kc_{\min}) \\ Kcb_{end} &= KcbLAI_fcn(Kcb_{full}, LAI_{end}, Fr_{end}, Kc_{\min}) \end{aligned} \tag{14}$$

where LAI = the leaf area index specified by the user for the initial, mid-season, or late season

Kc_{min} = the minimum crop coefficient for bare soil specified by the user

The Kcb_fcn is then called to estimate the Kcb for any given day of the year:

$$Kcb = Kcb_fcn(DOY, Tdew_{cell}, T\max_{cell}, wind, h_{plant}, LAI_{mid}, Kcb_{ini}, Kcb_{mid}, Kcb_{end}, DevelopStart, MidStart, MidEnd, LateEnd, Kc_{\min}) \tag{15}$$

where DOY = the day of the calendar year

$Tdew_{cell}$ = the dewpoint temperature at the cell

$Tmax_{cell}$ = the maximum daily temperature at the cell

wind = the wind speed at the reference location

h_{plant} = the plant height during the peak season specified by the user for the vegetation type

LAI_{mid} = the peak season leaf area index

DevelopStart = the start of growth development after the initial period

MidStart = the start of the peak growth season

MidEnd = the end of the peak growth season

LateEnd = the end of the late season

The Kcb function returns the basal transpiration coefficient for the cell and day (Kcb), maximum transpiration coefficient (Kc_{max}), and the fraction of ground covered by vegetation canopy (f_c).

Next, water is transferred between Nodes 1 and 2 in Layer 1 as the canopy cover fraction changes:

$$Varying_f_c_fcn(f_c, f_{c_old}, Wlevel_1, Wlevel_2) \tag{16}$$

where f_c = the new vegetation canopy cover fraction

f_{c_old} = the previous canopy cover fraction

$Wlevel_1$ = the water level in Node 1

$Wlevel_2$ = the water level in Node 2

This function is necessary to maintain the water balance in the cell. As the canopy cover increases or decreases, the sizes of Nodes 1 and 2 change and water must be appropriately transferred. The function returns updated values for Wlevel1 and Wlevel2.

The snow function updates snowpack levels, snowmelt and sublimation:

$$SnowB_fcn(Slevel_{old}, Tavg_{cell}, T\max_{cell}, T\min_{cell}, Tdew_{cell}, P, C_{sublime}, C_{melt}) \tag{17}$$

where $Slevel_{old}$ = the old snowpack level

P = the total daily precipitation

$C_{sublime}$ = the fraction of snowpack that sublimates over the winter

C_{melt} = the rate of snowmelt

If the snowpack level is greater than zero, the albedo of the cell is set to 0.60 (as done in the HELP model [Schroeder et al., 1994]).

The reference evapotranspiration (RefET) is calculated using the function RefET_fcn:

$$RefET = RefET_fcn(Tavg_{cell}, T\max_{cell}, T\min_{cell}, Tdew_{cell}, Tavg_{ws}, T\max_{ws}, T\min_{ws}, Tdew_{ws}, Elev_{cell}, Elev_{ref}, Slope_{cell}, Azimuth_{cell}, DOY, wind, Krs, Latitude, Albedo) \quad (18)$$

where $Tavg_{cell}$ = the average daily temperature at the cell

$Tmax_{cell}$ = the maximum daily temperature at the cell

$Tmin_{cell}$ = the minimum daily temperature at the cell

$Tdew_{cell}$ = the dewpoint temperature at the cell

$Tavg_{ws}$ = the average daily temperature for the average elevation in the watershed

$Tmax_{ws}$ = the maximum daily temperature for the average elevation in the watershed

$Tmin_{ws}$ = the minimum daily temperature for the average elevation in the watershed

Tdew = the dewpoint temperature for the average elevation in the watershed

$Elev_{cell}$ = the elevation of the cell

$Elev_{ref}$ = the elevation of the reference station

$Slope_{cell}$ = the slope of the cell

$Azimuth_{cell}$ = the azimuth of the cell

DOY = the calendar day of the year

wind = the wind speed at the reference location

Krs = the adjustment to incoming solar radiation for sky clearness

latitude = the average latitude of cells in the basin

albedo = the specified albedo of the cell

Next, the volume of runon from upstream cells is converted to an equivalent height for the cell water balance. The total water applied to the cell includes precipitation, runon, and snowmelt. If the total water applied exceeds the saturated hydraulic conductivity of the upper soil layer, the excess water becomes runoff. The remaining water is added to the water levels in Layer 1 of the cell and the GroupBalance function is called to update the water levels in all nodes, and compute runoff and net infiltration. The runoff is added to the runon term in the downstream cell.

The amount of evaporation, transpiration, and the resultant changes in nodal water levels are computed in ET_fcn:

$$ET_fcn(DOY, Kcb, Kc_{\max}, f_c, RefET, Wlevel_{1-4}, FC_{1-4}, WP_{1-4}, REW, TEW, TAW, Snow, Thick_{1-4}, p) \quad (19)$$

where DOY = the calendar day of the year

Kcb = the basal transpiration coefficient

Kc_{max} = the maximum basal transpiration coefficient

f_c = the canopy cover fraction

RefET = the reference evapotranspiration

$Wlevel_{1-4}$ = the water levels in Nodes 1 through 4

FC_{1-4} = the water level equivalent to field capacity in Nodes 1 through 4

WP_{1-4} = the water level equivalent to the wilting point in Nodes 1 through 4

p = the depletion coefficient that controls when transpiration is reduced due to water stress

After the water balance has been computed for each cell in the watershed, the main routine prints results to the output files.

2.1.6 Functions

2.1.6.1 BalanceFC_fcn

This function calculates water redistribution between nodes for a cell using the field capacity method, and computes runoff and net infiltration. If precipitation or snowmelt occurs on a particular day, BalanceFC_fcn is called twice—first for the duration of the precipitation/melting event and then for the balance of the day. If no precipitation or melting occurs, water in excess of the field capacity may yet exist in one or more nodes due to precipitation or melting on a previous day. For this case, BalanceFC_fcn is called once for the entire day.

The initial step in the function is to reduce the soil and bedrock saturated hydraulic conductivities for the fraction of the day for the calculation, as follows:

$$\begin{aligned} Ksoil_{frac} &= Ksoil \cdot fracDt \\ Krock_{frac} &= Krock \cdot fracDt \end{aligned} \tag{20}$$

where $Ksoil_{frac}$ = the reduced soil hydraulic conductivity (mm)
Ksoil = the soil saturated hydraulic conductivity (mm/d)
fracDt = the fraction of the day for the balance calculation (day)
$Krock_{frac}$ = the reduced bedrock saturated hydraulic conductivity (mm)
Krock = the bedrock saturated hydraulic conductivity (mm/d)

The next step is to calculate the amount of water that can drain from Node 1 if the water level in Node 1 exceeds field capacity. Drainage is the minimum of the difference between the water level and field capacity or the reduced soil hydraulic conductivity. The water level in Node 1 is reduced for any drainage that occurs from Node 1, as follows:

$$\begin{aligned} Drain_1 &= \min(Ksoil_{frac}, Wlevel_1 - FC_1) \geq 0 \\ Wlevel_1 &= Wlevel_1 - Drain_1 \end{aligned} \tag{21}$$

where $Drain_1$ = the drainage from Node 1 (mm)

$Ksoil_{frac}$ = the reduced soil saturated hydraulic conductivity (mm)

$Wlevel_1$ = the water level in Node 1 (mm)

FC_1 = the water level equivalent of field capacity (mm)

Next, the drainage from Node 2 is calculated if the water level in Node 2 is greater than field capacity. Drainage is the minimum of the water level and field capacity drainage or the adjusted soil hydraulic conductivity, as follows:

$$\begin{aligned} Drain_2 &= \min(Ksoil_{frac}, Wlevel_2 - FC_2) \geq 0 \\ Wlevel_2 &= Wlevel_2 - Drain_2 \end{aligned} \tag{22}$$

where $Drain_2$ = the drainage from Node 2 (mm)

$Ksoil_{frac}$ = the adjusted soil saturated hydraulic conductivity (mm)

$Wlevel_2$ = the water level in Node 2 (mm)

FC_2 = the water level equivalent of field capacity (mm)

If drainage from Node 2 is less than the adjusted soil hydraulic conductivity and there is water in Node 1 in excess of the saturated water content water level (θs_level1), the excess water in Node 1 is transferred to Node 2 and drainage from Node 2 is recomputed, as follows:

$$\begin{aligned} Wlevel_2 &= Wlevel_2 + Drain_2 \\ \Delta Wlevel_\max_1 &= Wlevel_1 - \theta s_level_1 \\ \Delta Wlevel_\max_2 &= Ksoil_{frac} - \max(Wlevel_2 - FC_2, 0) \\ \Delta Wlevel_2 &= \min\left(\Delta Wlevel_\max_1 \frac{1 - f_c}{f_c}, \Delta Wlevel_\max_2\right) \\ Wlevel_2 &= Wlevel_2 + \Delta Wlevel_2 \\ Drain_2 &= Wlevel_2 - FC_2 \geq 0 \\ Wlevel_2 &= Wlevel_2 - Drain_2 \\ Wlevel_1 &= Wlevel_1 - \Delta Wlevel_2 \frac{f_c}{1 - f_c} \end{aligned} \tag{23}$$

Similarly, if there is excess water in Node 2 and drainage in Node 1 is not at the maximum, water is transferred from Node 2 to Node 1 and Node 1 drainage is recomputed, as follows:

$$
\begin{aligned}
&Wlevel_1 = Wlevel_1 + Drain_1 \\
&\Delta Wlevel_\max_2 = Wlevel_2 - \theta s_level_2 \\
&\Delta Wlevel_\max_1 = Ksoil_{frac} - \max(Wlevel_1 - FC_1, 0) \\
&\Delta Wlevel_1 = \min\left(\Delta Wlevel_\max_2 \frac{f_c}{1-f_c}, \Delta Wlevel_\max_1\right) \\
&Wlevel_1 = Wlevel_1 + \Delta Wlevel_1 \\
&Drain_1 = Wlevel_1 - FC_1 \geq 0 \\
&Wlevel_1 = Wlevel_1 - Drain_1 \\
&Wlevel_2 = Wlevel_2 - \Delta Wlevel_1 \frac{1-f_c}{f_c}
\end{aligned}
\tag{24}
$$

Then the drainage from Nodes 1 and 2 in Layer 1 is added to the water level in Layer 2 (Node 3), as follows:

$$
Wlevel_3 = Wlevel_3 + (1-f_c)Drain_1 + f_c \cdot Drain_2 \tag{25}
$$

Water in excess of field capacity Layer 2 (Node 3) is added to Layer 3 (Node 4), as follows:

$$
\begin{aligned}
&Drain_3 = Wlevel_3 - FC_3 \geq 0 \\
&Drain_3 = \min(Drain_3, Ksoil_{frac}) \\
&Wlevel_3 = Wlevel_3 - Drain_3 \\
&Wlevel_4 = Wlevel_4 + Drain_3
\end{aligned}
\tag{26}
$$

Water in excess of field capacity in Layer 3 (Node 4) becomes net infiltration, as follows:

$$
\begin{aligned}
&Drain_4 = \min(Wlevel_4 - FC_4, Ksoil_{frac}, Krock_{frac}) \geq 0 \\
&Wlevel_4 = Wlevel_4 - Drain_4 \\
&Infil = Drain_4
\end{aligned}
\tag{27}
$$

After net infiltration has been computed, any water in excess of the saturated water content in the layers is passed back up to the overlying layer. If water is in excess of the saturated water content in Layer 3 (Node 4), the excess water is added to Layer 2 (Node 3), as follows:

$$\begin{aligned} Wlevel_3 &= Wlevel_3 + \left(Wlevel_4 - \theta s_level_4\right) \\ Wlevel_4 &= \theta s_level_4 \end{aligned} \tag{28}$$

If water is in excess of the saturated water content in Layer 2 (Node 3), it is passed back up to Layer 1 and is proportioned between Nodes 1 and 2 based on the original drainage, as follows:

$$\begin{aligned} \Delta Wlevel_3 &= Wlevel_3 - \theta s_level_3 \\ Wlevel_3 &= \theta s_level_3 \\ Wlevel_1 &= Wlevel_1 + \frac{Drain_1}{\left[(1-f_c)Drain_1 + f_c \cdot Drain_2\right]} \cdot \Delta Wlevel_3 \\ Wlevel_2 &= Wlevel_2 + \frac{Drain_2}{\left[(1-f_c)Drain_1 + f_c \cdot Drain_2\right]} \cdot \Delta Wlevel_3 \end{aligned} \tag{29}$$

If the water level of Node 1 is greater than the saturation limit and the water level of Node 2 is below the saturation limit, the excess water in Node 1 is transferred to Node 2 up to the capacity of Node 2 before computing runoff, as follows:

$$\begin{aligned} \Delta Wlevel_\max_1 &= Wlevel_1 - \theta s_level_1 \\ \Delta Wlevel_2 &= \min\left(\Delta Wlevel_\max_1 \frac{1-f_c}{f_c}, \theta s_level_2 - Wlevel_2\right) \\ Wlevel_2 &= Wlevel_2 + \Delta Wlevel_2 \\ Wlevel_1 &= Wlevel_1 - \Delta Wlevel_2 \frac{f_c}{1-f_c} \end{aligned} \tag{30}$$

Similarly, if the water content in Node 2 is greater than the saturated water content and the water level in Node 1 is less than the saturated water content, the excess water in Node 2 is passed to Node 1 up to the saturated water content of Node 1 before computing runoff, as follows:

$$
\begin{aligned}
&\Delta Wlevel_\max_2 = Wlevel_2 - \theta s_level_2 \\
&\Delta Wlevel_1 = \min\left(\Delta Wlevel_\max_2 \frac{f_c}{1-f_c}, \theta s_level_1 - Wlevel_1\right) \\
&Wlevel_1 = Wlevel_1 + \Delta Wlevel_1 \\
&Wlevel_2 = Wlevel_2 - \Delta Wlevel_1 \frac{1-f_c}{f_c}
\end{aligned}
\tag{31}
$$

Water in excess of the saturated water content in Nodes 1 and 2 is transferred to runoff, as follows:

$$
\begin{aligned}
&Runoff = Wlevel_1 - \theta s_level_1(1-f_c) \\
&Wlevel_1 = \theta s_level_1 \\
&Runoff = Runoff + (Wlevel_2 - \theta s_level_2) f_c \\
&Wlevel_2 = \theta s_level_2
\end{aligned}
\tag{32}
$$

The function returns the water levels in each of the nodes, runoff, and net infiltration.

2.1.6.2 BalanceVGM_fcn

The VGM balance model is very similar to the field capacity model except that drainage from nodes is calculated using the van Genuchten–Mualem equation for the water content of the node. Unlike the field capacity model, where drainage is instantaneous when wetted above the field capacity point and zero below the field capacity point, the VGM model allows for continuous drainage at progressively slower rates. Like BalanceFC_fcn, BalanceVGM_fcn is called once on “dry” days and twice on days having precipitation or snowmelt.

The initial step in the function is to reduce the soil and bedrock saturated hydraulic conductivities for the fraction of the day for the calculation, as follows:

$$
\begin{aligned}
&Ksoil_{frac} = Ksoil \cdot fracDt \\
&Krock_{frac} = Krock \cdot fracDt
\end{aligned}
\tag{33}
$$

where $Ksoil_{frac}$ = the reduced soil hydraulic conductivity (mm)

Ksoil = the soil saturated hydraulic conductivity (mm/d)

fracDt = the fraction of the day for the balance calculation (day)

$Krock_{frac}$ = the reduced bedrock saturated hydraulic conductivity (mm)

Krock = the bedrock saturated hydraulic conductivity (mm/d)

The next step is to calculate the water content in Node 1 (θ_1), as follows:

$$\theta_1 = \frac{Wlevel_1}{Thick_1} \tag{34}$$

where $Wlevel_1$ = the water level in Node 1

$Thick_1$ =the thickness of Node 1

The drainage from Node 1 is equivalent to the unsaturated hydraulic conductivity corresponding to the above water content assuming a unit gradient. The van Genuchten–Mualem equation is used to estimate the unsaturated hydraulic conductivity at the water content:

$$K_u(\theta)_1 = Ktheta_fcn(Ksoil_{frac}, \theta r, \theta s, m, Wlevel_1, Thick_1) \tag{35}$$

where θr = the residual water content

θs = the saturated water content

m = the van Genuchten curve fitting parameter

$Wlevel_1$ = the water level in Node 1

$Thick_1$ = the thickness in Node 1

Drainage from Node 1 ($drain_1$) is the minimum of the unsaturated hydraulic conductivity or the water available:

$$drain_1 = \min(Wlevel_1 - \theta r \cdot Thick_1, K_u(\theta)_1) \tag{36}$$

The water level in Node1 is then adjusted for drainage:

$$Wlevel_1 = Wlevel_1 - Drain_1 \tag{37}$$

Drainage from Node 2 is computed next in a similar manner as Node 1:

$$\begin{aligned}
&\theta_2 = \frac{Wlevel_2}{Thick_2} \\
&K_u(\theta_2)_2 = Ktheta_fcn(Ksoil_{frac}, \theta r, \theta s, m, Wlevel_2, Thick_2) \\
&drain_2 = \min(Wlevel_2 - \theta r \cdot Thick_2, K_u(\theta_2)_2) \\
&Wlevel_2 = Wlevel_2 - drain_2
\end{aligned} \tag{38}$$

If drainage from Node 2 is below the adjusted soil hydraulic conductivity limit and there is water in Node 1 in excess of the saturated water content water level (θs_level1), the excess water in Node 1 is transferred to Node 2 and drainage from Node 2 is recomputed:

$$\begin{aligned}
&Wlevel_2 = Wlevel_2 + Drain_2 \\
&\Delta Wlevel_\max_1 = Wlevel_1 - \theta s_level_1 \\
&\Delta Wlevel_\max_2 = Ksoil_{frac} - \left(Wlevel_2 - \theta r \cdot Thick_2\right) \\
&\Delta Wlevel_2 = \min\left(\Delta Wlevel_\max_1 \frac{1 - f_c}{f_c}, \Delta Wlevel_\max_2\right) \\
&Wlevel_2 = Wlevel_2 + \Delta Wlevel_2 \\
&\theta_2 = \frac{Wlevel_2}{Thick_2} \\
&K_u\left(\theta_2\right)_2 = Ktheta_fcn\left(Ksoil_{frac}, \theta r, \theta s, m, Wlevel_2, Thick_2\right) \\
&Drain_2 = \min\left(Wlevel_2 - \theta r \cdot Thick_2, K_u\left(\theta_2\right)_2\right) \geq 0 \\
&Wlevel_2 = Wlevel_2 - Drain_2 \\
&Wlevel_1 = Wlevel_1 - \Delta Wlevel_2 \frac{f_c}{1 - f_c}
\end{aligned} \tag{39}$$

Similarly, if there is excess water in Node 2 and drainage in Node 1 is not at the maximum, water is transferred from Node 2 to Node 1 and Node 1 drainage is recomputed:

$$
\begin{aligned}
&Wlevel_1 = Wlevel_1 + Drain_1 \\
&\Delta Wlevel_\max_2 = Wlevel_2 - \theta s_level_2 \\
&\Delta Wlevel_\max_1 = Ksoil_{frac} - \left(Wlevel_1 - \theta r \cdot Thick_1\right) \\
&\Delta Wlevel_1 = \min\left(\Delta Wlevel_\max_2 \frac{f_c}{1-f_c}, \Delta Wlevel_\max_1\right) \\
&Wlevel_1 = Wlevel_1 + \Delta Wlevel_1 \\
&\theta_1 = \frac{Wlevel_1}{Thick_1} \\
&K_u(\theta_1)_1 = Ktheta_fcn\left(Ksoil_{frac}, \theta r, \theta s, m, Wlevel_1, Thick_1\right) \\
&Drain_1 = \min\left(Wlevel_1 - \theta r \cdot Thick_1, K_u(\theta_1)_1\right) \geq 0 \\
&Wlevel_1 = Wlevel_1 - Drain_1 \\
&Wlevel_2 = Wlevel_2 - \Delta Wlevel_1 \frac{1-f_c}{f_c}
\end{aligned}
\tag{40}
$$

Then the drainage from Nodes 1 and 2 in Layer 1 is added to the water level in Layer 2 (Node 3):

$$
Wlevel_3 = Wlevel_3 + (1-f_c)Drain_1 + f_c \cdot Drain_2 \tag{41}
$$

Drainage from Layer 2 (Node 3) is calculated using the van Genuchten-Mualem equation. Any drainage from Layer 2 (Node 3) is added to Layer 3 (Node 4), as follows:

$$
\begin{aligned}
&\theta_3 = \frac{Wlevel_3}{Thick_3} \\
&K_u(\theta_3)_3 = Ktheta_fcn\left(Ksoil_{frac}, \theta r, \theta s, m, Wlevel_3, Thick_3\right) \\
&Drain_3 = \min\left(Wlevel_3 - \theta r \cdot Thick_3, K_u(\theta_3)_3\right) \\
&Wlevel_3 = Wlevel_3 - Drain_3 \\
&Wlevel_4 = Wlevel_4 + Drain_4
\end{aligned}
\tag{42}
$$

Drainage from Layer 3 (Node 4) is computed using a field capacity approach. Water in Layer 3 above field capacity is beyond the root zone and will eventually become net infiltration at rates limited by the soil or bedrock saturated hydraulic conductivity. The VGM model predictions of

recharge for this layer may be delayed by years without further discretization, and although the water will ultimately become net infiltration, the timing of recharge will likely be beyond the simulation time of the scenario. Therefore, a field capacity approach identical to the BalanceFC_fcn function is used here:

$$\begin{aligned} &Drain_4 = \min(Wlevel_4 - FC_4, Ksoil_{frac}, Krock_{frac}) \geq 0 \\ &Wlevel_4 = Wlevel_4 - Drain_4 \\ &Infil = Drain_4 \end{aligned} \tag{43}$$

The remainder of the BalanceVGM_fcn function is the same as the BalanceFC_fcn function. After net infiltration has been computed, any water in excess of the saturated water content in the layers is passed back up to the overlying layer. If water is in excess of the saturated water content in Layer 3 (Node 4), the excess water is added to Layer 2 (Node 3):

$$\begin{aligned} &Wlevel_3 = Wlevel_3 + \left(Wlevel_4 - \theta s_level_4\right) \\ &Wlevel_4 = \theta s_level_4 \end{aligned} \tag{44}$$

If water is in excess of the saturated water content in Layer 2 (Node 3), it is passed back up to Layer 1 and is proportioned between Nodes 1 and 2 based on the original drainage, as follows:

$$\begin{aligned} &\Delta Wlevel_3 = Wlevel_3 - \theta s_level_3 \\ &Wlevel_3 = \theta s_level_3 \\ &Wlevel_1 = Wlevel_1 + \frac{Drain_1}{\left[(1 - f_c)Drain_1 + f_c \cdot Drain_2\right]} \cdot \Delta Wlevel_3 \\ &Wlevel_2 = Wlevel_2 + \frac{Drain_2}{\left[(1 - f_c)Drain_1 + f_c \cdot Drain_2\right]} \cdot \Delta Wlevel_3 \end{aligned} \tag{45}$$

If water is greater than the saturated water content in Node 1 but is less than the saturated water content in Node 2, the excess water in Node 1 is transferred to Node 2 up to the capacity of Node 2 before computing runoff:

$$
\begin{aligned}
&\Delta Wlevel_\max_1 = Wlevel_1 - \theta s_level_1 \\
&\Delta Wlevel_2 = \min\left(\Delta Wlevel_\max_1 \frac{1-f_c}{f_c}, \theta s_level_2 - Wlevel_2\right) \\
&Wlevel_2 = Wlevel_2 + \Delta Wlevel_2 \\
&Wlevel_1 = Wlevel_1 - \Delta Wlevel_2 \frac{f_c}{1-f_c}
\end{aligned}
\tag{46}
$$

Similarly, if the water content in Node 2 is greater than the saturated water content but the water level in Node 1 is less than the saturated water content, the excess water in Node 2 is passed to Node 1 up to the saturated water content of Node 1 before computing runoff:

$$
\begin{aligned}
&\Delta Wlevel_\max_2 = Wlevel_2 - \theta s_level_2 \\
&\Delta Wlevel_1 = \min\left(\Delta Wlevel_\max_2 \frac{f_c}{1-f_c}, \theta s_level_1 - Wlevel_1\right) \\
&Wlevel_1 = Wlevel_1 + \Delta Wlevel_1 \\
&Wlevel_2 = Wlevel_2 - \Delta Wlevel_1 \frac{1-f_c}{f_c}
\end{aligned}
\tag{47}
$$

Water in excess of the saturated water content in Nodes 1 and 2 is transferred to runoff:

$$
\begin{aligned}
&Runoff = Wlevel_1 - \theta s_level_1 f_c \\
&Wlevel_1 = \theta s_level_1 \\
&Runoff = Runoff + \left(Wlevel_2 - \theta s_level_2 (1 - f_c)\right) \\
&Wlevel_2 = \theta s_level_2
\end{aligned}
\tag{48}
$$

The function returns the water levels in each of the nodes, runoff and net infiltration.

2.1.6.3 cdepth_fcn

This function calculates the depth of each layer in a cell based on the total thickness of the cell. If the total thickness of the cell is less than evaporation layer thickness (Ze, specified by user), the thickness of Layer 1 (Nodes 1 and 2) is set to the total thickness and the thicknesses of Layers 2 (Node 3) and 3 (Node 4) are set to zero. If the total thickness is greater than the

evaporation layer thickness but less than the rooting depth, the thickness of Layer 1 is set to the evaporation layer thickness, the thickness of Layer 2 is set to the difference between the total soil thickness and the evaporation layer thickness, and Layer 3 thickness is set to zero. If the soil thickness is greater than the rooting depth, Layer 1 is set to the evaporation layer thickness, Layer 2 is set to the rooting depth minus the evaporation layer thickness, and Layer 3 is set to the total thickness minus the rooting depth. This function is called once for each cell during the calculation of initial properties.

2.1.6.4 CellP_fcn

This function calculates the atmospheric pressure at a cell for a given elevation (Allen et al., 1998, Equation 7):

$$P = 101.3\left(\frac{293 - 0.0065z}{293}\right)^{5.26} \tag{49}$$

where P = the atmospheric pressure (kPa)

z = the elevation above sea level in meters

2.1.6.5 Dc_fcn

This function calculates the depletion depth of the evaporative layer covered by vegetation canopy (Layer 1, Node 2).

$$Dc = \min\left(FC_2 - Wlevel_2, TEW\right) \geq 0 \tag{50}$$

where Dc = the depletion depth (mm)

FC_2 = the field capacity in Node 2

$Wlevel_2$ = the water level in Node 2

TEW = the total evaporable water

Depletion depth is a measurement of how far the water level in the node is below field capacity. When the water level in the cell is at or above field capacity, the depletion depth is zero. When

the water level is at one-half the wilting point, the depletion depth is at a maximum equal to the total evaporable water (e.g., De_{max} = TEW = (FC – ½WP) in units of mm).

2.1.6.6 De_fcn

This function calculates the depletion depth of the bare soil fraction of the evaporative layer (Layer 1, Node 1):

$$De = \min\left(FC_1 - Wlevel_1, TEW\right) \geq 0 \tag{51}$$

where De = the depletion depth (mm)

FC_1 = the field capacity in Node 1

$Wlevel_1$ = the water level in Node 1

TEW = the total evaporable water

2.1.6.7 Dr_fcn

This function calculates the root zone depletion depth:

$$\begin{aligned} Dr &= \left[FC_1\left(1 - f_c\right) + FC_2 f_c + FC_3\right] - \left[Wlevel_1\left(1 - f_c\right) + Wlevel_2 f_c + Wlevel_3\right] \geq 0 \\ Dr &= \min\left(Dr, TAW\right) \end{aligned} \tag{52}$$

where Dr = the root zone depletion depth

FC = the field capacity for the specified node

f_c = the vegetation canopy cover fraction

Wlevel = the water level in the specified node

TAW = the total available water for transpiration in the root zone

Dr is always greater than zero and less than or equal to TAW.

2.1.6.8 e0

This function calculates the mean saturation vapor pressure as a function of air temperature (Allen et al., 1998, Equation 11):

$$e0 = 0.6108 \exp\left[\frac{17.27T}{T + 237.3}\right] \tag{53}$$

where e0 = the saturation vapor pressure at the air temperature T (kPa)

T = the air temperature (°C)

2.1.6.9 ET_fcn

The ET_fcn calculates the amount of transpiration and evaporation from the cell for the day. If there is no soil or evaporative layer thickness, the evaporation and transpiration are set to zero. Otherwise, the transpiration and evaporation are computed. The depletion depth is calculated using the De_fcn for the bare soil fraction of Layer 1 (Node 1):

$$De = De_fcn(FC_1, Wlevel_1, TEW) \tag{54}$$

where De = the depletion depth

FC_1 = the water level equivalent to field capacity in Node 1

$Wlevel_1$ = the water level in Node 1

TEW = the total evaporable water

Next, if the water level in Node 1 is greater than one-half the wilting point and there is no snow present, the evaporation from Node 1 is computed:

$$\begin{aligned} Kr &= Kr_fcn(De, REW, TEW) \\ Ke &= Ke_fcn(Kr, Kc_{\max}, Kcb, f_c) \\ Evaporation &= \min(Ke \cdot RefET, (Wlevel_1 - 0.5WP_1)(1 - f_c)) \end{aligned} \tag{55}$$

where Kr = the dimensionless evaporation reduction coefficient

REW = the readily evaporable water

Ke = the soil evaporation coefficient

Kc_{max} = the maximum basal transpiration coefficient

Kcb = the basal transpiration coefficient

f_c = the canopy cover coefficient

RefET = the potential reference evapotranspiration

WP1 = the water level equivalent to the wilting point in Node 1

The evaporation is subtracted from the water level in Node 1, as follows:

$$Wlevel_1 = Wlevel_1 - \frac{Evaporation}{1 - f_c} \tag{56}$$

Next, transpiration is computed for Layers 1 and 2 (Nodes 1 through 3). The maximum transpiration possible from each node is calculated as follows:

$$\begin{aligned} Transpire_\max_1 &= Wlevel_1 - WP_1 \geq 0 \\ Transpire_\max_2 &= Wlevel_2 - WP_2 \geq 0 \\ Transpire_\max_3 &= Wlevel_3 - WP_3 \geq 0 \end{aligned} \tag{57}$$

where Transpire_max$_1$ through Transpire_max$_3$ = the maximum transpirations from Nodes 1 through 3

Wlevel$_1$ through Wlevel$_3$ = the water levels in Nodes 1 through 3

WP$_1$ through WP$_3$ = the water level equivalents for the wilting point in Nodes 1 through 3.

The total maximum transpiration (Transpire_max) from the model cell is:

$$Transpire_\max = Transpire_\max_1 (1 - f_c) + Transpire_\max_2 \cdot f_c + Transpire_\max_3 \tag{58}$$

Transpiration is not limited to the canopy covered fraction and occurs in Layers 1 and 2 (Nodes 1 through 3) over the entire area of the cell. If Transpire_max is greater than zero, the actual total transpiration is computed. First the depletion depth for the root zone is calculated:

$$Dr = Dr_fcn(FC_{1-3}, Wlevel_{1-3}, TAW, f_c) \tag{59}$$

Next, the unadjusted evapotranspiration is computed as follows (Allen et al., 1998, Equation 69):

$$ET_c = \left(K_e + Kcb\right) RefET \tag{60}$$

The water stress coefficient is as follows:

$$Ks = Ks_fcn\left(Dr, TAW, ET_c, p\right) \tag{61}$$

where Ks = the transpiration reduction coefficient due to water stress

p = the fraction of the total available water (TAW) for transpiration that is readily available

Actual total transpiration is as follows:

$$Transpiration = \min\left(Ks \cdot Kcb \cdot RefET, Transpire_\max\right) \tag{62}$$

The transpiration is then proportioned between the nodes using the extension to FAO-56 described by Allen et al. (2005a). If the water level in Node 1 is above the wilting point, Node 1 transpiration coefficient (Ktp) is:

$$Ktp = Ktp_fcn\left(De, Dr, TEW, TAW, Thick\right) \tag{63}$$

where Thick = the thickness of each node

The transpiration from Node 1 is:

$$Transpiration_1 = \min\left(Ktp \cdot Transpiration, Transpire_\max_1\right) \tag{64}$$

Similarly, the transpiration from the fraction of Layer 1 covered by the vegetation canopy (Node 2) is calculated as follows:

$$
\begin{aligned}
&Dc = Dc_fcn(FC_2, Wlevel_2, TEW) \\
&Ktpc = Ktpc_fcn(Dc, Dr, TEW, TAW, Thick) \\
&Transpiration_2 = \min(Ktpc \cdot Transpiration, Transpire_max_2)
\end{aligned}
\tag{65}
$$

The transpiration from the root zone (Layer 2, Node 3) is:

$$
Transpiration_3 = \min(Transpiration - Transpiration_1(1 - f_c) - Transpiration_2 f_c, Transpire_max_3)
\tag{66}
$$

Next, the total transpiration is recalculated:

$$
Transpiration = Transpiration_1(1 - f_c) + Transpiration_2 f_c + Transpiration_3 \tag{67}
$$

The water levels are adjusted for transpiration, as follows:

$$
\begin{aligned}
Wlevel_1 &= Wlevel_1 - Transpiration_1 \\
Wlevel_2 &= Wlevel_2 - Transpiration_2 \\
Wlevel_3 &= Wlevel_3 - Transpiration_3
\end{aligned}
\tag{68}
$$

The function returns the updated water levels, total transpiration, and total evaporation.

2.1.6.10 Fr_fcn

This function calculates the stomatal resistance correction factor, as follows (Allen et al., 1998, Equation 102):

$$
Fr = \frac{\Delta + \gamma(1 + 0.34u_2)}{\Delta + \gamma\left(1 + 0.34u_2 \frac{r_l}{100}\right)} \tag{69}
$$

where Fr = the resistance correction factor

Δ = the slope of the saturation vapor pressure temperature relationship (kPa/°C)

u_2 = the mean daily wind speed at 2 meters above ground (m/s)

γ = the psychrometric constant (kPa/°C)

r_l = the mean leaf resistance (s/m)

The mean leaf resistance for the ET_0 reference grass and many agricultural crops is 100 s/m (Allen et al., 1998).

2.1.6.11 GroupBalance

The *GroupBalance* function calls the appropriate balance model specified by the user (e.g., field capacity balance model or VGM balance model) for the fraction of the day where precipitation, if any, occurs and for the non-precipitation fraction of the day. The function returns the total net infiltration and runoff for the cell for the day and updates the water levels in each node based on the balance model results.

2.1.6.12 Kcb_fcn

This function calculates the basal transpiration coefficient (Kcb), the maximum basal transpiration coefficient (Kc_{max}), and the canopy cover fraction. The first step is to estimate the minimum relative humidity from the vapor pressure, as follows (Allen et al., 1998, Equation 10):

$$RH_{\min} = \frac{e0(Tdew)}{e0(T\max)} \cdot 100 \tag{70}$$

where RH_{min} = the daily minimum relative humidity

e0 = the function described above

Tdew = the daily dewpoint temperature (°C)

Tmax = the maximum daily temperature (°C)

Next, the adjustment for the crop coefficient based on wind speed, minimum relative humidity, and plant height is estimated (e.g., Allen et al., 1998, Equations 62 and 100):

$$adjust = \left[0.04(u_2 - 2) - 0.004 \cdot (RH_{\min} - 45)\right]\left(\frac{h_{plant}}{3}\right)^{1/3} \tag{71}$$

where adjust = the transpiration coefficient adjustment

u_2 = the mean daily wind speed (m/s)

RH_{min} = the daily minimum relative humidity calculated above

h_{plant} = the plant height (m)

If there is no vegetation present on the cell (e.g., bare rock), the Kcb value is set to Kc_{min}, Kc_{max} is set to 1.2 + adjust, and the canopy cover fraction (f_c) is set to a minimum of 0.0001. If vegetation is present, the function *KcbStage_fcn* is called to obtain the Kcb for the day of the year. The Kcb is then adjusted using the adjustment factor calculated above, as follows:

$$Kcb = Kcb_{stage} + adjust \tag{72}$$

If the adjusted Kcb is less than the initial transpiration coefficient (Kcb_{ini}), the Kcb is set to Kcb_{ini}. The maximum Kc (Kc_{max}) is equal to 1.2 + adjust or Kcb + 0.05, whichever is greater (Allen et al., 1998, Equation 75). The canopy cover fraction is calculated according to Allen et al. (1998) (Equation 76), as follows:

$$f_c = \max\left[\left(\frac{Kcb - Kc_{\min}}{Kc_{\max} - Kc_{\min}}\right)^{1+0.5h_{plant}}, 0.0001\right] \tag{73}$$

where f_c = the canopy cover fraction

Kcb = the basal transpiration coefficient calculated above

Kc_{min} = the minimum transpiration coefficient for bare soil specified by the user

Kc_{max} = the maximum transpiration coefficient calculated above

h_{plant} = the plant height (m)

2.1.6.13 KcbFull_fcn

This function estimates the transpiration coefficient for natural vegetation with full ground cover during the peak of the growing season (Kcb_{full}). The first step is to estimate Kcb for full cover vegetation under sub-humid and calm wind conditions (Kcb_h), as follows (Allen et al., 1998):

$$Kcb_h = 1.0 + 0.1h \leq 1.20 \tag{74}$$

where h = the plant height

For vegetation greater than 2 meters in height, Kcb_h is limited to a value of 1.20 (Allen et al., 1998). Kcb_{full} is then estimated for the site climate conditions using Allen et al. (1998), Equation 100, as follows:

$$Kcb_{full} = Kcb_h + \left[0.04(u_2 - 2) - 0.004(RH_{\min} - 45)\right]\left(\frac{h}{3}\right)^{0.3} \tag{75}$$

where u_2 = the daily mean wind speed (m/s)

RH_{min} = the daily minimum relative humidity

h = the plant height

2.1.6.14 KcbLAI_fcn

The basal transpiration coefficient is estimated from LAI as follows (Allen et al., 1998, Equation 97):

$$Kcb = Kc_{\min} + \left(Kcb_{full} - Kc_{\min}\right)\cdot\left(1 - \exp\left(-0.7 \cdot LAI\right)\right) \tag{76}$$

where Kc_{min} = the minimum Kc for bare soil (user input)

Kcb_{full} = the basal Kcb for peak plant height and cover (calculated in *Kcbfull_fcn*)

Kcb = the basal transpiration coefficient

The function then adjusts Kcb using the stomatal resistance adjustment as follows (Allen et al., 1998, p. 191-193):

$$Kcb = Kcb \cdot Fr \tag{77}$$

where Fr = the stomatal resistance correction factor (Allen et al., 1998, Equation 102) calculated in the function *Fr_fcn*

2.1.6.15 KcbStage_fcn

This function estimates the Kcb for any day of the year based on the start of the growth stages specified by the user using a simple lookup algorithm. During periods of plant development and plant decline, the Kcb is linearly interpolated between Kcb_{ini} and Kcb_{mid} for the development stage or Kcb_{mid} and Kcb_{end} for the decline stage. Adjustments are made if the year is a leap year.

2.1.6.16 KdewOffset_fcn

This function calculates the dewpoint offset based on the day of the year. In arid climates, the dewpoint temperature is typically less than the daily minimum temperature. The DPWM allows for either a constant dewpoint offset from the daily minimum temperature or a harmonic fit to observed dewpoint offset from measurements of minimum relative humidity and temperature. The harmonic fit equation is as follows:

$$Ko = \left(A \cdot \sin\left(\frac{2\pi}{L} \left(DOY - V \right) \right) \right) + C \tag{78}$$

where A, L, V, and C = the fitted parameters of the harmonic function supplied by the user

DOY = the day of the year

If C is specified by the user as negative, the harmonic fit is not used and the absolute value of C is used as a constant offset.

2.1.6.17 Ke_fcn

This function calculates the reduction in evaporation as the soil dries in the evaporative layer (Allen et al., 1998, Equation 71):

$$Ke = Kr\left(Kc_{\max} - Kcb\right) \leq f_{ew} Kc_{\max} \tag{79}$$

where Ke = the soil evaporation coefficient

Kr = the dimensionless evaporation reduction coefficient (calculated outside of this function by *Kr_fcn*)

Kc_{max} = the maximum value of Kc following rain or irrigation

Kcb = the basal crop coefficient

f_{ew} = the fraction of the soil that is both exposed and wetted

2.1.6.18 Kr_fcn

This function calculates the dimensionless evaporation reduction coefficient for the evaporative layer. If all of the water that is available for evaporation (TEW) has been depleted, then Kr is equal to zero. If the soil water in the evaporative layer exceeds the amount of readily evaporable water (REW), then Kr is equal to 1. Otherwise, Kr ranges from 0 to 1 based on the following equation (Allen et al., 1998, Equation 74):

$$Kr = \frac{TEW - De}{TEW - REW} \tag{80}$$

where De = the cumulative depth of evaporation (depletion) in the evaporative layer

REW = the readily evaporable water equal to the difference between the field capacity and one-half the wilting point

2.1.6.19 Krel_fcn

This function calculates the relative permeability (K_{rel}) using the van Genuchten–Mualem equation. If the water content is less than the residual water content, K_{rel} is set to zero. Otherwise the relative permeability is calculated as follows (Selker et al., 1999):

$$K_{rel} = \sqrt{\frac{\theta - \theta r}{\theta s - \theta r}}\left[1 - \left(1 - \left(\frac{\theta - \theta r}{\theta s - \theta r}\right)^{\frac{1}{m}}\right)^{m}\right]^{2} \tag{81}$$

where θ = the water content of the node (L^3/L^3)

θr = the residual water content (L^3/L^3)

θs = the saturated water content (L^3/L^3)

m = the dimensionless van Genuchten exponent

2.1.6.20 Ks_fcn

This function calculates the reduction in transpiration due to the depletion in water content in the root zone. The transpiration reduction coefficient (Ks) is calculated as follows (Allen et al., 1998, Equation 84):

$$Ks = \frac{TAW - Dr}{TAW - RAW} \tag{82}$$

where TAW = the total available water for transpiration

Dr = the root zone depletion

RAW = the readily available water for transpiration

RAW is computed from TAW as follows (Allen et al., 1998, Equation 83):

$$RAW = p \cdot TAW \tag{83}$$

where p = the average fraction of TAW that can be depleted from the root zone before moisture stress reduces ET

The value of p depends on the plant and the climate and ranges from 0.30 for shallow rooted plants under high ET to 0.70 for deep rooted plants with low ET. The DPWM adjusts the user-supplied value of p depending on ET, as follows (Allen et al., 1998):

$$p_{adj} = p + 0.04(5 - ET_c) \tag{84}$$

where p = the user-supplied value

ET_c = the potential ET for the given plant

The value of p_{adj} is constrained to be between 0.1 and 0.8.

2.1.6.21 Ktheta_fcn

The *Ktheta_fcn* estimates the unsaturated hydraulic conductivity based on the relative permeability calculated by the function *Krel_fcn*. The unsaturated hydraulic conductivity as a function of water content [K(θ)] is estimated as follows:

$$K(\theta) = Ksat \cdot Krel_fcn \tag{85}$$

where Ksat = the saturated hydraulic conductivity

If the water level is greater than the thickness of the layer, K(θ) is set to Ks. Assuming a unit gradient, the rate of drainage from the layer is equivalent to the unsaturated hydraulic conductivity. This function is used in the VGM balance model.

2.1.6.22 Ktp_fcn

This function implements one of the extensions to the FAO-56 method described by Allen et al. (2005a) where transpiration is proportioned between the evaporative layer and root layer depending on the water contents of each layer and the rooting depth of the vegetation. The function Ktp_fcn is for bare soil node of Layer 1 (Node 1). The proportion of basal transpiration extracted from the evaporative layer is as follows (Allen et al., 2005a, Equation 29):

$$K_{tp} = \left(\frac{1 - \frac{De}{TEW}}{1 - \frac{Dr}{TAW}} \right) \left(\frac{Ze}{Zr} \right)^{0.6} \tag{86}$$

where De = the cumulative depletion in Node 1 (bare soil fraction of evaporative layer)

Dr = the cumulative depletion in Node 3 (root layer)

TEW = the total evaporable water

TAW = the total available water

Ze = the evaporation layer depth

Zr = the rooting depth

2.1.6.23 Ktpc_fcn

This function implements the extension to the FAO-56 method where transpiration is proportioned between the evaporative layer and root layer. This function is virtually the same as Ktp_fcn except that Ktpc_fcn is for the fraction of the evaporative layer that is covered by the plant canopy (Node 2). The proportion of basal transpiration extracted from the evaporative layer is calculated as follows (Allen et al., 2005a, Equation 29):

$$K_{tp} = \left(\frac{1 - \frac{Dc}{TEW}}{1 - \frac{Dr}{TAW}} \right) \left(\frac{Ze}{Zr} \right)^{0.6} \tag{87}$$

where Dc = the cumulative depletion in Node 2 (canopy covered fraction of evaporative layer)

Dr = the cumulative depletion in Node 3 (root layer)

TEW = the total evaporable water

TAW = the total available water

Ze = the evaporation layer depth

Zr = the rooting depth

2.1.6.24 Precip_elev_cor_fcn

This function estimates the precipitation at a cell for a given elevation based on the reference precipitation value for the day supplied by the climate file. The correction to precipitation for elevation is based on the slope of the correlation between precipitation and elevation supplied by the user. The elevation correction for precipitation is as follows:

$$P_{cell} = P_{ref} \left(1 + \left(elev_{cell} - elev_{ref} \right) Cprecip \right) \tag{88}$$

where P_{cell} = the daily precipitation at the cell (mm)

P_{ref} = the reference precipitation supplied in the user file (mm)

$elev_{cell}$ = the elevation of the cell (m) supplied in the watershed file

$elev_{ref}$ = the elevation of the reference precipitation supplied in the parameter input file (m)

Cprecip = the correlation between precipitation and elevation (mm/m)

Although negative values for daily precipitation are not expected, this function is set to zero if the result is negative.

2.1.6.25 Psych_fcn

This function calculates the psychrometric constant, as follows (Allen et al., 1998, Equation 8):

$$\gamma = \frac{c_p P}{\varepsilon \lambda} \tag{89}$$

where γ = the psychrometric constant (kPa/°C)

c_p = the specific heat at constant pressure (1.013 x 10^{-3} MJ/(kg°C))

P = the atmospheric pressure

ε = the ratio of molecular weight of water vapor to dry air (0.622)

λ = the latent heat of vaporization (2.45 MJ/kg)

2.1.6.26 RefET_fcn

This function calculates the reference evapotranspiration adjusted for the slope and azimuth of the cell. Values of latitude, slope, and aspect provided in units of degrees are converted to radians at the beginning of RefET_fcn. The procedure is the same as that described by Allen and Trezza (2006).

Step 1: The mean daily dewpoint temperature is set to the reference dewpoint temperature:

$$Tdew = Tdew_{ref} \tag{90}$$

Step 2: The general, actual vapor pressure (ea) is calculated for use in the Penman-Monteith equation and for estimating precipitable water (W) over the watershed, as follows (Allen et al., 1998, Equation 14):

$$e_{a_general} = 0.6108 \cdot \exp\left[\frac{17.27 \cdot Tdew}{Tdew + 237.3}\right] \tag{91}$$

$e_{a_general}$ is in units of kPa. It is assumed that the entire air mass over the watershed has this actual vapor pressure.

Step 3: The inverse square relative distance between the earth and the sun (d_r) is then calculated for use in the Ra calculation, as follows (Allen et al., 1998, Equation 23):

$$d_r = 1 + 0.033\cos\left(\frac{2\pi}{365} DOY\right) \tag{92}$$

where DOY = the calendar day of the year between January 1 and December 31

Step 4: The declination of the earth (δ) is then calculated as follows (Allen et al., 1998, Equation 24):

$$\delta = 0.409\sin\left[\frac{2\pi}{365} DOY - 1.39\right] \tag{93}$$

Step 5: The sunset hour angle (w_s) for a horizontal surface is then calculated as follows (Allen et al., 1998, Equation 25):

$$\omega_s = ar\cos\left[-\tan(Latitude)\tan(\delta)\right] \tag{94}$$

where latitude= the average latitude of the watershed

Step 6: Extraterrestrial radiation on a horizontal surface for a 24-hour period (Ra_hor) is calculated as follows (Allen et al., 1998, Equation 21):

$$R_{a_hor} = \frac{12(60)}{\pi} G_{sc} d_r \left[\omega_s \cdot \sin(Latitude)\sin(\delta) + \cos(Latitude)\cos(\delta)\sin(\omega_s)\right] \tag{95}$$

where G_{sc} = the solar constant (0.0820 MJ/(m^{2}min))

Latitude = the average latitude of the watershed

Step 7: The sine of mean solar elevation over a 24-hour period weighted by extraterrestrial radiation is calculated as follows (Allen et al., 2005b, Equation D-5):

$$\sin\beta_{24} = \sin\left[0.85 + 0.3Latitude \cdot \sin\left(\frac{2\pi}{365}DOY - 1.39\right) - 0.42(Latitude)^2\right] \geq 0.001 \qquad (96)$$

The value of $\sin\beta_{24}$ is limited to values greater than 0.001 for numerical stability in Step 10.

Step 8: The mean atmospheric pressure for the reference weather station is calculated using the elevation of the weather station, as follows (Allen et al., 1998, Equation 7):

$$P_{ref} = 101.3\left(\frac{293 - 0.0065Elev_{ref}}{293}\right)^{5.26} \qquad (97)$$

where $Elev_{ref}$ = the reference elevation of the weather station

Step 9: Precipitable water (W) at the reference location is calculated as follows (Allen et al., 2005b, Equation D-3):

$$W = 0.14 \cdot e_{a_general} \cdot P_{ref} + 2.1 \qquad (98)$$

where W = the precipitable water over the watershed

Step 10: The 24-hour transmissivity for beam radiation is calculated as follows (Allen et al., 2005b, Equation D-2):

$$K_{Bo_hor} = 0.98\exp\left[\frac{-0.00146P_{ref}}{K_{c\ln} \cdot \sin\beta_{24}} - 0.075\left(\frac{W}{\sin\beta_{24}}\right)^{0.4}\right] \qquad (99)$$

where P_{ref} = the atmospheric pressure at the reference location (kPa)

W = the precipitable water in the atmosphere (mm)

Kcln = the atmospheric clearness (turbidity) coefficient

K_{cln} ranges from less than 0.5 for extremely turbid, dusty or polluted air to 1.0 for clean air. The air in Big Bear Valley is expected to be clear; therefore, the value of K_{cln} is set as 1.0 in the code.

Step 11: The 24-hour transmissivity for diffuse radiation is calculated as follows (Allen et al., 2005b, Equation D-4):

$$\begin{aligned} K_{Do_hor} &= 0.35 - 0.36 K_{Bo_hor} \quad \textit{for } K_{Bo_hor} \geq 0.15 \\ K_{Do_hor} &= 0.18 - 0.82 K_{Bo_hor} \quad \textit{for } K_{Bo_hor} < 0.15 \end{aligned} \tag{100}$$

Step 12: Clear sky solar radiation over the 24-hour period is calculated as follows (Allen et al., 2005b, Equation D-1):

$$R_{so_hor} = \left(K_{Bo_hor} + K_{Do_hor}\right) R_{a_hor} \tag{101}$$

Step 13: "Measured" solar radiation on a horizontal surface is estimated using Hargreave's method, as follows (Allen et al., 1998, Equation 50):

$$R_{sm_hor} = k_{rs} \sqrt{T\max_{ref} - T\min_{ref}}\, R_{a_hor} \leq R_{so_hor} \tag{102}$$

where R_{sm_hor} = the estimated "measured" solar radiation (MJ/(m^{2}d))

k_{rs} = the adjustment coefficient (typically 0.16 to 0.19)

$Tmax_{ref}$ = the maximum daily temperature at the reference location (°C)

$Tmin_{ref}$ = the minimum daily temperature at the reference location (°C)

Step 14: The total short-wave transmissivity (also known as clearness index) associated with the "measured" R_s value is calculated as follows (Duffie and Beckman, 1980, Equation 2.9.2):

$$\tau_{sw_hor} = \frac{R_{sm_hor}}{R_{a_hor}} \tag{103}$$

Step 15: The atmospheric transmissivity in Step 14 is partitioned into its diffusive and direct beam components. The procedure as described by Trezza and Allen (2006) is adapted from Duffie and Beckman (1980, 1991), who cite Orgill and Hollands (1977). Allen and Trezza (2006) rearranged the equations and made minor modifications to match measured transmissivity data at Yucca Mountain.

$$\begin{aligned} K_{D_hor} &= 0.12\tau_{sw_hor} \quad for\ \tau_{sw_hor} \geq 0.78 \\ K_{D_hor} &= 1.557\tau_{sw_hor} - 1.84\left(\tau_{sw_hor}\right)^2 \quad for\ 0.35 < \tau_{sw_hor} < 0.78 \\ K_{D_hor} &= \tau_{sw_hor} - 0.249\left(\tau_{sw_hor}\right)^2 \quad for\ \tau_{sw_hor} \leq 0.35 \end{aligned} \tag{104}$$

Step 16: The actual direct beam transmissivity is calculated as the difference between total transmissivity and diffuse transmissivity, as follows (Allen, 1996, Equation 7):

$$K_{B_hor} = \tau_{sw_hor} - K_{D_hor} \tag{105}$$

Step 17: The direct beam radiation on the horizontal surface is calculated based on the measured R_{sm_hor}, as follows:

$$I_{b_hor} = K_{B_hor} \cdot R_{a_hor} \tag{106}$$

Step 18: The diffuse component of measured R_{sm_hor} for a horizontal surface is calculated as follows:

$$I_{d_hor} = K_{D_hor} \cdot R_{a_hor} \tag{107}$$

Step 19: The albedo (α_T) is the value specified by the user or estimated for snow cover:

$$\alpha_T = albedo \tag{108}$$

Step 20: The ratio of beam radiation R_b on an incline to the beam radiation on a horizontal plane is calculated. Allen and Trezza (2006) suggest making a lookup table for many slope-aspect-day of year combinations, but the DPWM calculates the ratio exactly for the given slope-aspect and day of year combination.

Step 20a: The effective latitude for a given slope and aspect is calculated as described by Revfeim (1976) (Equation 2):

$$\varphi_{eff} = \arcsin\left[\cos(s)\sin(\varphi) + \sin(s)\cos(\gamma + \pi)\right] \tag{109}$$

where φ_{eff} = the effective latitude

s = the slope in radians

φ = the average latitude for the watershed in radians

γ = the surface aspect angle in radians

Step 20b: Check whether surface receives any direct beam radiation during the day. If the cell does not receive any direct beam radiation (i.e., during winter on extreme northerly slopes), Rb is zero and the remaining Step 20 sub-steps are skipped:

$$if\ \varphi_{eff} - \delta \geq \frac{\pi}{2} \quad then\ Rb = 0 \tag{110}$$

where δ = the declination from Step 4

Step 20c: Set up for the solution of daily integration limits for beam (direct) radiation using Duffie and Beckman (1991). Parameter A for the slope-aspect combination is calculated as follows:

$$A = \cos(s) + \tan(\varphi)\cos(\gamma)\sin(s) \tag{111}$$

where s = the slope in radians

φ = the latitude in radians

g = the surface aspect angle in radians

Step 20d: Parameter B for the slope-aspect combination and day of the year is calculated as follows:

$$B = \cos(\omega_s)\cos(s) + \tan(\delta)\sin(s)\cos(\gamma) \tag{112}$$

where w_s = the sunset hour angle from Step 5

s = the slope in radians

d = the solar declination from Step 4

g = the surface aspect angle in radians

Step 20e: Parameter C for the specified slope-aspect combination is calculated as follows:

$$C = \frac{\sin(s)\sin(\gamma)}{\cos(\varphi)} \tag{113}$$

Step 20f: The 24-hour integration limits on w_{sr} and w_{ss} for the Rb equation are calculated assuming that the sun appears only once during a 24-hour period:

$$|\omega_{sr}| = \min\left[\omega_s, ar\cos\left(\frac{A \cdot B + C\sqrt{A^2 - B^2 + C^2}}{A^2 + C^2}\right)\right]$$

$$\omega_{sr} = \begin{cases} -|\omega_{sr}| \; if \; (A > 0 \; and \; B > 0 \; or \; A \geq B) \\ |\omega_{sr}| \;\; otherwise \end{cases}$$

$$\left|\omega_{ss}\right| = \min\left[\omega_s, ar\cos\left(\frac{A\cdot B - C\sqrt{A^2 - B^2 + C^2}}{A^2 + C^2}\right)\right]$$
$$\omega_{sr} = \begin{cases}\left|\omega_{sr}\right| if\ \left(A > 0\, and\ B > 0\ or\ A \geq B\right) \\ -\left|\omega_{sr}\right|\ \ otherwise\end{cases} \tag{114}$$

The program checks that the square root term (A2 - B2 + C2) is positive and that the arcos terms are within the domain bounds of –1 to 1. If the square root term is negative, $w_{sr} = -w_s$ and $w_{ss} = w_s$. If one of the arcos terms is out of bounds, the respective integration limit is $w_{sr} = -w_s$ and/or $w_{ss} = w_s$. Another check is performed before calculating Rb to prevent negative values of Rb. Negative values for Rb may occur under conditions of very low sun angles during the day (e.g., winter) on north-facing slopes. Negative values of Rb are prevented by changing the signs for the integration limits:

$$\begin{aligned} &if\ \left(A < B\right)\ and\ \ \gamma > 0\ \ then\ \omega_{sr} = -\omega_{sr} \\ &f\ \left(A < B\right)\ and\ \ \gamma < 0\ \ then\ \omega_{ss} = -\omega_{ss} \end{aligned} \tag{115}$$

Step 20g: The beam adjustment ratio Rb is calculated as follows:

$$Rb = \frac{\begin{aligned}&\sin(\delta)\sin(\varphi)\cos(s)(\omega_{ss} - \omega_{sr}) \\ &-\sin(\delta)\cos(\varphi)\sin(s)\cos(\gamma)(\omega_{ss} - \omega_{sr}) \\ &+\cos(\delta)\cos(\varphi)\cos(s)(\sin(\omega_{ss}) - \sin(\omega_{sr})) \\ &+\cos(\delta)\sin(\varphi)\sin(s)\cos(\gamma)(\sin(\omega_{ss}) - \sin(\omega_{sr})) \\ &-\cos(\delta)\sin(s)\sin(\gamma)(\cos(\omega_{ss}) - \cos(\omega_{sr}))\end{aligned}}{2(\cos(\varphi)\cos(\delta)\sin(\omega_s) + \omega_s\sin(\varphi)\sin(\delta))} \tag{116}$$

Step 21: The direct beam on the inclined surface for a given slope-aspect combination is calculated using the Rb adjustment factor from Step 20:

$$I_b = I_{b_hor} \cdot Rb \tag{117}$$

where I_{b_hor} is from Step 17. I_b and I_{b_hor} have units of MJ/(m^{2}d).

Step 22: The anisotropic index is equivalent to the actual direct beam transmissivity (K_{B_hor}):

$$A_t = K_{B_hor} \tag{118}$$

where K_{B_hor} is from Step 16.

Step 23: The modulating function f is calculated as follows:

$$f = \sqrt{\frac{I_{b_hor}}{R_{sm_hor}}} \tag{119}$$

where I_{b_hor} is from Step 21 and R_{sm_hor} is from Step 13.

Step 24: The diffuse component for the inclined surface is calculated as follows:

$$I_d = I_{d_hor}\left[\left(1 - A_t\right)\left(\frac{1+\cos(s)}{2}\right)\left(1 + f \cdot \sin^3(s/2)\right) + A_t Rb\right] \tag{120}$$

Step 25: The reflected radiation component for the inclined surface is calculated as follows:

$$I_r = R_{sm_hor} \cdot \alpha_T \cdot \left(\frac{1-\cos(s)}{2}\right) \tag{121}$$

where α_T = the albedo of the terrain (Step 19)

s = the slope in radians

Step 26: The total radiation received by the inclined surface is calculated as follows:

$$R_{sm_inc} = I_b + I_d + I_r \tag{122}$$

where I_b = the beam radiation on the incline (Step 21)

I_d = the anisotropic diffuse radiation on the incline (Step 24)

I_r = reflected radiation from lower-lying terrain (Step 25)

Step 27: Reproject R_{sm_inc} to a horizontal projection (equivalent), as follows:

$$R_{s(equiv)_hor} = \frac{R_{sm_inc}}{\cos(s)} \tag{123}$$

Step 28: The mean saturation vapor pressure associated with the lapsed daily extreme temperature for the cell is calculated as follows:

$$e_s = \frac{e0(T\max_{cell}) + e0(T\min_{cell})}{2} \tag{124}$$

where $Tmax_{cell}$ = the maximum temperature for the cell (°C)

$Tmin_{cell}$ = the minimum temperature for the cell (°C)

e0 = the function described above

Step 29: The actual vapor pressure of the cell is limited to a value equal or greater than e_a from Step 2:

$$e_a = \min\left(e_{a_general}, e_s\right) \tag{125}$$

Step 30: The slope of the saturation vapor pressure curve is calculated as follows:

$$\Delta = slope_es_fcn(Tavg_{cell}) \tag{126}$$

Step 31: The atmospheric pressure at the cell is calculated as follows:

$$P_{cell} = CellP_fcn(Elev_{cell}) \tag{127}$$

Step 32: The psychrometric constant is calculated as follows:

$$\gamma_c = Psych_fcn(P_{cell}) \tag{128}$$

Step 33: The horizontal equivalent for net short wave radiation on the incline is calculated as follows (Allen et al., 1998, Equation 38):

$$R_{ns} = (1 - \alpha_T) \cdot R_{s(equiv)hor} \tag{129}$$

Step 34: The net outgoing radiation is calculated as follows (Allen et al., 1998, Equation 39):

$$R_{nl} = \sigma \left[\frac{T\max_{cell,K}{}^4 + T\min_{cell,K}{}^4}{2} \right] \left(0.34 - 0.14\sqrt{e_a}\right)\left(1.35\frac{R_{sm_hor}}{R_{so_hor}} - 0.35\right) \tag{130}$$

where R_{nl} = the net outgoing longwave radiation (MJ/(m^{2}d)) on a horizontal equivalent projection

s = the Stefan-Boltzmann constant at 4.903 x 10^{-9} MJ K^{-4} M^{-2} day^{-1}

$Tmax_{cell,K}$ = the maximum absolute temperature at the cell (°K)

$Tmin_{cell,K}$ = the minimum absolute temperature at the cell (°K)

e_a = the actual vapor pressure at the grid cell (kPa)

R_{sm_hor} = the measured or calculated solar radiation on a horizontal surface (MJ m^{-2} day^{-1})

R_{sm_hor} = the calculated clear-sky radiation on a horizontal surface (MJ m^{-2} day^{-1})

The ratio of R_{sm_hor}/R_{so_hor} is limited to values less than or equal to 1.

Step 35: Net radiation on the inclined surface projected to a horizontal projection is calculated as follows (Allen et al., 1998, Equation 40):

$$R_n = R_{ns} - R_{nl} \tag{131}$$

Step 36: The reference evapotranspiration (ET_0) is calculated as follows (Allen et al., 1998, Equation 6):

$$ET_0 = \frac{0.408\Delta(R_n - G) + \gamma_c \frac{900}{Tavg_{cell} + 273} u_2 (e_s - e_a)}{\Delta + \gamma_c (1 + 0.34 u_2)} \tag{132}$$

where ET_0 = the reference evapotranspiration (mm/d) for an inclined surface but expressed on a horizontal basis

R_n = the net radiation at the incline (but horizontal projection) (MJ m^{-2} day^{-1})

G = the soil heat flux density, which is zero for daily time steps

$Tavg_{cell}$ = the average temperature at the cell (°C)

u_2 = the wind speed at a 2-meter height provided from user input (m/s)

e_s = the saturation vapor pressure (kPa)

e_a = the actual vapor pressure (kPa)

$e_s - e_a$ = the saturation vapor pressure deficit (kPa)

Δ = the slope of the vapor pressure curve (kPa/°C)

γ_c = the psychrometric constant (kPa/°C)

2.1.6.27 RH_min_fcn

This function calculates the daily minimum relative humidity from the daily dewpoint and maximum temperatures, as follows (Allen et al., 1998, Equation 10):

$$RH_{\min} = \frac{e0(Tdew)}{e0(T\max)} \cdot 100 \tag{133}$$

where RH_{min} = the daily minimum relative humidity

e0 = the function described above

Tdew = the daily dewpoint temperature (°C)

Tmax = the maximum daily temperature (°C)

2.1.6.28 slope_es_fcn

This function estimates the slope of the saturation vapor pressure curve (Allen et al., 1998, Equation 13):

$$\Delta = \frac{4098\left[0.6108\exp\left(\frac{17.27T}{T+237.3}\right)\right]}{(T+237.3)^2} \tag{134}$$

where D = the slope of the saturation pressure curve (kPa/°C)

T = the air temperature (°C)

2.1.6.29 SnowB_fcn

This function calculates the snowpack level, snowmelt, and sublimation of new snow. If the average daily temperature is less than 0°C, precipitation occurs as snow and is added to the snowpack level. The total seasonal sublimation of the snowpack specified by the user in the input file is applied all at once at the time of precipitation. Snowmelt occurs if there is snowpack available and the daily average temperature is greater than 0°C. The snowmelt occurs at the rate specified by the user in the input file.

2.1.6.30 T_dew_fcn

This function calculates the daily dewpoint temperature from the daily minimum temperature:

$$Tdew = T\min - Ko \tag{135}$$

where Tdew = the daily dewpoint temperature

Tmin = the daily minimum temperature from the climate input file

Ko = the dewpoint offset calculated from KdewOffset_fcn

2.1.6.31 T_elev_cor_fcn

This function returns the minimum, maximum, average, and dewpoint temperatures for a cell. The minimum and maximum temperatures are estimated for the elevation of the cell from the reference minimum and maximum temperatures, as follows:

$$Tcell = Tref - \left(elev_{cell} - elev_{ref}\right) C_Tcor \tag{136}$$

where Tcell = the minimum or maximum temperature for the cell (°C)

Tref = the corresponding minimum or maximum reference temperature (°C)

$elev_{cell}$ = the elevation of the cell (m)

$elev_{ref}$ = the elevation of the reference temperature (m)

C_Tcor = the correlation between temperature and elevation

The average daily temperature for the cell is the average of the minimum and maximum temperatures estimated. The dewpoint temperature is calculated from the minimum daily temperature using the function *T_dew_fcn*.

2.1.6.32 Varying_f_c_fcn

This function maintains the water balance as the sizes of Nodes 1 and 2 change with changing canopy cover. Node 1 represents the bare soil area of the evaporative layer while Node 2 is the remaining area of the cell covered by vegetation canopy cover. As the canopy cover changes, the corresponding volumes of Nodes 1 and 2 change and water must be transferred to maintain the water balance.

If the canopy cover fraction (f_c) decreases, the water level in Node 1 increases, as follows:

$$Wlevel_1 = \frac{\left(1 - f_{c_old}\right) Wlevel_{1_old} + \left(f_{c_old} - f_c\right) Wlevel_{2_old}}{1 - f_c} \tag{137}$$

where $Wlevel_1$ = the new water level in Node 1 (mm)

f_{c_old} = the old canopy cover fraction

$Wlevel_{1_old}$ = the old water level in Node 1 (mm)

$Wlevel_{2_old}$ = the old water level in Node 2 (mm)

f_c = the new canopy cover fraction

The water level in Node 2 does not need to be changed when the canopy cover decreases to maintain the water mass balance in Layer 1.

If the canopy cover fraction increases, the water level in Node 2 increases, as follows:

$$Wlevel_2 = \frac{f_{c_old} \cdot Wlevel_{2_old} + \left(f_c - f_{c_old}\right) Wlevel_{1_old}}{f_c} \tag{138}$$

where $Wlevel_2$ = the new water level in Node 2 (mm)

f_{c_old} = the old canopy cover fraction

$Wlevel_{1_old}$ = the old water level in Node 1 (mm)

$Wlevel_{2_old}$ = the old water level in Node 2 (mm)

f_c = the new canopy cover fraction

The water level in Node 1 does not need to be changed when the canopy cover increases to maintain the water mass balance in Layer 1.

2.1.6.33 vg_head_to_wc

This function calculates the water content for a given capillary pressure using the van Genuchten equation:

$$\theta = \frac{\left(\theta s - \theta r\right)}{\left(1 + \left[\alpha \cdot h_c\right]^n\right)^m} + \theta r \tag{139}$$

where θ = the water content (L^3/L^3)

θs = the saturated water content (L^3/L^3)

θr = the residual water content (L^3/L^3)

α and n = the van Genuchten curve fitting parameters (1/L and unitless, respectively)

m = the van Genuchten curve parameter calculated as m = 1 – 1/n

h_c = the capillary pressure (L)

This function is used to estimate water contents for the field capacity and wilting point pressure points.

2.1.6.34 vg_wc_to_head

This function calculates the capillary pressure for a given water content based on the van Genuchten equation. This equation is not directly used to calculate water balance components in the DPWM but is provided to output average capillary pressures associated with water contents in the cell nodes:

$$h_c = \frac{1}{\alpha}\left[\left(\frac{\theta - \theta r}{\theta s - \theta r}\right)^{-\frac{1}{m}} - 1\right]^{\frac{1}{n}} \tag{140}$$

where hc = the capillary pressure

α and n = the van Genuchten curve fitting parameters

m = the van Genuchten curve parameter estimated from n as m = 1 – 1/n

θ = the water content

θr = the residual water content

θs = the saturated water content

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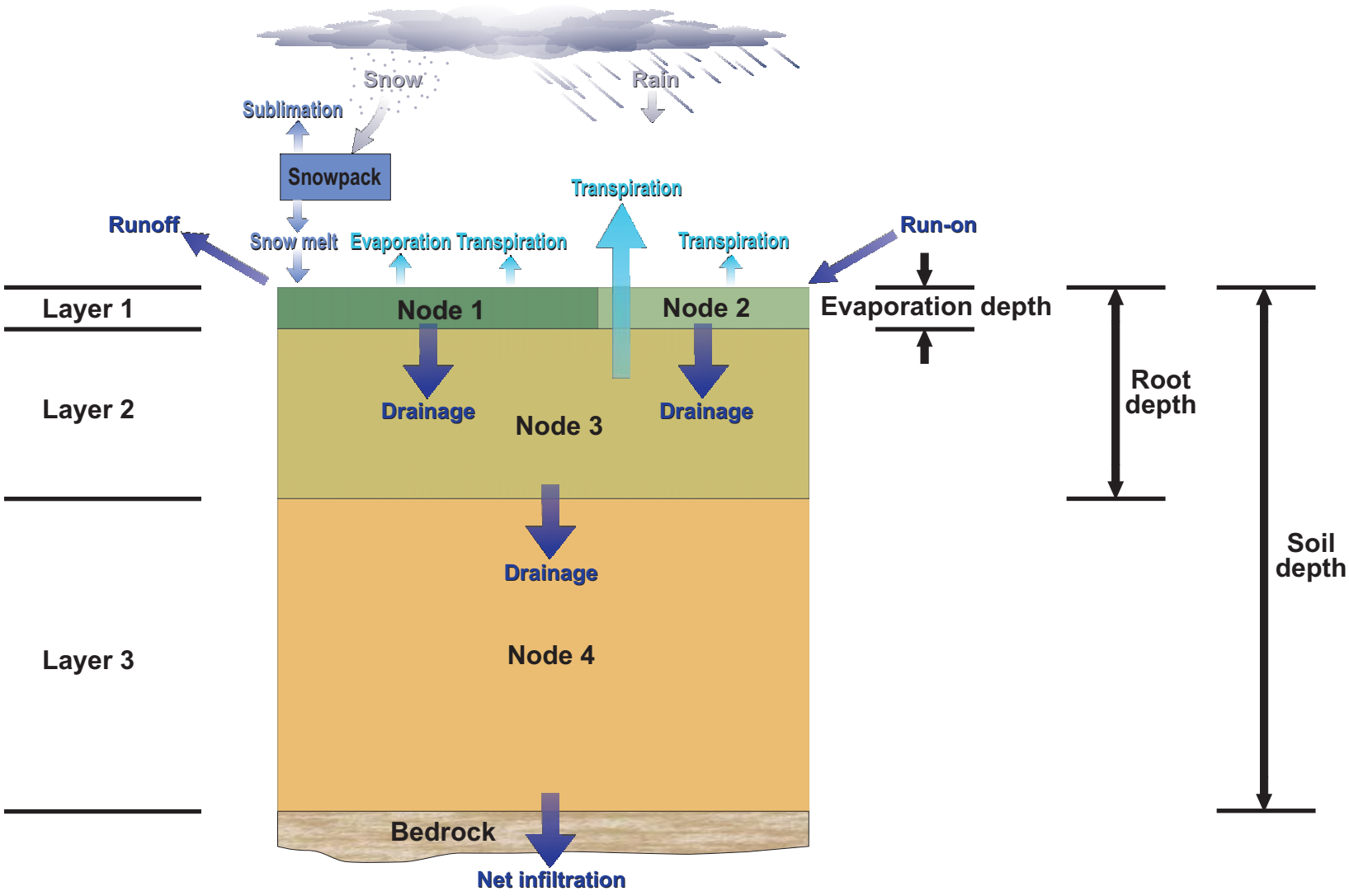
Figure

S:\PROJECTS\WR08.0064_BBCCSD\VR_DRAWINGS|WR08_0064_01W.CDR



Notes:

Not to scale

Node 1 = fraction exposed and wetted (f_{ew})

Node 2 = fraction covered by vegetation canopy (f_c)

BIG BEAR CITY - COMMUNITY SERVICES DISTRICT

Schematic of Water Balance Components and Computational Nodes Present in a Single Model Cell

Daniel B. Stephens & Associates, Inc.

6-9-09

JN WR08.0064

Figure 1

Table

Table 1. Description of Input File ipm Parameters
Page 1 of 2

Input File Row(s)	Parameters	Description	Units
1	CTcor	Temperature lapse rate (dry adiabatic lapse rate)	°C/m
2	Kdew_amp	Dewpoint offset - 1st order harmonic amplitude	---
3	Kdew_wave	Dewpoint offset - 1st order harmonic wavelength	---
4	Kdew_Xoff	Dewpoint offset - 1st order harmonic X offset	---
5	Kdew_Yoff	Dewpoint offset - 1st order harmonic Y offset	°C
6	elevref	Elevation of reference weather station	m
7	elevavg	Average elevation for entire basin	m
8	Latavg	Average latitude for entire basin	degree
9	CPrecipcor	Precipitation coefficient of elevation change	mm/m
10	Krs	Adjustment coefficient in Hargreaves' radiation formula	$°C^{-0.5}$
11	CN2	Not currently used in DPWM	---
12	Ze	Evaporation layer depth	m
13	rew	Readily evaporable water	mm
15–18	IC1 - IC4	Initial capillary pressure head in nodes 1 through 4 for all cells	cm
19	p	Depletion factor: average fraction of total available soil water (TAW) that can be depleted from the root zone before moisture stress (reduction in ET) occurs [0 – 1].	---
20	albedo	Terrain albedo	---
21	Csublime	Fraction of snowfall that eventually sublimates over entire season	---
22	Csnowmelt	Coefficients describing snow melting with temperature	mm/day/°C
23	fc_switch	Crop canopy coefficient (–1 indicates calculate internally)	---
24	Kc_min	Minimum transpiration coefficient (Kc) for dry surface soil (upper 0.10 to 0.15 m) with no vegetation cover	---
25	head_fc	Field capacity pressure head	cm
26	head_wp	Wilting point pressure head	cm
27–56	soilks	Soil saturated hydraulic conductivity	m/s
57–86	Reswc (θr)	Residual water content	---
87–116	nn (n)	van Genuchten curve fitting exponent n	---
117–146	Alpha (α)	van Genuchten curve fitting parameter α	1/cm
147–176	Porosity (θs)	Saturated water content	---

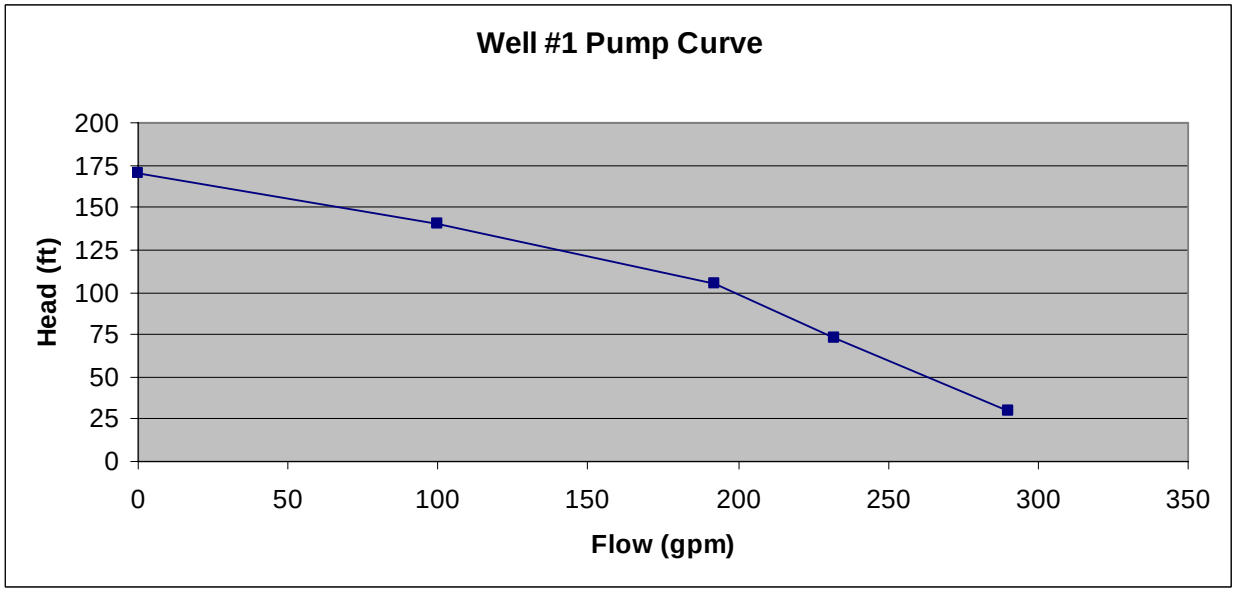
Table 1. Description of Input File ipm Parameters
Page 2 of 2

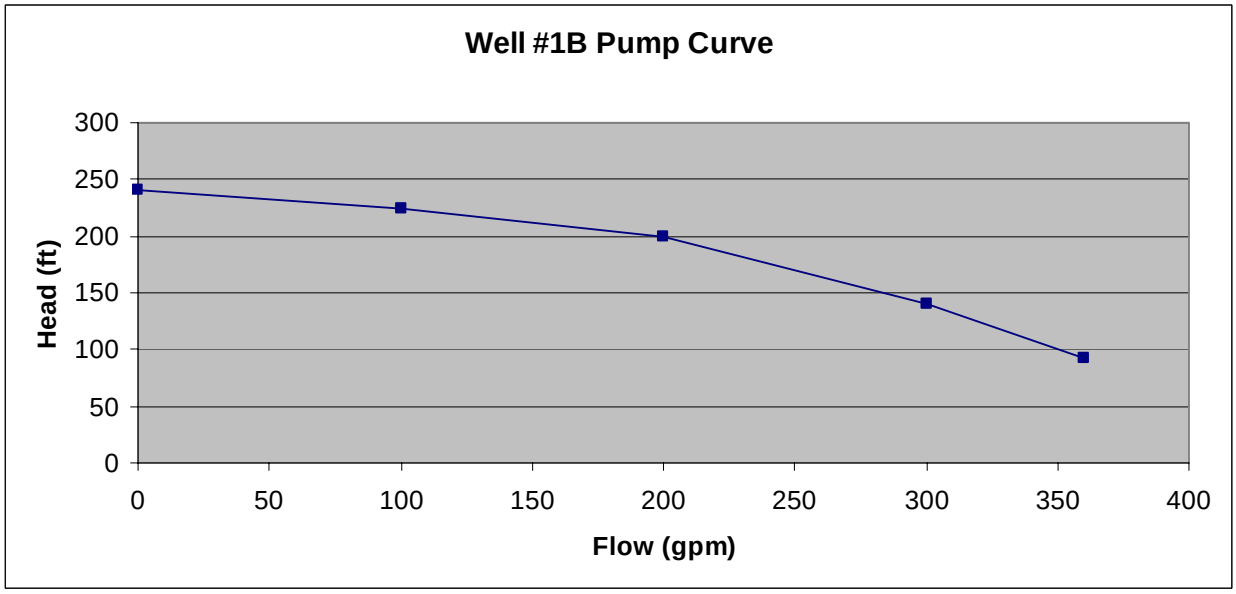
Input File Row(s)	Parameters	Description	Units
177–205	Sdepth	Soil depth	m
206–220	rockks	Bedrock saturated hydraulic conductivity	m/s
221–231	hplant	Mean maximum plant height	m
232–242	DS	Calendar day for start of vegetation development	day
243–253	MS	Calendar day for start of mid-season	day
254–264	ME	Calendar day for end of mid-season	day
265–275	LE	Calendar day for end of late seasons / start of dormancy	day
276–286	LAI	Mid-season leaf area index	---
287–297	LAIini	Initial or early season leaf area index	---
298–308	LAIlate	Late season / dormant leaf area index	---
309–319	rlmid	Stomatal resistance for mid-season	s/m
320–330	Zr	Mean maximum plant rooting depth	m
331–341	rlInit	Stomatal resistance for initial or early season	s/m
342–352	rlLate	Stomatal resistance for late or dormant season	s/m
353	P_adj	Adjustment to precipitation (1.0 = no adjustment; line not used in early versions of DPWM)	---
354	T_adj	Adjustment to temperature (1.0 = no adjustment; line not used in early versions of DPWM)	---
355	Cell_Output	Output data on cell by cell basis (1 = yes, 0 = no; line not used in early versions of DPWM)	---
356	Duration_slope	Slope of regression between total daily precipitation and duration of precipitation with intercept set at 1.0. If negative, duration from climate file is used. (line not used in early versions of DPWM)	hr/mm

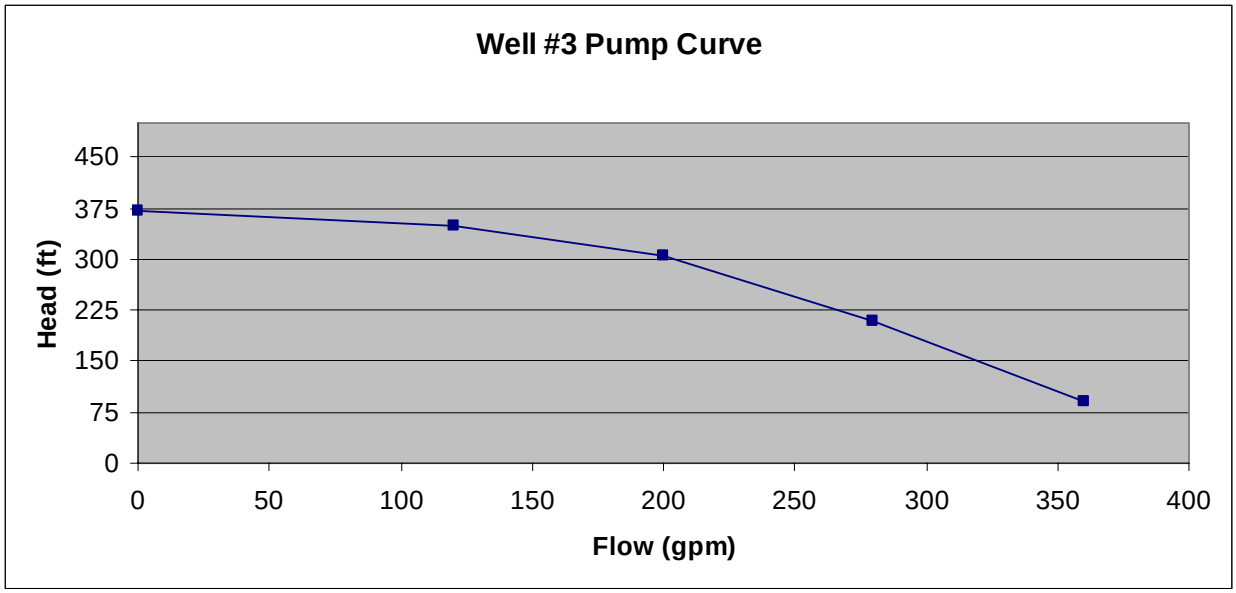
Appendix C

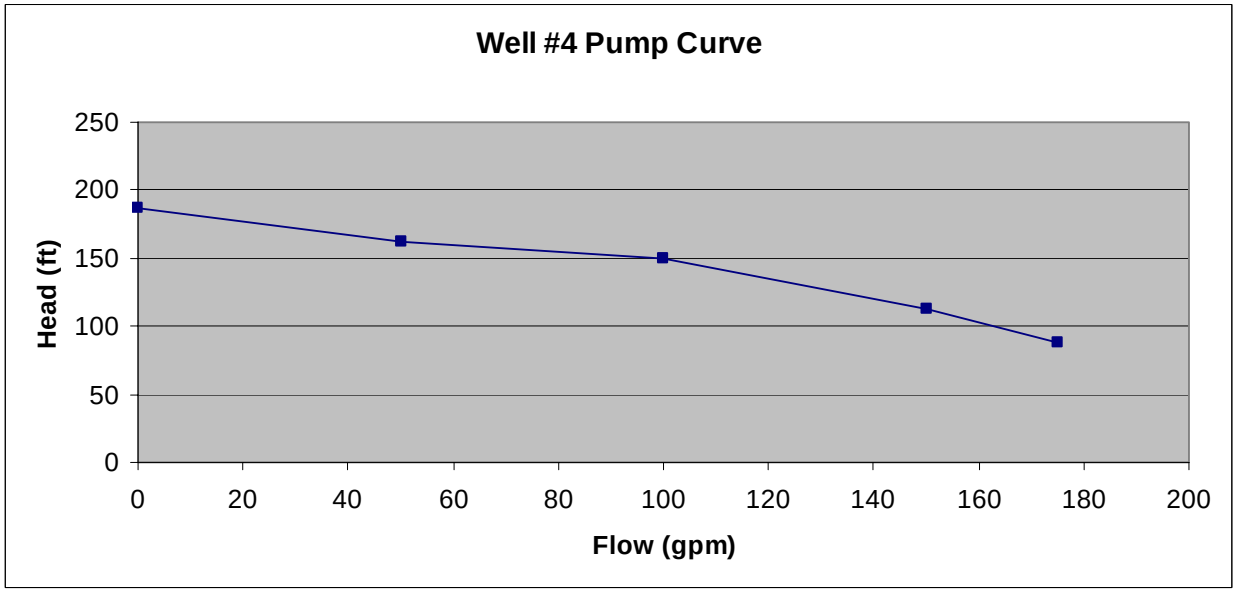
Well Pump Curves

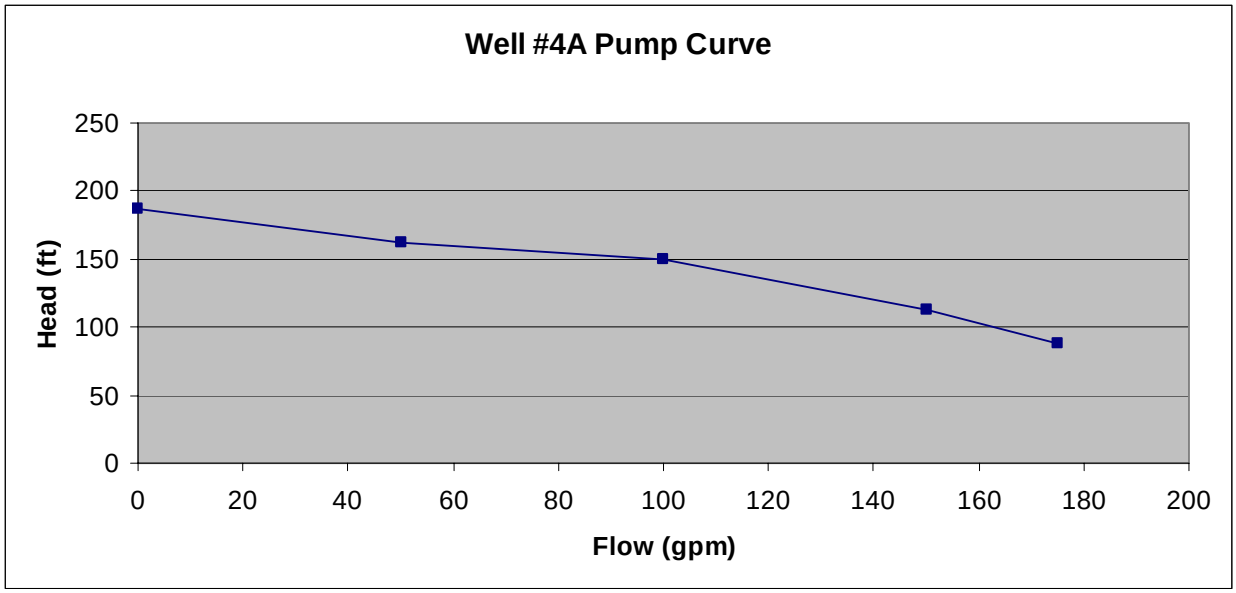
APPENDIX C – BBCCSD Well Pump Curves

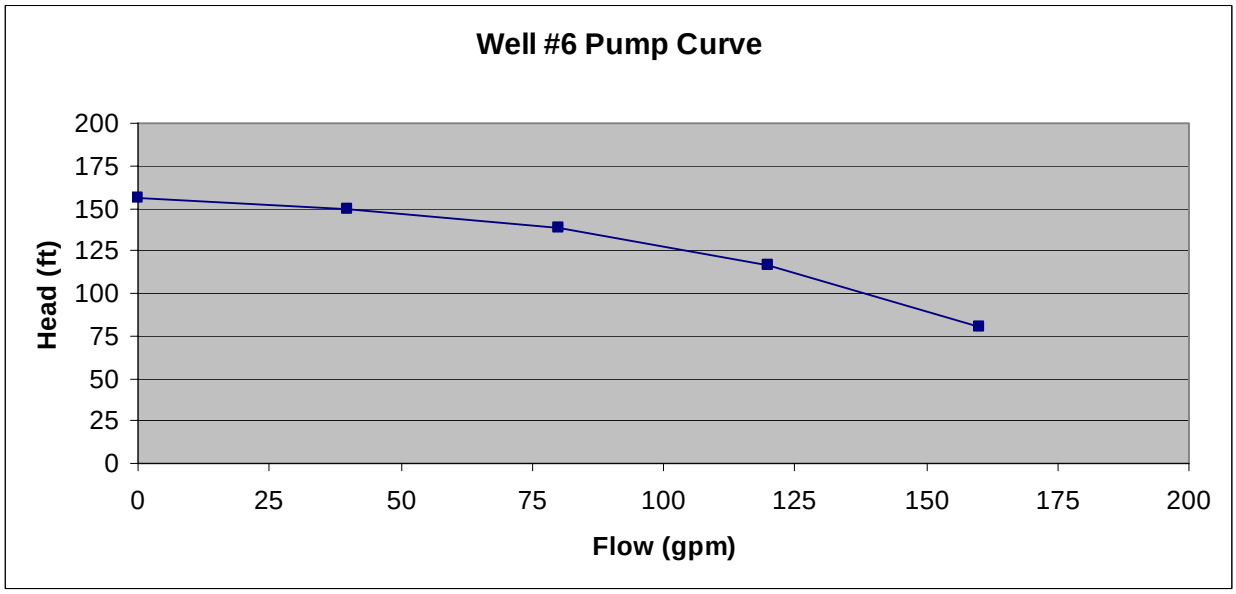


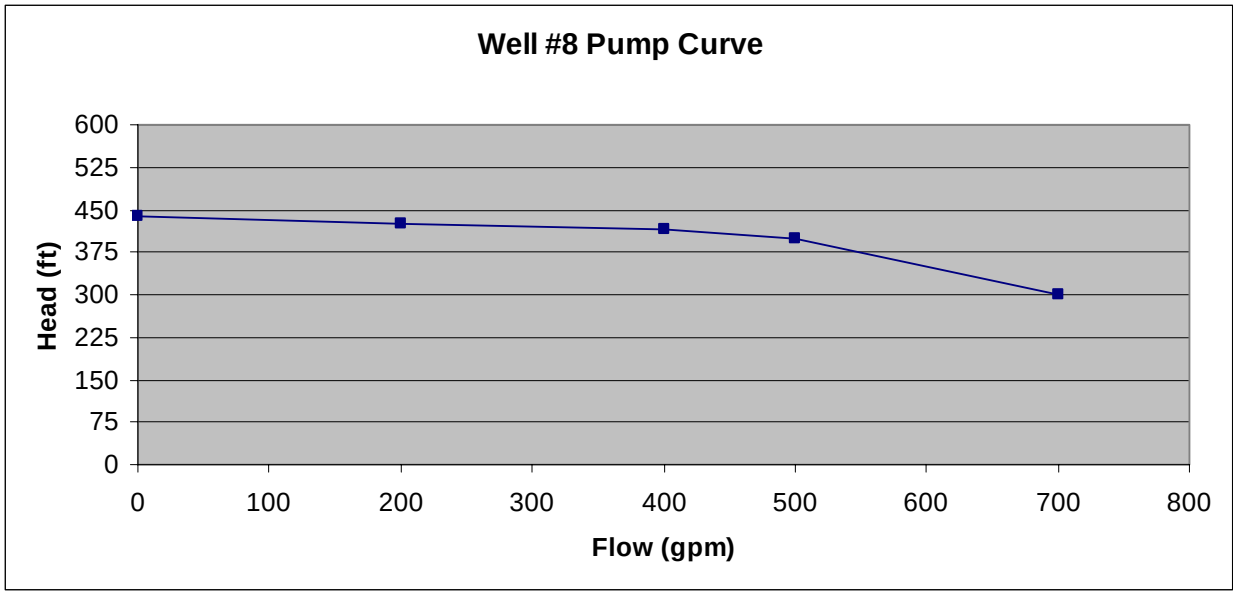


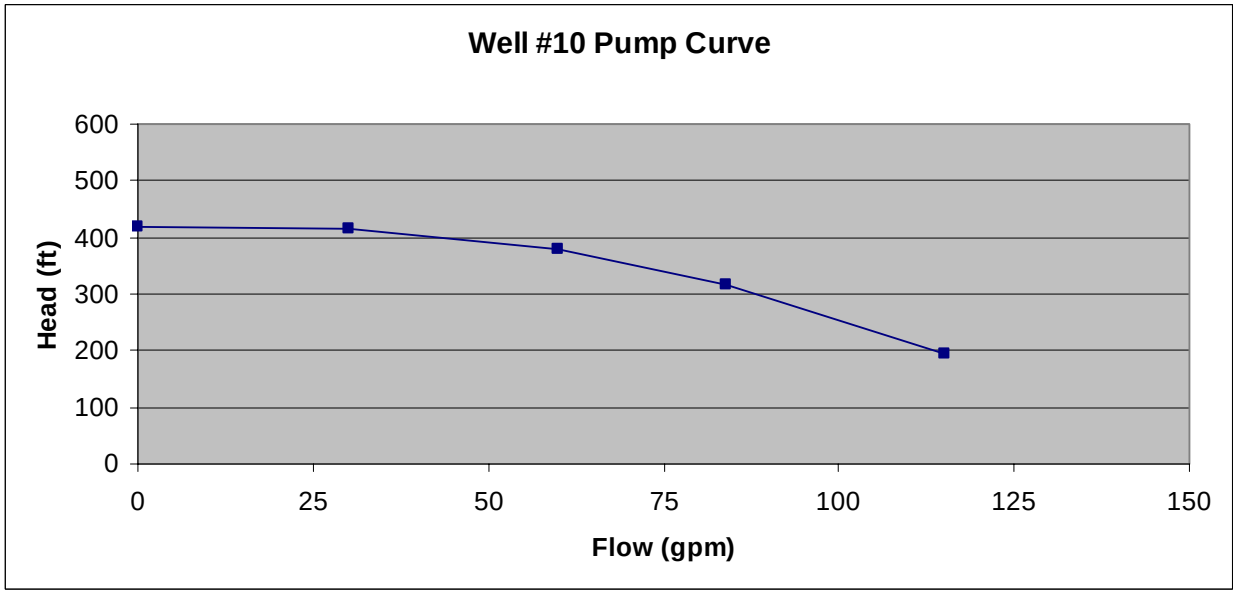

Well #3 Pump Curve
Head (ft)
450
375
300
225
150
75
0
0
50
100
150
200
250
300
350
400
Flow (gpm)


Well #4 Pump Curve
Head (ft)
250
200
150
100
50
0
0
20
40
60
80
100
120
140
160
180
200
Flow (gpm)


Well #4A Pump Curve
Head (ft)
250
200
150
100
50
0
0
20
40
60
80
100
120
140
160
180
200
Flow (gpm)


Well #6 Pump Curve
Head (ft)
200
175
150
125
100
75
50
25
0
0
25
50
75
100
125
150
175
200
Flow (gpm)


Well #8 Pump Curve
Head (ft)
600
525
450
375
300
225
150
75
0
0
100
200
300
400
500
600
700
800
Flow (gpm)


Well #10 Pump Curve
Head (ft)
600
500
400
300
200
100
0
0
25
50
75
100
125
150
Flow (gpm)

Appendix D

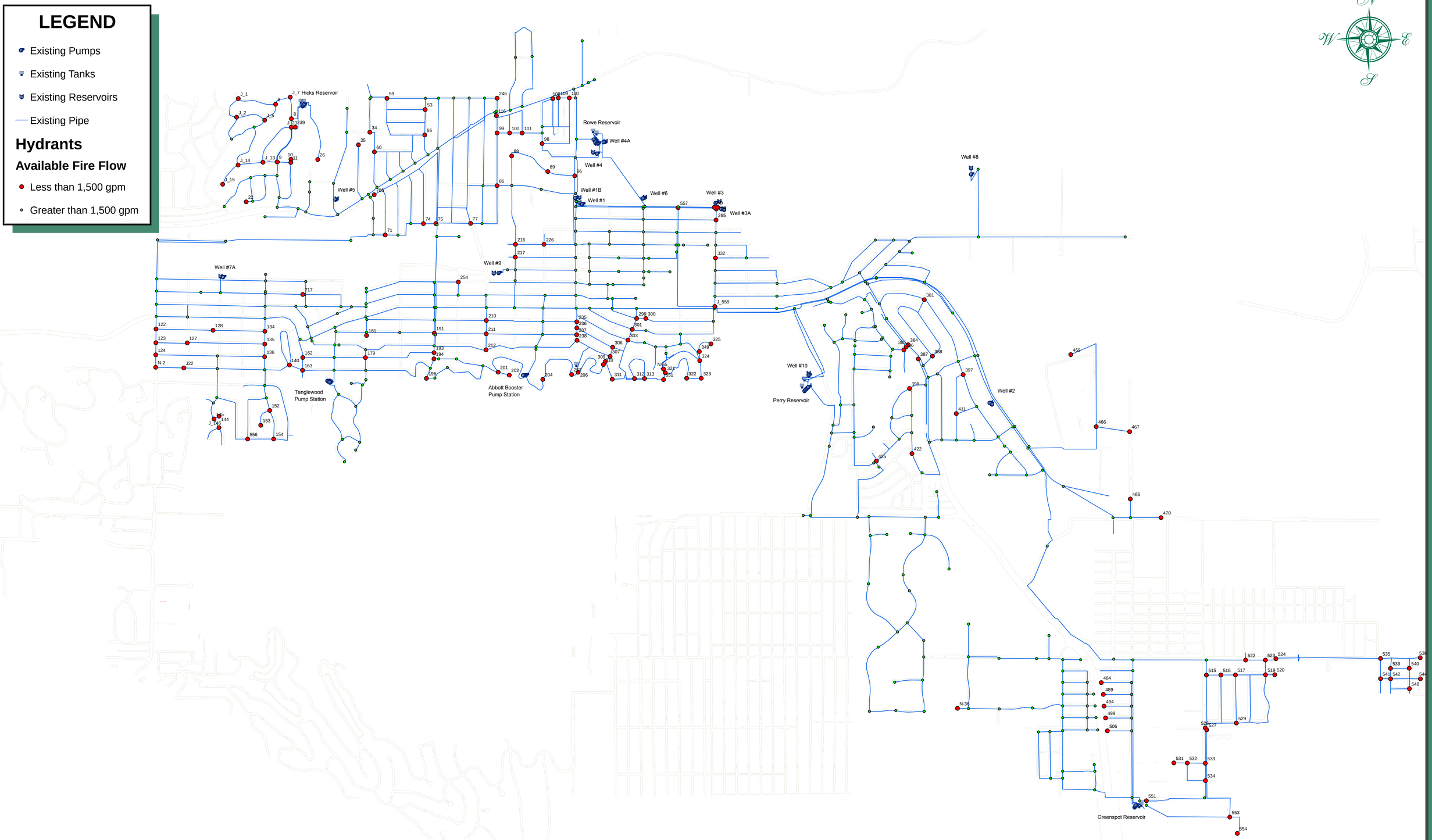
Hydraulic Model
Steady-State Calibration

APPENDIX D – DETAILED STEADY STATE CALIBRATION RESULTS

Hydrant Test ID	TEST NO.	MODEL JUNCTION ID	FLOW RATE (GPM)	FIELD STATIC PRESSURE (PSI)	MODEL STATIC PRESSURE (PSI)	STATIC VALUE DIFF.	STATIC % DIFF.	MODEL RESULTS	FIELD DYNAMIC PRESSURE (PSI)	MODEL DYNAMIC PRESSURE (PSI)	DYNAMIC VALUE DIFF.	DYNAMIC % DIFF.	MODEL RESULTS	MODEL PRESS DROP (PSI)	FIELD PRESS DROP (PSI)	DIFFERENCE IN DROP (PSI)	MODEL RESULTS
Q1	Test 1	J_2	1250	0	87.26				56	62.75	6.75	12.1		24.51	-56		
Q2	Test 1	J_14	0	0	93.76				0	77.16	77.16			16.6	0		
R1	Test 1	J_13	0	80	83.37	3.37	4.2	Great	65	68.92	3.92	6	Great	14.45	15	-0.55	Great
R2	Test 1	J_23	0	52	60.84	8.84	17	Good	38	47.69	9.69	25.5	Good	13.15	14	-0.85	Great
R3	Test 1	J_7	0	58	63	5	8.6	Great	41	47.63	6.63	16.2	Good	15.37	17	-1.63	Great
R4	Test 1	J_1	0	50	49.13	-0.87	1.7	Great	24	27.54	3.54	14.8	Great	21.59	26	-4.41	Great
R5	Test 1	J_5	0	82	86.83	4.83	5.9	Great	62	66.42	4.42	7.1	Great	20.41	20	0.41	Great
R6	Test 1	J_15	0	96	98.09	2.09	2.2	Great	88	81.49	-6.51	7.4	Good	16.6	8	8.6	Good
Q1	Test 2	J18	1360	0	84.55	84.55			66	72.48	6.48	9.8		12.07	-66		
Q2	Test 2	J_146	890	0	59.34	59.34			28	31.05	3.05	10.9		28.29	-28		
R1	Test 2	J_141	0	70	68.44	-1.56	2.2	Great	43	44.28	1.28	3	Great	24.16	27	-2.84	Great
R2	Test 2	J20	0	85	87.58	2.58	3	Great	71	76.64	5.64	7.9	Good	10.94	14	-3.06	Great
R3	Test 2	J_259	0	82	88.02	6.02	7.3	Good	78	76.25	-1.75	2.2	Great	11.77	4	7.77	Good
R4	Test 2	J_137	0	75	76.25	1.25	1.7	Great	53	56.98	3.98	7.5	Great	19.27	22	-2.73	Great
R5	Test 2	J22	0	80	87.08	7.08	8.9	Good	56	65.37	9.37	16.7	Good	21.71	24	-2.29	Great
R6	Test 2	J_121	0	88	87.15	-0.85	1	Great	73	75.27	2.27	3.1	Great	11.88	15	-3.12	Great
Q1	Test 3	J_182	1320	0	84.86	84.86			62	66.66	4.66	7.5		18.2	-62		
Q2	Test 3	J_180	1175	0	74.9	74.9			49	63.47	14.47	29.5		11.43	-49		
R1	Test 3	J_186	0	90	91.79	1.79	2	Great	84	80.96	-3.04	3.6	Great	10.83	6	4.83	Great
R2	Test 3	J24	0	82	86.16	4.16	5.1	Great	69	71.8	2.8	4.1	Great	14.36	13	1.36	Great
R3	Test 3	J_189	0	83	85.3	2.3	2.8	Great	78	75.7	-2.3	2.9	Great	9.6	5	4.6	Great
R4	Test 3	J_192	0	76	77.06	1.06	1.4	Great	65	66.71	1.71	2.6	Great	10.35	11	-0.65	Great
R5	Test 3	J_178	0	60	60.16	0.16	0.3	Great	47	48.98	1.98	4.2	Great	11.18	13	-1.82	Great
R6	Test 3	J_252	0	88	85.28	-2.72	3.1	Great	80	78.48	-1.52	1.9	Great	6.8	8	-1.2	Great
Q1	Test 4	J28	1425	0	87.45	87.45			72	72.3	0.3	0.4		15.15	-72		
Q2	Test 4	J26	1445	0	90.93	90.93			74	75.7	1.7	2.3		15.23	-74		
R1	Test 4	J_268	0	99	100.44	1.44	1.5	Great	92	87.55	-4.45	4.8	Great	12.89	7	5.89	Good
R2	Test 4	J_802	0	87	90.04	3.04	3.5	Great	79	77.16	-1.84	2.3	Great	12.88	8	4.88	Great
R3	Test 4	J_285	0	90	84.82	-5.18	5.8	Good	81	72.85	-8.15	10.1	Good	11.97	9	2.97	Great
R4	Test 4	J_286	0	96	97.02	1.02	1.1	Great	87	84.2	-2.8	3.2	Great	12.82	9	3.82	Great
R5	Test 4	J_231	0	90	90.91	0.91	1	Great	82	78.57	-3.43	4.2	Great	12.34	8	4.34	Great
R6	Test 4	J_559	0	93	91.57	-1.43	1.5	Great	92	90.1	-1.9	2.1	Great	1.47	1	0.47	Great
Q1	Test 5	N-29	980	0	38.42	38.42			34	35.26	1.26	3.7		3.16	-34		
Q2	Test 5	N-30	0	0	56.19	56.19			74	53.12	-20.88	28.2		3.07	-74		
R1	Test 5	N-18	0	80	74.82	-5.18	6.5	Good	70	74.28	4.28	6.1	Great	0.54	10	-9.46	Good
R2	Test 5	N-26	0	87	92.15	5.15	5.9	Good	88	91.28	3.28	3.7	Great	0.87	-1	1.87	Great
R3	Test 5	J30	0	69	68.76	-0.24	0.3	Great	68	67.07	-0.93	1.4	Great	1.69	1	0.69	Great
R4	Test 5	N-22	0	45	51.42	6.42	14.3	Good	45	50.05	5.05	11.2	Good	1.37	0	1.37	Great
R5	Test 5	J32	0	50	47.52	-2.48	5	Great	43	45.12	2.12	4.9	Great	2.4	7	-4.6	Great
R6	Test 5	J34	0	63	71.35	8.35	13.3	Good	60	68.66	8.66	14.4	Good	2.69	3	-0.31	Great
Q1	Test 6	N-37	1060	0	78.36	78.36			40	47.41	7.41	18.5		30.95	-40		
Q2	Test 6	N-35	805	0	61.03	61.03			23	29.51	6.51	28.3		31.52	-23		
R1	Test 6	J40	0	66	68.4	2.4	3.6	Great	39	39.78	0.78	2	Great	28.62	27	1.62	Great
R2	Test 6	J38	0	90	81.4	-8.6	9.6	Good	63	53.2	-9.8	15.6	Good	28.2	27	1.2	Great
R3	Test 6	J_504	0	78	77.07	-0.93	1.2	Great	50	51.13	1.13	2.3	Great	25.94	28	-2.06	Great
R4	Test 6	N-38	0	83	86.6	3.6	4.3	Great	54	55.65	1.65	3.1	Great	30.95	29	1.95	Great
R5	Test 6	J_482	0	86	90.06	4.06	4.7	Great	60	63.01	3.01	5	Great	27.05	26	1.05	Great
R6	Test 6	J_509	0	60	59.73	-0.27	0.5	Great	34	35.66	1.66	4.9	Great	24.07	26	-1.93	Great

Appendix E

Existing Fire Flow Deficiencies



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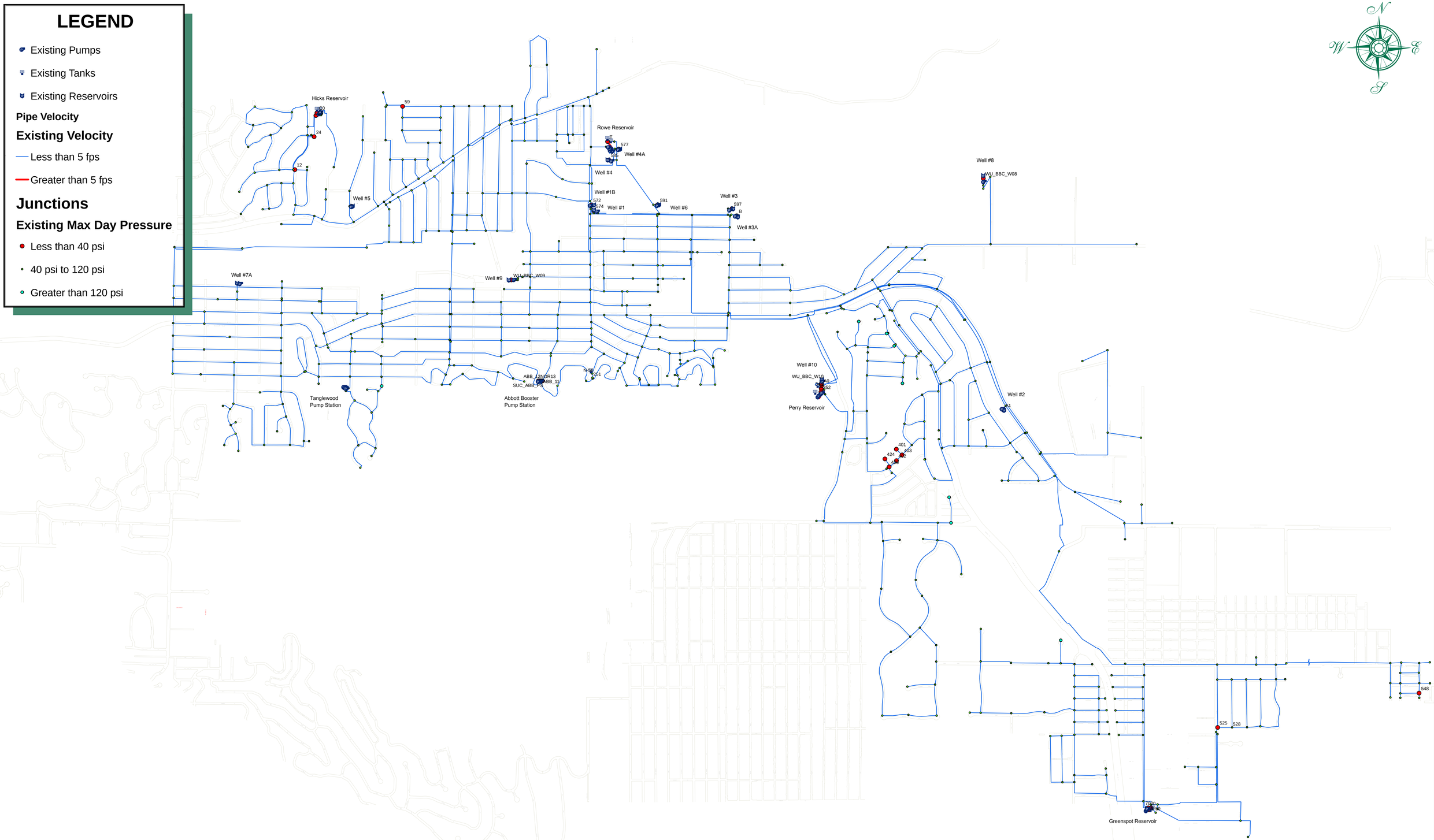
BIG BEAR CITY COMMUNITY SERVICE DISTRICT
MASTER PLAN

Existing Fire Flow Deficiencies

APPENDIX
E

Appendix F

Existing Maximum Day Pressure and Velocity Deficiencies



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MASTER PLAN

Existing Max Day Pressures and Velocities

APPENDIX F

Appendix G

Proposed Pipeline Improvement Projects

APPENDIX G1: BBCCSD Year 2008 Proposed CIP Improvements

CIP Project Number	MODEL ID	LENGTH	DIAMETER	ROUGH NESS	ZONE	LOCATION	Improvement Type	Original Model ID	Deficient Hydrants	Reason for Deficiency	Existing Available FF (Assuming Velocity Constraint) gpm	Existing Available FF (NO Velocity Constraint) gpm	Improved FF (Assuming Velocity Constraint) gpm	Notes
(Year 2008) PIPELINE VELOCITY IMPROVEMENTS: PROPOSED PIPES														
43	MAIN_IMP_113	10.44	8	130	MAIN	E. Big Bear Blvd. and Shore Dr.	Replacement	735						
	MAIN_IMP_114	44.23	8	130	MAIN	E. Big Bear Blvd. and Shore Dr.	Replacement	452						
LENGTH OF EX. VELOCITY IMPROVEMENTS (ft)					***54.7***									
LENGTH OF EX. VELOCITY IMPROVEMENTS(miles)					***0.01***									

CIP Project Number	MODEL ID	LENGTH	DIAMETER	ROUGH NESS	ZONE	LOCATION	Improvement Type	Original Model ID	Deficient Hydrants	Reason for Deficiency	Existing Available FF (Assuming Velocity Constraint) gpm	Existing Available FF (NO Velocity Constraint) gpm	Improved FF (Assuming Velocity Constraint) gpm	Notes
(Year 2008) FIREFLOW IMPROVEMENTS FOR HYDRANTS WITH LOW PRESSURE AS THE DRIVER : PROPOSED PIPELINES														
1	MAIN_IMP_01	1,405.59	12	130	MAIN	State Ln. - b/t Lakewood Dr. and Willow	Replacement	1699	515	Pressure	1,391	1,391	2,110	
	MAIN_IMP_101	1,117.00	12	130	MAIN	State Ln. - b/t Willow Ln. and Ash Ln.	Parallel	1704/1701	516	Pressure	1,402	1,402	2,663	
	MAIN_IMP_102	1,405.59	12	130	MAIN	State Ln. - b/t Ash Ln. and Cypress Ln.	Parallel	1705	517	Pressure	1,427	1,427	2,943	
									519	Pressure	1,739	1,739	4,369	
									520	Pressure	1,399	1,399	1,567	
									522	Pressure	1,350	1,350	2,111	
									523	Pressure	1,394	1,394	4,224	
									524	Pressure	853	1,199	2,814	
									529	Pressure	659	1,286	3,363	
									535	Pressure	1,433	1,433	3,416	
									538	Pressure	1,447	1,447	3,304	
									539	Pressure	1,394	1,394	3,203	
									540	Pressure	1,423	1,423	3,257	
									541	Pressure	1,303	1,303	2,940	
									542	Pressure	1,294	1,294	2,963	
									544	Pressure	1,321	1,321	1,567	
									548	Pressure	327	327	648	
2	Hicks Pump Improvement (see last section of CIP table for details)													
3a	Abbott Pump Improvement (see last section of CIP table for details)													
3b	ABB_IMP_01	1,087.86	8	130	ABBOTT	Beaumont Lane - b/t Abbot PS and Shak	Replacement	289	204	Pressure	881	1,787	1,567	Top Priority with replacement of pump for Abbot Improvements
	ABB_IMP_02	168.73	8	130	ABBOTT	Beaumont Lane - West of Shakespeare	Replacement	1296	257	Pressure	568	568	1,552	
	ABB_IMP_03	1,135.14	8	130	ABBOTT	Beaumont Lane - b/t Shakespeare Lane a	Replacement	1411	206	Pressure	881	1,787	1,567	
	ABB_IMP_04	73.85	8	130	ABBOTT	Beaumont Lane and Kean Way	Replacement	1413	310	Pressure	626	626	1,555	
	ABB_IMP_05	371.82	8	130	ABBOTT	Kean Way - Southeast fo Beaumont Lane	Replacement	1412	311	Pressure	580	580	1,556	
4	GRE_IMP_01	581.95	8	130	GREENSPOT	Cedar Ln. - b/t Hwy 38 and Hatchery Dr.	Replacement	1634	484	Pressure	392	612	1,455	
	GRE_IMP_02	538.46	8	130	GREENSPOT	Cedarpine Ln. - b/t Hwy 38 and Hatchery	Replacement	637	489	Pressure	1,654	1,915	3,055	
	GRE_IMP_03	525.56	8	130	GREENSPOT	Shady Ln. - b/t Hwy 38 and Hatchery Dr.	Replacement	1638	494	Pressure	1,654	1,915	3,055	
	GRE_IMP_04	499.66	8	130	GREENSPOT	Manzita Ln. - b/t Hwy 38 and Hatchery D	Replacement	641	499	Pressure	1,654	1,915	3,055	
	GRE_IMP_05	467.38	8	130	GREENSPOT	Fox Rd. - b/t Hwy 38 and Hatchery Dr.	Replacement	642	506	Pressure	881	1,286	1,567	
5	GRE_IMP_10	258.35	10	130	GREENSPOT	Fern Ln. - West of Willow Ln.	Replacement	682	531	Pressure	881	910	1,627	Appears to be very rural area
	GRE_IMP_11	675.84	8	130	GREENSPOT	Willow Ln. - b/t Fern Ln. and Fox Rd.	Replacement	684	532	Pressure	985	985	1,657	
	GRE_IMP_12	58.58	8	130	GREENSPOT	Willow Ln, at Fox Rd.	Replacement	P77	533	Pressure	986	986	1,649	
	GRE_IMP_13	344.89	8	130	GREENSPOT	Fern Ln. - West of Willow Ln.	Replacement	683	526	Pressure	1,001	1,001	1,684	
									527	Pressure	503	957	1,672	
6	MAIN_IMP_99	1,572.34	8	130	MAIN	Lakewood Dr. - b/t Jenson Dr. and Erwin	New	N/A	466	Pressure	1,259	1,259	1,692	
	MAIN_IMP_02	656.47	8	130	MAIN	Lakewood Dr. - b/t Jenson Dr. and Erwin	Replacement	621	467	Pressure	1,421	1,421	2,083	
									465	Pressure	881	1,155	2,347	
									470	Pressure	1,386	1,386	1,567	
7	ABB_IMP_06	280.51	8	130	ABBOTT	Bernhardt Lane - West of Wallace Ln.	Replacement	P-5	312	Pressure	561	561	1,866	2nd Priority for Abbot Improvements
	ABB_IMP_07	204.21	8	130	ABBOTT	Bernhardt Lane - b/t Wallace Ln.	Replacement	P-5	313	Pressure	523	523	1,789	
	ABB_IMP_08	24.15	8	130	ABBOTT	Bernhardt Lane -at Wallace Ln.	New	N/A	351	Pressure	518	518	1,560	

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APPENDIX G1: BBCCSD Year 2008 Proposed CIP Improvements

CIP Project Number	MODEL ID	LENGTH	DIAMETER	ROUGH NESS	ZONE	LOCATION	Improvement Type	Original Model ID	Deficient Hydrants	Reason for Deficiency	Existing Available FF (Assuming Velocity Constraint) gpm	Existing Available FF (NO Velocity Constraint) gpm	Improved FF (Assuming Velocity Constraint) gpm	Notes
8	ABB_IMP_09	86.46	8	130	ABBOTT	Marlowe Dr. - West of Barrymore Road	Replacement	1442	N-55	Pressure	477	558	1,619	3rd Priority for Abbot Improvements
									321	Pressure	581	581	1,561	
9	ABB_IMP_10	339.75	8	130	ABBOTT	Adams Dr. - Southeast of Barrymore Rd.	Replacement	1140	323	Pressure	522	522	1,672	4th Priority for Abbot Improvements
									324	Pressure	570	570	1,717	
									349	Pressure	566	566	1,566	
10	ABB_IMP_11	403.57	8	130	ABBOTT	Adams Dr. - Northeast of Barrymore Rd.	Replacement	434	325	Pressure	511	512	1,567	5th Priority for Abbot Improvements - Deadend
11	MAIN_IMP_027	184.47	8	130	MAIN	Wesley Ct. - off of Hillen Dale Ln.	Replacement	1315	144	Pressure	392	831	1,556	Deadend
12	MAIN_IMP_100	946.84	8	130	MAIN	Monmouth Rd. - West of Ringwood Trail	Replacement	1625	469	Pressure	660	660	954	
13	GRE_IMP_14	222.51	10	130	GREENSPOT	Shady Ln, Cul de Sac - West of Wilderne	Replacement	P-40	N-36	Pressure	1,416	1,416	1,461	Elevation makes it difficult to get above 1500 gpm. May want to consider 8-in for water qual
14	GRE_IMP_09	486.48	8	130	GREENSPOT	South of Hemlock Ln.	Replacement	1727	553	Pressure	210	210	220	Appears to be very rural area
									554	Pressure	283	283	402	
(Year 2008) FIREFLOW IMPROVEMENTS FOR HYDRANTS WITH HIGH VELOCITY AND/OR LOW PRESSURE AS THE DRIVER : PROPOSED PIPELINES														
15	MAIN_IMP_021	238.27	8	130	MAIN	W. Division Dr. - b/t W. Big Bear Blvd. an	Replacement	1336	122	Velocity	1,369	1,927	4,946	
	MAIN_IMP_020	260.28	8	130	MAIN	W. Division Dr. - b/t Sherwood Blvd. amd	Replacement	1329	128	Velocity	1,169	1,610	2,346	
	MAIN_IMP_022	228.02	8	130	MAIN	W. Division Dr. - b/t Robinhood Blvd. and	Replacement	1328	134	Velocity	1,172	2,941	1,939	
	MAIN_IMP_023	236.56	8	130	MAIN	W. Division Dr. - b/t Rainbow Blvd. and S	Replacement	P-53	123	Velocity	779	1,553	2,718	
	MAIN_IMP_024	533.28	8	130	MAIN	Sugarloaf Blvd. - b/t Division Dr. and Gild	Replacement	P19	127	Pressure	470	471	2,521	
	MAIN_IMP_025	633.47	8	130	MAIN	Sugarloaf Blvd. - b/t Gildart Dr, and Hiller	Replacement	1323	135	Velocity	1,217	2,420	2,265	
	MAIN_IMP_026	891.53	8	130	MAIN	Sugarloaf Blvd. - b/t Hillen Dale Dr. and F	Replacement	1324	124	Velocity	1,067	1,682	2,426	
	MAIN_IMP_037	1,088.56	8	130	MAIN	Sherwood Blvd. - b/t Division Dr. and Hill	Replacement	1335	136	Velocity	1,356	2,166	2,159	
	MAIN_IMP_120	593.79	8	130	MAIN	Robinhood Blvd. - b/t Division Dr. and Gil	Replacement	1330	N-2	Pressure	574	984	2,350	
	MAIN_IMP_16	1,467.15	8	130	MAIN	Robinhood Blvd. - b/t Gildart Dr. and Pine	Replacement	1331	J22	Pressure	7,153	7,003	6,836	
									J_146	Velocity	1,266	1,266	1,562	
									556	Pressure	1,293	1,293	2,085	
									152	Pressure	1,198	1,198	1,997	
									153	Pressure	1,201	1,202	1,567	
									154	Pressure	1,314	1,314	2,186	
16	MAIN_IMP_74	189.92	8	130	MAIN	Bernhardt Lane - b/t E. Big Bear Blvd. an	Replacement	1403	299	Velocity	1,523	1,523	1,713	
	MAIN_IMP_75	222.45	8	130	MAIN	Bernhardt Lane - b/t Irving Way and Barr	Replacement	1423	300	Velocity	1,207	3,284	2,183	
	MAIN_IMP_76	222.45	8	130	MAIN	Bernhardt Lane - b/t Barrett Way and Boo	Replacement	1422	301	Velocity	1,255	2,952	2,837	
	MAIN_IMP_77	322.11	8	130	MAIN	Booth Way - East of Bernhardt Lane	Replacement	1421	303	Velocity	1,119	2,402	6,044	
	MAIN_IMP_78	173.13	8	130	MAIN	Irving Way - East of Bernhardt Lane	Replacement	1424	306	Velocity	1,046	2,768	2,718	
	MAIN_IMP_79	221.88	8	130	MAIN	Bernhardt Lane - b/t Booth Way and Gar	Replacement	1420	307	Pressure	1,445	2,768	2,718	
	MAIN_IMP_80	217.7	8	130	MAIN	Garrick Way - b/t Bernhardt Lane and Be	Replacement	1416						
	MAIN_IMP_89	179.23	8	130	MAIN	Beaumont Lane - b/t Garrick Way and Ke	Replacement	415						
17	MAIN_IMP_84	463.45	8	130	MAIN	Wendy Ave. - b/t Tiger Lily Dr. and Hale I	Replacement	96	60	Velocity	1,312	2,144	2,001	NorthSouth Improvements are priority, additional east/west improvement create a loop which
	MAIN_IMP_83	635.46	8	130	MAIN	Wendy Ave. - north from Hale Dr.	Replacement	97	59	Pressure	1,312	2,144	2,001	
	MAIN_IMP_82	36.83	8	130	MAIN	North End of Wendy Ave.	Replacement	1780	53	Pressure	1,312	2,144	2,001	
	MAIN_IMP_81	310.26	8	130	MAIN	West of Pioneer Ln. - b/t Wendy Ave and	Replacement	98	55	Pressure	1,312	2,144	2,001	
	MAIN_IMP_03	393.81	8	130	MAIN	Michael Ave. - b/t North Shore Drive E. a	Replacement	1110	246	Velocity	811	2,028	2,203	
	MAIN_IMP_029	238.6	8	130	MAIN	Michael Ave. - b/t Cinderella Dr. and Curl	Replacement	1109						
	MAIN_IMP_030	242.24	8	130	MAIN	Michael Ave. - b/t Curly Dr. and Mullins D	Replacement	1108						
	MAIN_IMP_04	212.5	8	130	MAIN	Michael Ave. - b/t Mullins Dr. and Pionee	Replacement	1107						
	MAIN_IMP_05	726.14	8	130	MAIN	Pioneer Ln. - b/t Nana Ave. and Michael	Replacement	101						
	MAIN_IMP_40	248.69	8	130	MAIN	Pioneer Ln. - b/t Michael Ave. and Anita	Replacement	111						
	MAIN_IMP_41	289.84	8	130	MAIN	Pioneer Ln. - b/t Anita Ave. and Myrtle Av	Replacement	112						
	MAIN_IMP_42	290.27	8	130	MAIN	Pioneer Ln. - b/t Myrtle Ave. and Greenw	Replacement	113						
	MAIN_IMP_43	291.73	8	130	MAIN	Pioneer Ln. - b/t Greenway Dr. and Sierra	Replacement	114						
	MAIN_IMP_44	236.16	8	130	MAIN	Pioneer Ln. - b/t Sierra Ave. and Sequoia	Replacement	115						
	MAIN_IMP_45	293.86	8	130	MAIN	Sequoia Ave. - b/t Pioneer and North Sho	Replacement	1147						
18	MAIN_IMP_10	136.35	8	130	MAIN	Paradise Way - b/t E. Big Bear Blvd. and	Replacement	1190	235	Velocity	686	1,796	2,161	

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APPENDIX G1: BBCCSD Year 2008 Proposed CIP Improvements

CIP Project Number	MODEL ID	LENGTH	DIAMETER	ROUGH NESS	ZONE	LOCATION	Improvement Type	Original Model ID	Deficient Hydrants	Reason for Deficiency	Existing Available FF (Assuming Velocity Constraint) gpm	Existing Available FF (NO Velocity Constraint) gpm	Improved FF (Assuming Velocity Constraint) gpm	Notes
	MAIN_IMP_11	121.21	8	130	MAIN	Paradise Way - b/t Booth Way and Mojav	Replacement	1191	236	Velocity	801	1,639	2,293	
	MAIN_IMP_12	117.64	8	130	MAIN	Paradise Way - b/t Mojave Blvd. and Gar	Replacement	1192	237	Pressure	833	1,455	4,540	
	MAIN_IMP_13	129.4	8	130	MAIN	Paradise Way - b/t Garrick Way and She	Replacement	1193	238	Pressure	1,164	1,347	3,755	
	MAIN_IMP_14	106.21	8	130	MAIN	Paradise Way - b/t Sherwood Blvd. and k	Replacement	1194						
19	MAIN_IMP_122	662.5	8	130	MAIN	Maltby Blvd. - b/t Pintail and Bluebill Dr.	Replacement	1363	557	Pressure	734	634	1,884	
	MAIN_IMP_09	728.1	8	130	MAIN	Maltby Blvd. - b/t Bluebill Dr. and Shore D	Replacement	1850	264	Velocity	1,093	2,067	1,858	
	MAIN_IMP_105	40.29	8	130	MAIN	Maltby Blvd. and Shore Drive	Replacement	188	265	Velocity	2,881	4,062	2,651	
	MAIN_IMP_106	209.29	8	130	MAIN	Shore Dr. - b/t Maltby Blvd. and Meadow	Replacement	368	332	Velocity	1,391	3,362	2,791	
	MAIN_IMP_107	238.5	8	130	MAIN	Shore Dr. - b/t Meadow Lane and Elysian	Replacement	369						
	MAIN_IMP_108	237.4	8	130	MAIN	Shore Dr. - b/t Elysian Blvd. and Mtn. Vie	Replacement	446						
	MAIN_IMP_109	243.08	8	130	MAIN	Shore Dr. - b/t Mtn. View Blvd. and Ange	Replacement	447						
	MAIN_IMP_110	711	8	130	MAIN	Shore Dr. - b/t Angeles Blvd. and Country	Replacement	448						
20	MAIN_IMP_60	520.42	8	130	MAIN	San Anselmo Dr. - East of Paradise Way	Replacement	1154	89	Pressure	1,312	2,144	2,001	
	MAIN_IMP_53	50	8	130	MAIN	San Anselmo Dr. and Paradise Way	New	N/A	96	Velocity	1,312	2,144	2,001	
	MAIN_IMP_20	561.53	8	130	MAIN	Mt. Doble Dr. - b/t Arbor Ln. and San Ans	Replacement	1152	88	Pressure	1,312	2,144	2,001	
	MAIN_IMP_54	656.22	8	130	MAIN	Arbor Ln. - East of Paradise Way	Replacement	157						Needed to get flow to hydrant 88
	MAIN_IMP_55	279.34	8	130	MAIN	Arbor Ln. - b/t Mt. Doble Dr. and Sequoia	Replacement	1151	86	Velocity	1,312	2,144	2,001	
	MAIN_IMP_56	265.86	8	130	MAIN	Arbor Ln. - b/t Sequoia Ave. and Sierra A	Replacement	1082						Only needed to create loop for better transmission.
21	MAIN_IMP_08	599.21	8	130	MAIN	Mnt. View Blvd. - b/t Paradise Way and G	Replacement	1188	226	Pressure	735	1,323	2,529	
	MAIN_IMP_71	539.38	8	130	MAIN	Mnt. View Blvd. - b/t Gold Mtn. and Mt. D	Replacement	1175	216	Velocity	881	1,787	1,567	
	MAIN_IMP_72	242.21	8	130	MAIN	Mt Doble Dr. - b/t Mnt. View Blvd. and An	Replacement	1183	217	Velocity	881	1,787	1,567	
	MAIN_IMP_73	240.74	8	130	MAIN	Mt Doble Dr. - b/t Angeles Blvd. and Bark	Replacement	1185						
22	MAIN_IMP_019	474.04	8	130	MAIN	Sugarloaf Blvd. - b/t Pineview Dr. and Wi	Replacement	1303	140	Pressure	652	1,240	2,534	
	MAIN_IMP_018	24.56	8	130	MAIN	Sugarloaf Blvd. At Winding Lane	Replacement	1302	163	Velocity	1,498	2,155	3,251	
	MAIN_IMP_15	267.73	8	130	MAIN	Sugarloaf Blvd. - b/t Winding Lane and D	Replacement	1628	162	Velocity	872	1,942	2,655	
	MAIN_IMP_017	241.53	8	130	MAIN	Dawn Dr. - b/t Sugarloaf Blvd. and Rainb	Replacement	1266						
	MAIN_IMP_016	370.42	8	130	MAIN	Dawn Dr. - b/t Rainbow Blvd. and W. Big	Replacement	1265						
23	MAIN_IMP_033	971.56	8	130	MAIN	Aeroplane Blvd. - b/t Gold Mtn. and Gree	Replacement	1204	212	Pressure	881	1,787	1,567	
	MAIN_IMP_034	303.43	8	130	MAIN	Greenway Dr. - b/t Aeroplane Blvd. and S	Replacement	1213	211	Velocity	881	1,787	1,567	
	MAIN_IMP_035	253.52	8	130	MAIN	Greenway Dr. - b/t Sherwood Blvd. and M	Replacement	1211	210	Velocity	881	1,787	1,567	
	MAIN_IMP_036	244.89	8	130	MAIN	Greenway Dr. - b/t Mojave Blvd. and Eas	Replacement	1209						
24	MAIN_IMP_94	250.02	8	130	MAIN	Stanford. - b/t Whipple and Greenspot Ro	Replacement	1522	388	Velocity	1,078	2,248	1,870	
	MAIN_IMP_97	327.76	8	130	MAIN	Sites Way - b/t Montclair Dr. and Greensp	Replacement	526	398	Pressure	1,500	1,500	1,976	
	MAIN_IMP_123	530.00	8	130	MAIN	Greenspot Dr. north of Sites Way	Replacement	1523	387	Velocity	38	77	1,566	Deadend
25	GRE_IMP_06	133.86	8	130	GREENSPOT	Glencove Drive	Replacement	YY	551	Velocity	866	1,868	1,548	Appears to be very rural area
	GRE_IMP_07	1,000.00	8	130	GREENSPOT	Glencove Drive	Replacement	728	534	Velocity	1,008	1,008	1,747	
	GRE_IMP_15	460.38	8	130	GREENSPOT	Willow Ln. - b/t Glencove Dr. and Tuplin I	Replacement	728						
26	MAIN_IMP_17	150	8	130	MAIN	Holden Ave at Hale Dr.	New	N/A	35	Pressure	1,312	2,144	2,001	Deadend (creating loop)
	MAIN_IMP_115	150	8	130	MAIN	Hale Dr. - b/t Van Dusen Canyon Dr. and	New	N/A	34	Velocity	1,312	2,144	2,001	
	MAIN_IMP_116	150	8	130	MAIN	Hale Dr. - b/t Holden Ave. and Wendy Av	New	N/A						
(Year 2008) FIREFLOW IMPROVEMENTS FOR HYDRANTS WITH HIGH VELOCITY AS THE DRIVER : PROPOSED PIPELINES														
27	MAIN_IMP_46	76.97	8	130	MAIN	Sequoia Ave. at North Shore Drive E.	Replacement	1167	116	Velocity	543	2,636	2,095	All of these improvements are driven by high velocity during fire flow.
	MAIN_IMP_47	287.56	8	130	MAIN	Sequoia Ave. - b/t North Shore Drive E. a	Replacement	1080	99	Velocity	1,312	2,144	2,001	
	MAIN_IMP_48	287.56	8	130	MAIN	Cinderella Dr. - b/t Sequoia and Mt. Doble	Replacement	1128	100	Velocity	1,312	2,144	2,001	
	MAIN_IMP_49	236.28	8	130	MAIN	Cinderella Dr. - b/t Mt. Doble and Gold M	Replacement	1129	101	Velocity	1,312	2,144	2,001	
	MAIN_IMP_50	377.52	8	130	MAIN	Cinderella Dr. - b/t Gold Mtn. and Hugo L	Replacement	1130	98	Velocity	1,312	2,144	2,001	
	MAIN_IMP_85	120.15	8	130	MAIN	Hugo Ln. - b/t Cinderella Dr. and Cindere	Replacement	144	108	Velocity	1,312	2,144	2,001	
	MAIN_IMP_88	673.32	8	130	MAIN	Hugo Ln. - b/t Cinderella Dr. and North S	Replacement	135	109	Velocity	1,312	2,144	2,001	
	MAIN_IMP_111	105.91	8	130	MAIN	North Shore Drive - b/t Hugo Ln. and Dur	Replacement	134	110	Velocity	1,025	4,545	2,038	
	MAIN_IMP_87	208.8	8	130	MAIN	North Shore Drive - b/t Dumas Ln. and Pa	Replacement	132						
	MAIN_IMP_86	115.08	8	130	MAIN	North Shore Drive - b/t Pan Springs Ln. a	Replacement	133						
	MAIN_IMP_51	199.8	8	130	MAIN	Hugo Ln. - b/t Tiger Lily Dr. and Grenfall	Replacement	1145						

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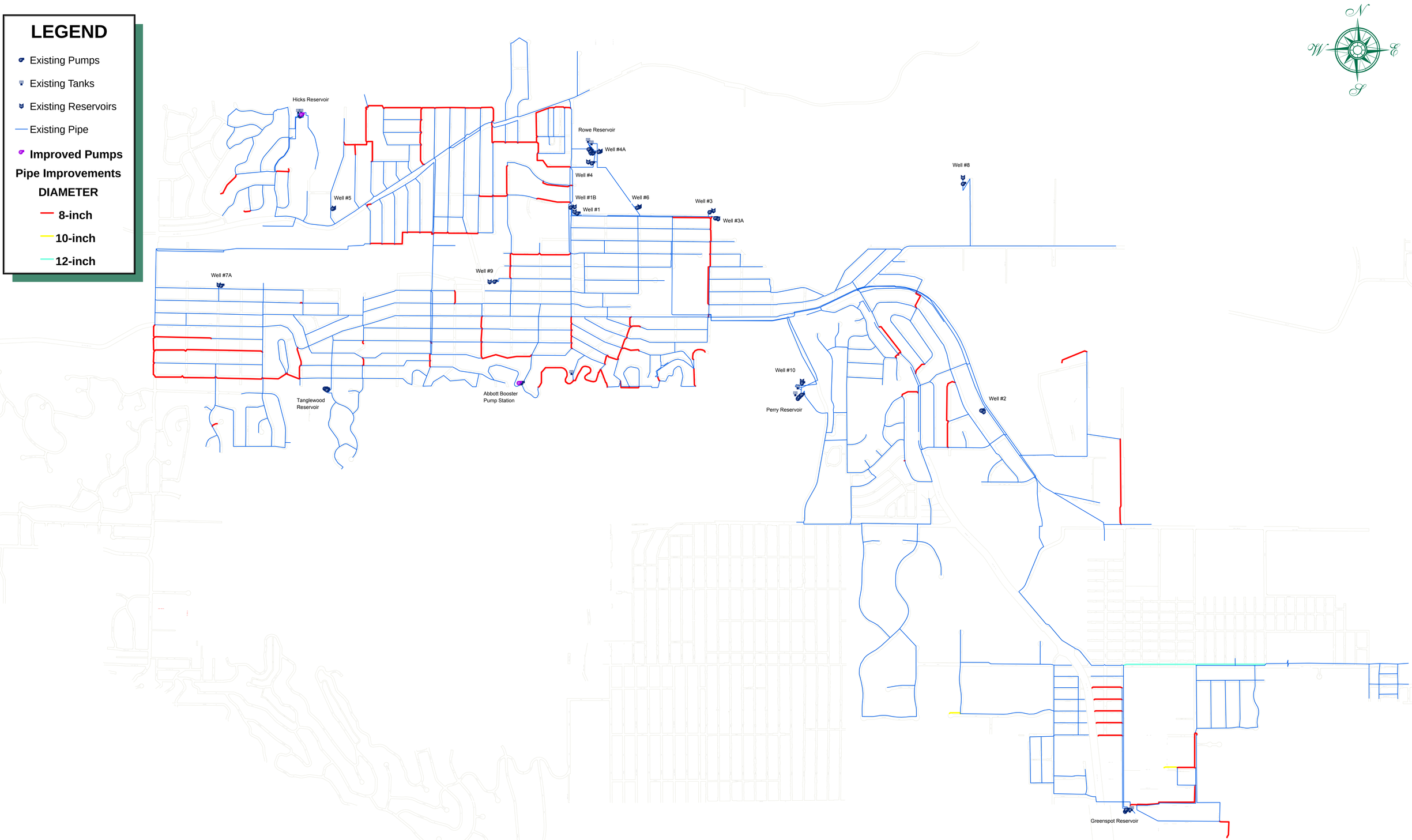
APPENDIX G1: BBCCSD Year 2008 Proposed CIP Improvements

CIP Project Number	MODEL ID	LENGTH	DIAMETER	ROUGH NESS	ZONE	LOCATION	Improvement Type	Original Model ID	Deficient Hydrants	Reason for Deficiency	Existing Available FF (Assuming Velocity Constraint) gpm	Existing Available FF (NO Velocity Constraint) gpm	Improved FF (Assuming Velocity Constraint) gpm	Notes
	MAIN_IMP_52	824.91	8	130	MAIN	Grenfall Ln - Cul de sac to Paradise Way	Replacement	1150						
28	MAIN_IMP_57	217.51	8	130	MAIN	West Meadow Ln. - b/t Sierra Ave. and G	Replacement	1074	71	Velocity	1,312	2,144	2,001	All of these improvements are driven by high velocity during fire flow.
	MAIN_IMP_58	363.71	8	130	MAIN	West Meadow Ln. - b/t Greenway Dr. and	Replacement	1073	74	Velocity	1,312	2,144	2,001	
	MAIN_IMP_59	296.56	8	130	MAIN	West Meadow Ln. - b/t Myrtle Ave. and A	Replacement	1064	75	Velocity	1,312	2,144	2,001	
	MAIN_IMP_61	39.83	8	130	MAIN	West Meadow Ln. at Anita Ave.	Replacement	P45	77	Velocity	1,312	2,144	2,001	
	MAIN_IMP_62	234.84	8	130	MAIN	West Meadow Ln. - b/t Anita Ave. and Mi	Replacement	57						
	MAIN_IMP_63	239.64	8	130	MAIN	West Meadow Ln. - b/t Michael Ave. and	Replacement	52						
	MAIN_IMP_64	455.25	8	130	MAIN	West Meadow Ln. - b/t Peter Ave. and Ti	Replacement	51						
	MAIN_IMP_65	241.44	8	130	MAIN	West Meadow Ln. - b/t Tinkerbell Ave. an	Replacement	46						
	MAIN_IMP_66	226.52	8	130	MAIN	West Meadow Ln. - b/t Nana Ave. and W	Replacement	45						
29	MAIN_IMP_90	696	8	130	MAIN	Greenspot Rd. - b/t Malabar Way and Sta	Replacement	508	384	Velocity	1,342	3,478	3,614	
	MAIN_IMP_92	55.53	8	130	MAIN	Greenspot Rd. and Mann Dr.	Replacement	1511	385	Velocity	827	2,673	2,532	
	MAIN_IMP_93	74.46	8	130	MAIN	Greenspot Rd. and Mann Dr.	Replacement	1510	386	Velocity	1,269	3,145	5,052	
30	MAIN_IMP_67	130.11	8	130	MAIN	Sawmill Dr. - b/t Aeroplane Blvd. and Rai	Replacement	222	193	Velocity	881	1,787	1,567	
	MAIN_IMP_68	116.54	8	130	MAIN	Sawmill Dr. - at Rainbow Blvd.	Replacement	223	194	Velocity	881	1,787	1,567	
31	MAIN_IMP_118	232.43	8	130	MAIN	East of Panamint Mtn. Dr. - b/t Whisperin	Replacement	15	11	Velocity	1,312	2,144	2,001	
	MAIN_IMP_119	61.4	8	130	MAIN	White Mtn. Road cul de sac	Replacement	26	10	Velocity	881	1,894	1,567	Deadend
32	MAIN_IMP_95	818.18	8	130	MAIN	Brewer Way - b/t Malabar Way and Gilroy	Replacement	1552	397	Velocity	1,500	1,500	1,976	
	MAIN_IMP_96	501.78	8	130	MAIN	Brewer Way - b/t Gilroy Dr. and Zaca Rd.	Replacement	1554	411	Velocity	1,023	1,624	3,493	
33	MAIN_IMP_117	123.8	8	130	MAIN	Feather Mtn. Dr. - b/t Glen Mtn. Rd west	Replacement	20	22	Velocity	1,312	2,144	2,001	Deadend
34	MAIN_IMP_70	249.98	8	130	MAIN	Rosehill Dr. - b/t West Fair way and West	Replacement	1215	254	Velocity	1,238	2,919	2,241	
35	MAIN_IMP_028	25	8	130	MAIN	Country Club Blvd. at Keiner	New	N/A	717	Velocity	881	1,698	1,968	
36	MAIN_IMP_032	60.31	8	130	MAIN	Big Tree Dr.- off of Sherwood Blvd.	Replacement	1240	181	Velocity	392	1,825	1,567	Deadend
37	MAIN_IMP_031	149.25	8	130	MAIN	Big Tree Dr.- b/t Aeroplane Blvd. and Rai	Replacement	1245	179	Velocity	2,447	2,795	2,447	
38	MAIN_IMP_69	38.39	8	130	MAIN	Sherwood Blvd. And Sawmill Dr.	Replacement	P57	191	Velocity	881	1,787	1,567	
39	MAIN_IMP_98	11.92	8	130	MAIN	Monclair Dr. - North of Bodie	Replacement	566	422	Velocity	881	1,521	1,567	
40	HICK_IMP_1	479.48	8	130	HICKS	Panamint Mtn. Dr. - b/t Lookout Mnt. Rd \	Replacement	11	J_15	Velocity	881	1,292	1,567	
41	MAIN_IMP_91	274.96	8	130	MAIN	Walkway - b/t E. Big Bear Blvd. and Mala	New	N/A	381	Velocity	986	1,521	2,085	
42	MAIN_IMP_121	88.43	8	130	MAIN	North Shore Drive - East of Wendy Ave.	Replacement	1058	705	Velocity	902	3,657	1,591	
LENGTH OF EX. FF IMPROVEMENTS (ft)					***52,822***									
LENGTH OF EX. FF IMPROVEMENTS(miles)					***10.0***									

CIP Project Number	MODEL ID	PROPOSED FLOW	PROPOSED HEAD	ZONE	LOCATION	Improvement Type	Original Model ID	Deficient Hydrants	Reason for Deficiency	Existing Available FF (Assuming Velocity Constraint) gpm	Existing Available FF (NO Velocity Constraint) gpm	Improved FF (Assuming Velocity Constraint) gpm	Notes
(Year 2008) PUMPING IMPROVEMENTS: PROPOSED PUMPS													
2	HIC_FP	1500	120	Hicks	Mound St.	Replacement	HIC_FP	J_7	Pressure	1,235	1,235	1,858	
								J_1	Pressure	908	908	1,570	
								J_2	Pressure	1,422	1,422	2,211	
								J_5	Pressure	1,454	1,454	2,307	
								8	Pressure	1,249	1,249	1,532	
								6	Pressure	1,255	1,255	1,994	
								J_23	Pressure	1,228	1,228	1,738	

APPENDIX G1: BBCCSD Year 2008 Proposed CIP Improvements

CIP Project Number	MODEL ID	LENGTH	DIAMETER	ROUGH NESS	ZONE	LOCATION	Improvement Type	Original Model ID	Deficient Hydrants	Reason for Deficiency	Existing Available FF (Assuming Velocity Constraint) gpm	Existing Available FF (NO Velocity Constraint) gpm	Improved FF (Assuming Velocity Constraint) gpm	Notes
									239	Pressure	1,235	1,235	1,567	
									9	Pressure	1,417	1,417	1,659	
									J_13	Pressure	1,435	1,435	2,319	
									J_14	Pressure	1,480	1,480	1,529	
3a	ABB_BP3	1500	320		Abbot	Starr Dr. and Bold Mountain Dr.	Replacement	ABB_BP3	201	Pressure	881	1,787	1,567	
									202	Pressure	881	1,787	1,567	
TOTAL PUMP CAPACITY IMPROVEMENTS (gpm)					***3,000.00***									
TOTAL PUMP HEAD IMPROVEMENTS (feet)					***440.00***									



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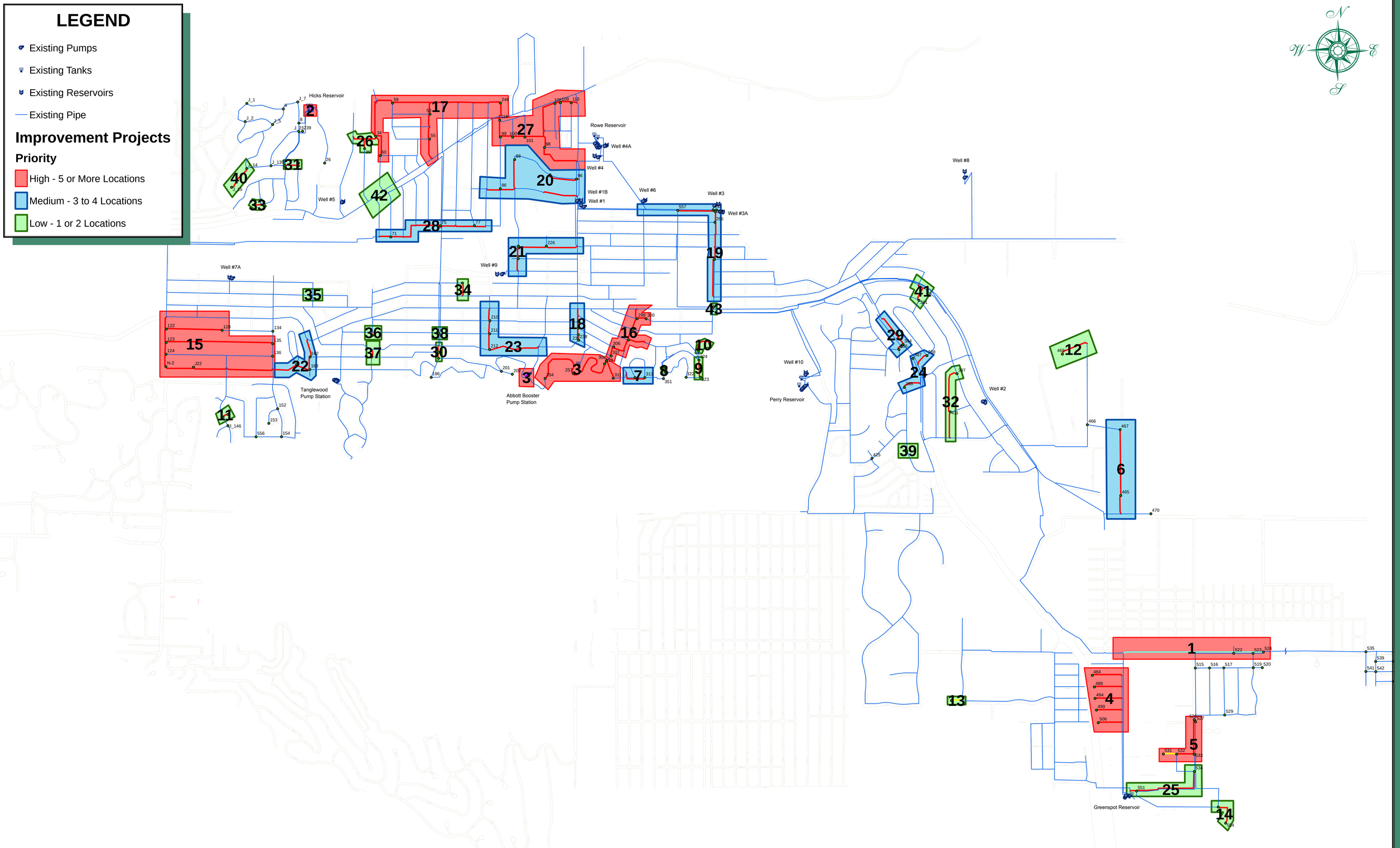
BIG BEAR CITY COMMUNITY SERVICE DISTRICT
MASTER PLAN

All Pipe Improvements

APPENDIX G2



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BIG BEAR CITY COMMUNITY SERVICE DISTRICT MASTER PLAN

CIP Projects

APPENDIX G3

Appendix H

Fluoride Treatment Concept Design Report (SAE, 2006)

9-18-06

BBCCSD – Fluoride Treatment Concept Design Report

1. Introduction

2. Fluoride Removal Systems

3. Disposal Alternatives of Fluoride Backwash Water

4. Plant Layout

5. Comparison & Summary

District Engineer: Wilson F. So, P.E.

Assistant Project Engineer: Hao Zhang, E.I.T.

So & Associates Engineers, Inc.

September 18th, 2006

1. Introduction

Following completion of drilling of Well No. 3B, fluoride concentration was found to be exceeding the Maximum Contaminant Level (MCL) of the Safe Drinking Water Act (SDWA). Groundwater (or well water) may contain a high concentration of fluoride coming from soils and rocks. Fluoride ions have also been found in the waste discharges from process streams in a number of industries, including glass manufacturing, electroplating operations, fertilizer, and the semiconductor industry.

Fluoride is not considered a health threat to human beings. As a matter of fact, the importance of fluoride in promoting strong human teeth and the role of fluoride intake from drinking water in controlling the characteristics of tooth structure was recognized during the 1930s. Fluoridation of drinking water since then has been a common practice in many countries. Despite the debate over fluoridation of drinking water, it is widely accepted that a fluoride concentration of 0.7 to 1.2 ppm (parts per million) in drinking water would be beneficial because it results in the development of a hard, decay-resistant enamel during the formation of permanent teeth. However, when fluoride concentration is over 4 ppm, it may cause the damage to tooth enamel and other potential health problems (pain and tenderness of bones, mottled teeth in children, etc.). Severe forms of the disease may develop when the fluoride concentration of drinking water is greater than 5 to 10 ppm. For example, Fluorosis, an endemic disease caused by the long-term ingestion of high fluoride drinking water, affects many millions of people, particularly children, in south Africa, Mongolia, India, Pakistan, Thailand and China. In practice, the fluoride concentration of 1 ppm in drinking water is considered the upper limit. Therefore, the goal of treating fluoride-containing well water or groundwater in this project is targeted at reducing the concentration to below 1 ppm.

An outline of the applicable systems of treatment for the drinking water containing Fluorides is reported both on the USEPA (United States Environmental Protection Agency) and AWWA (American Water Works Association), which lists the Best Available Technology as follows:

(1) Adsorption on Activated Granular Alumina (at pH 5.0-6.0) and following washing and regeneration;

(2) Reverse Osmosis;

(3) ED (Electrodialysis) and EDR (Reverse Electrodialysis)

In addition, bone charcoal has been recommended for use in the de-fluoridation of drinking water in developing countries. Among the listed best available technologies to remove fluoride, operating experiences suggest that activated alumina is the best way to reduce fluoride levels down to below 1 mg/L in large-scale treatment plants. Activated alumina is semi-crystalline inorganic adsorbent and highly selective for fluorides over other ions in water.

2. Fluoride Removal Systems

Fluoride removal with activated alumina is similar to ion exchange processes. During the process, activated alumina is first saturated with adsorbed fluoride ions, and then activated alumina will be regenerated to get ready for the treatment process again. The regeneration method using $NaOH/H_2SO_4$ has been used extensively in large-scale operation at municipal water treatment plants across the country. It is a two-step process. During the backwash cycle, activated alumina is first regenerated with sodium hydroxide and rinsed; next it is washed with a dilute sulfuric acid, and then the bed is rinsed down to the proper pH before turning back to service (or placed on stand-by mode). The following section will describe the operational processes of fluoride removal system using activated alumina in details.

The removal package unit for the Big Bear City Community Services District Fluoride Treatment Plant (which has been coordinated with Whittier Filtration) is similar to that used in the 29 Palms Water District. The treatment system is designed to process up to 1.5 million gallons of well water per day (1.5 MGD) for delivery to an existing potable

water reservoir. The treatment process is to reduce the fluoride concentration to a level below 1.0 ppm. The treatment process employs granular activated alumina with pH adjustment. The feed water pH is approximately 8.6. The optimum treatment takes place at about pH 5.5. After the fluoride is removed/reduced, the treated water pH will be readjusted to a level determined by the plant operator in the range of 7.5 to 8.2.

Fluoride removal treatment takes place in three separate parallel modules. Only two treatment modules will be in service at a given time with the third on stand-by. Each module consists of two treatment vessels (A and B). Initially, the pH adjusted feed water (pH = 5.5) flows through vessel-A and then through vessel-B in that sequence. At this sequence, vessel-A is considered in the lead position while vessel-B is in the lag position. Eventually, vessel-A becomes totally saturated, at which point the fluoride concentration of the water entering is the same as that leaving the vessel. At that moment, the treatment will only take place in vessel-B, and vessel-A is then removed from service for media regeneration. The regeneration of exhausted treatment media is an isolated process. After vessel-A is regenerated it is returned to service in the lag position, which means that the water feeding to vessel-A is the treated water from vessel-B. At that sequence, vessel-B is in the lead position. Once vessel-B media reaches break-point, it is also removed from service for regeneration and then be placed back in the lag position again following regeneration. This mode of operation provides a continuous treatment train of two vessels in series allowing the flow sequence to alternate. (Source: Drawing NO. CU-1-081, Plan set for 29 Palms Water District Fluoride Removal Water Treatment Plant). Figure 1 presents a schematic illustration for the treatment vessel module.

The pH of the treated water is controlled three ways to minimize chemical consumption. First, a predetermined fraction of the feed water bypasses treatment and is blended with the treated water to raise the pH. Second, the degassifier system will remove the carbon dioxide (CO2) in the treated water, which also raises pH. Third, caustic feeds into the degasified treated and/or blended water raises the pH to the final desired level required for potable usage. (Source: Drawing NO. CU-1-081, Plan set for 29 Palms Water District Fluoride Removal Water Treatment Plant).

The regeneration of exhausted treatment media contained in a treatment vessel is an isolated process. As described above, the isolated vessel will be initially regenerated up-flow with a 1% sodium hydroxide (NaOH) solution, flushed with well water, then regenerated down-flow with 1% NaOH solution. After the first step, the treatment media at about pH 12.8 will be neutralized with dilute sulfuric acid to pH 5.5 where fluoride adsorption takes place. Instead of separating the fluoride backwash water into two streams based on its fluoride concentration as was at the 29 Palms fluoride removal plant, all the backwash water in the project will be directed to a single backwash water reservoir. Further treatment of the backwash water will be discussed in detail in the following section.

3. Disposal Alternatives of the Fluoride Backwash Water

It is estimated that approximately 6% of backwash water is generated during the backwash and regeneration processes for saturated activated alumina media on a daily basis. The disposal of the backwash water will take into consideration of several factors, such as cost-effectiveness, technological feasibility, and site-specific environmental factors for the Big Bear area. The following four alternatives to address the treatment of backwash water will be discussed to include the inherent advantages and disadvantages.

Alternative 1 (RO treatment of backwash water with solar evaporation of RO concentrate):

Alternative 1 is to treat all the activated alumina backwash water totally with solar evaporation. Due to climatic restrictions in the Big Bear area, a small Reverse Osmosis (RO) system will be installed to treat the fluoride backwash water before discharging the concentrated salt solution to the evaporation pond(s). RO unit is to apply a pressure gradient greater than the osmotic pressure, therefore making the flow from the more

concentrated to the less concentrated region across the semi-permeable membrane. RO membranes can reject particles as small as 0.0001 μm.

In the 29 Palms plant design, the fluoride backwash water flow stream is fed with a 37% calcium chloride solution to precipitate calcium fluoride and acid to adjust pH to 8.0 at which point the solubility of calcium fluoride is at minimum. After the fluoride backwash water is thoroughly mixed, and the calcium fluoride suspended solids are allowed to settle, the slurry is then pumped to a filter press for dewatering. The Calcium Fluoride solids are disposed to landfills. The filtrate from the filter press is recycled to the high fluoride wastewater reservoir at the site. It should be noted that even after calcium fluoride precipitates, there is approximately 8 ppm calcium fluoride exists in the supernatant of the backwash water reservoir due to the CaF_2 solubility limitation.

By adding reverse osmosis (RO) system to meet the special need in the Big Bear City Project as mentioned above, the residual calcium fluoride would become concentrated on the surface of RO membrane as the water is pressed through the membrane. The locally concentrated CaF2 solution would precipitate again and cause fouling problem in the reverse osmosis system. The term "fouling" is used to describe the potential deposition and accumulation of calcium fluoride precipitate on the RO membrane. Membrane fouling is an important consideration in the operation of reverse osmosis unit as it affects pretreatment needs, operating pressure, cleaning requirements, performance and cost.

As a result of the above concerns, an alternative to reduce/remove the salt will be to treat the fluoride treatment backwash waste totally with the RO system, thus eliminating the need for the filter press. Removing calcium chloride feed system and filter press would also offset cost of the RO secondary treatment system. The permeate flow from the RO system could be delivered to the potable water reservoir or be used to blend with the "feed water" to the fluoride treatment facility. The reject flow from RO system contains concentrated sodium fluoride in smaller volume that can be dried in the evaporation ponds and collected as solid salt. Assuming that the plant will be operated for10 hours a day, it is estimated that the size of evaporation ponds needs to be about 1.5 acres in the summer months. The location of evaporation ponds is to be determined by further

discussion with District's operation staff. The flow diagram for this alternative is presented in Figure 2.

The advantages of this alternative include that (1) the fluoride treatment facility will not be relying on the BBARWA treatment plant because the only waste produced on-site is the dry sodium fluoride (NaF) collected from the evaporation ponds; and (2) nearly 99% of well water will be recovered via the total treatment system.

The disadvantage is that it requires a large evaporation pond area in order to evaporate the water from the RO system waste stream due to the low evaporation rate in the Big Bear City area. If evaporation pond capacity is not sufficient, the fluoride treatment facility running time during the cooler months (rainy season) will have to be reduced.

Alternative 2 (RO treatment of backwash water with RO concentrate discharging to BBARWA):

This alternative is to treat all the activated alumina backwash water with the Reverse Osmosis (RO) system first, then recycle the RO permeate back to the feed stream of alumina adsorption system and discharge the RO concentrate to the BBARWA facility. This alternative aims to eliminate the fluoride treatment facility's dependence on the evaporation ponds, thus being able to operate the fluoride facility even during the coolest months when the evaporation rate is at a minimal.

The advantages of this alternative include that (1) it requires a smaller site for the fluoride treatment system; (2) it reduces the BBARWA connection fee due to decreased amount of wastewater discharged to BBARWA; (3) it eliminates the dependence of the fluoride treatment operation on evaporation conditions; and (4) it achieves nearly 99% recovery of the well water.

The disadvantage for this alternative is that, in addition to the RO system, additional costs such as piping to connect to BBARWA facility via BBCCSD trunk sewer, and payment

for BBARWA connection fee, and monthly treatment cost based on flow will be required.

Alternative 3 (Filter Press treatment of backwash water and discharge of filtrate to BBARWA):

This alternative will discharge all the backwash water from activated alumina adsorption system to BBARWA after filter press treatment (exchanging sodium chloride for sodium fluoride). As at the 29 Palms fluoride treatment plant, calcium chloride ($CaCl_2$) is added to form calcium fluoride (CaF_2) precipitate that will be removed by the filter press. The waste stream coming out of filter press still contains about 8 ppm CaF_2 (solubility of CaF_2) and additional concentrations of Cl^- and other ions, such as SO_4^{2-} and Na^+. This waste stream will then be discharged to BBARWA.

The advantages of this alternative include that (1) it requires a smaller site for the fluoride treatment system as discussed in Alternative 2; and (2) it decreases the quantity of fluoride ions that will be discharged to the BBARWA treatment plant (but is replaced with chloride).

The disadvantages for this alternative is that (1) it will incur a higher connection fee for discharging waste stream to BBARWA similar to Alternative 4; and (2) this alternative has about a 94% recovery of the well water.

Alternative 4 (Directly discharging the activated alumina backwash water to BBARWA without treatment):

This alternative will discharge all the backwash water from activated alumina adsorption system to BBARWA without any additional treatment. This alternative assumes that BBARWA has no problem accepting the waste stream containing higher concentrations of fluoride ions, which will be blended with the total plant flow.

The advantages of this alternative include that (1) it requires a smaller site for the fluoride treatment system; (2) it reduces the capital investment on the fluoride system since the RO or filter press residual treatment will be eliminated; and (3) it decreases the engineering design requirement for the whole system.

The disadvantages for this alternative is that (1) it will incur a higher connection fee for discharging the waste stream to BBARWA; (2) the fluoride treatment operation will depend heavily on acceptance of activated alumina backwash waste stream by BBARWA; and (3) this alternative has about a 94% recovery of the well water.

4. Plant Layout

The basic fluoride treatment facility at BBCCSD is similar to the layout at 29 Palms Water District. One of the concerns in the layout is the size of building that will house the modular treatment units. Fluoride treatment plant in 29 Palms Water District has the design capacity at 3 MGD; while in Big Bear City, the design capacity is 1.5 MGD. However, the building size will not necessarily reduce by half. A preliminary comparison is made between the sizes of the equipments as shown in the Table 1.

Table 1

	(29 Palms Water District) Dimensions	(Big Bear City CSD) Dimensions
Total Flow Rate	3 MGD (2,100 GPM)	1.5 MGD (1,000 GPM)
Adsorbing Vessel Dia. (6)	11'-0"	8'-0"
Degassifier Diameter (1)	10'-0"	7'-0"
$CaCl_2$ Storage Tank Volume	6,700 Gal	Not Used (for Alternatives 1, 2 & 4)
H_2SO_4 Storage Tank Volume	6,700 Gal	6,700 Gal
NaOH Storage Tank Volume	6,700 Gal	6,700 Gal

The proposed chemical storage tank sizes are identical to those in the 29 Palms Plant even though the total treatment flow rate at BBCCSD is at 50%. The reason is that it is generally desirable to have a large enough tank to handle a full shipment of chemical, which is generally around 5,000 gals. The 6,700-gallon tanks are capable of storing a full shipment before the tank is completely empty. Based on the initial review, the size of facilities and building are as presented in Table 2.

Table 2

	Dimensions (29 Palms Water District)	Dimensions (Big Bear City CSD)
Treatment Building	138'×60'	115'×55'
$CaCl_2$ Storage Tank	16' ×4'	Not Used
H_2SO_4 Storage Tank	16' ×4'	16' ×4'
NaOH Storage Tank	16' ×4'	16' ×4'
Treated Water Reservoir	96'×24' (200,000 Gal)	To Existing Reservoir
High-Fluoride Waste Reservoir	24'×24' (50,000 Gal)	24'×24' (50,000 Gal)
Low-Fluoride Waste Reservoir	96'×24' (200,000 Gal)	Not Used
RO Treated Water Reservoir (For Alternative 1 & 2 Only)	None	24'×24' (50,000 Gal) (To blend with well water)

The proposed BBCCSD fluoride treatment facility layout is shown in Figure 3. As discussed earlier, if alternative 1 is adopted, the size and location of evaporation is to be determined on next design phase. The access road will be an extension of Maltby Boulevard from the west. This is strictly a concept design. The final site design will take into consideration access driveway, building foundation factor (from geotechnical investigation), groundwater depth, etc. The spatial arrangement between the treatment train and ancillary facilities such as office, electrical control room, and laboratory will be

thoroughly discussed with District's operations staff before commencing on the detailed design.

5. Comparison & Summary

When discharging wastewater to BBARWA facility for treatment, regional BBARWA fee at $2576.71/EDU must be considered based on flow and sodium fluoride concentration. A wastewater flow coefficient of 180 Gallons per EDU is used to determine the total BBARWA connection fee for the fluoride waste stream based on the review of BBARWA Sewer System Analysis Report. It should also be noted that currently BBARWA facility has a wastewater treatment capacity of 3.6 MGD. Therefore, it should have no capacity problem for accepting fluoride treatment waste stream. Table 3 lists connection fee for all the above alternatives.

Table 3

10 hrs x 7 Days

Alternative	Waste Stream Flow to BBARWA (Gal/Day)	EDU Equivalent *	BBARWA Connection Fees
1	0	0	$ 0
2	7,200	40	$ 103,068
3	36,000	200	$ 515,342
4	36,000	200	$ 515,342

* EDU Equivalent = 180 GPD/EDU

The concentrations of various ions (F^-, Na^+, HCO_3^-, SO_4^{2-}, Cl^-, etc) in the waste stream discharging to BBARWA facility vary under each alternative. The inorganic ions and pH in the waste stream for Alternative 2 for example are estimated as follows:

PH ≈7.7;

[F-] ≈500 ppm;

[Na+] ≈900ppm;

[HCO3-] ≈ 350 ppm

[SO4-] ≈ 400 ppm

[Cl-] ≈ 50 ppm

The waste stream of Alternative 3 has about 8 ppm fluoride ion and up to 500 ppm Cl^- ion; other ion concentrations remain the same. Since Alternative 4 has no treatment for the activated alumina backwash water, ionic concentration in the waste stream should be decreased. It should be noted that even though the total quantity of wastewater and their correspondent ion concentrations vary for each alternative, the total amount of ions discharging to BBARWA would be very close. Also, the waste stream from fluoride treatment facility will be mixed with the total wastewater flow to the BBARWA treatment plant. Therefore, the impact of various inorganic ions from the fluoride treatment waste stream to the BBARWA facility will be greatly minimized through mixing and dilution.

The estimated total project cost for each alternative is presented in Table 4. A breakdown of the capital cost estimate is presented in Appendix 1. The overall project cost consists of two parts. The first one is BBARWA connection fee while the second part is the capital cost for the fluoride treatment system. It should be noted that the capital costs could experience a 15% to 20% upward fluctuation depending on when the BBCCSD bids the project and changes in market price for steel and concrete.

Table 4

Alternative	Backwash water treatment	Evaporation Pond	BBARWA Connection Fees	Estimated Fluoride Treatment Cost	Estimated Total Project Cost	Well Water Recovery Percent
1	RO	YES	$ 0	$ 7,181,850	$ 7,181,850	99 % ±
2	RO	NO	$ 103,068 *	$ 6,925,850	$ 7,028,918	99 % ±
3	Filter Press	NO	$ 515,342 *	$7,424,350	$ 7,939,692	94 % ±
4	N/A	NO	$ 515,342 *	$ 6,120,600	$ 6,635,942	94 % ±

Appendix 1: Probable Cost Estimates

Alternative 1 (RO treatment of backwash water with solar evaporation of RO concentrate):

(1)	Fluoride removal system equipment (Package FOB to job site)	$1,900,000
	Equipment installation and mics. piping allowance (35% to 40%)	$ 760,000
(2)	Sodium hydro-oxide storage concrete pad/containment area	$ 23,400
	($15,600 × 1.5; Storage tank included in item #1)	
	Sulfuric acid storage concrete pad/containment area	$ 23,400
	($15,600 × 1.5; Storage tank included in item #1)	
(3)	Basic building cost (115'× 55'= 6,325 SF at $150/SF)	$ 948,750
(4)	Reverse Osmosis treatment unit for NaF removal	$ 278,000
	(Based on Dynatec System with 100,000 GDP and 80% Recovery)	
	RO Unit installation and mics. piping allowance (≥50%)	$ 150,000
(5)	Sodium hypochlorite feed system	$ 50,000
(6)	Site work and pavement allowance	$ 337,800
	- Concrete lined evaporation pond = $256,000	
	- Allowance for general clearing grading of site = $15,000	
	- Allowance for pavement (4-inch AC over 5-inch base) = $66,800	
(7)	Two 50,000 Gallons underground storage tanks (12ft ± deep)	$ 190,000
(8)	Electrical System (power distribution + controls)	$ 262,000
	(15%×$2,275,000)	
	Allowance for standby generator	$ 200,000
(9)	Well #3B equipping with building	$ 600,000
	Subtotal:	$ 5,723,350
(10)	Allowance for 15% project construction contingency	$ 858,500
	Allowance for engineering, district administration & construction	$ 600,000

Estimated Total Probable Project Cost **$ 7,181,850**

Alternative 2 (RO treatment of backwash water with RO concentrate discharging to BBARWA):

Items that save cost as compared to Alternative 1	Decrease in Cost
Item 1: Saving on evaporation ponds	$ 256,000
Total Cost Saving:	$ 256,000

Therefore, the capital project cost for alternative 2 = $ 7,181,850 - $ 256,000

= $ 6,925,850

Alternative 3 (Filter Press treatment of backwash water and discharge of filtrate to BBARWA):

Items that save cost as compared to Alternative 1	Decrease in Cost
Item 1: Saving on RO Units	$ 438,000
Item 2: Possible saving on building	$ 60,000
Item 3: Saving on evaporation ponds	$ 256,000
Saving Subtotal:	$ 754,000
Item 4: Saving on construction contingency & engineering	$ 188,500
Total Cost Saving:	$ 942,500

The price for filter press is about $ 1,185,000 as proposed by Whittier Filtration.

Therefore, the capital project cost for alternative 3 = $7,181,850 - $942,500 + $1,185,000

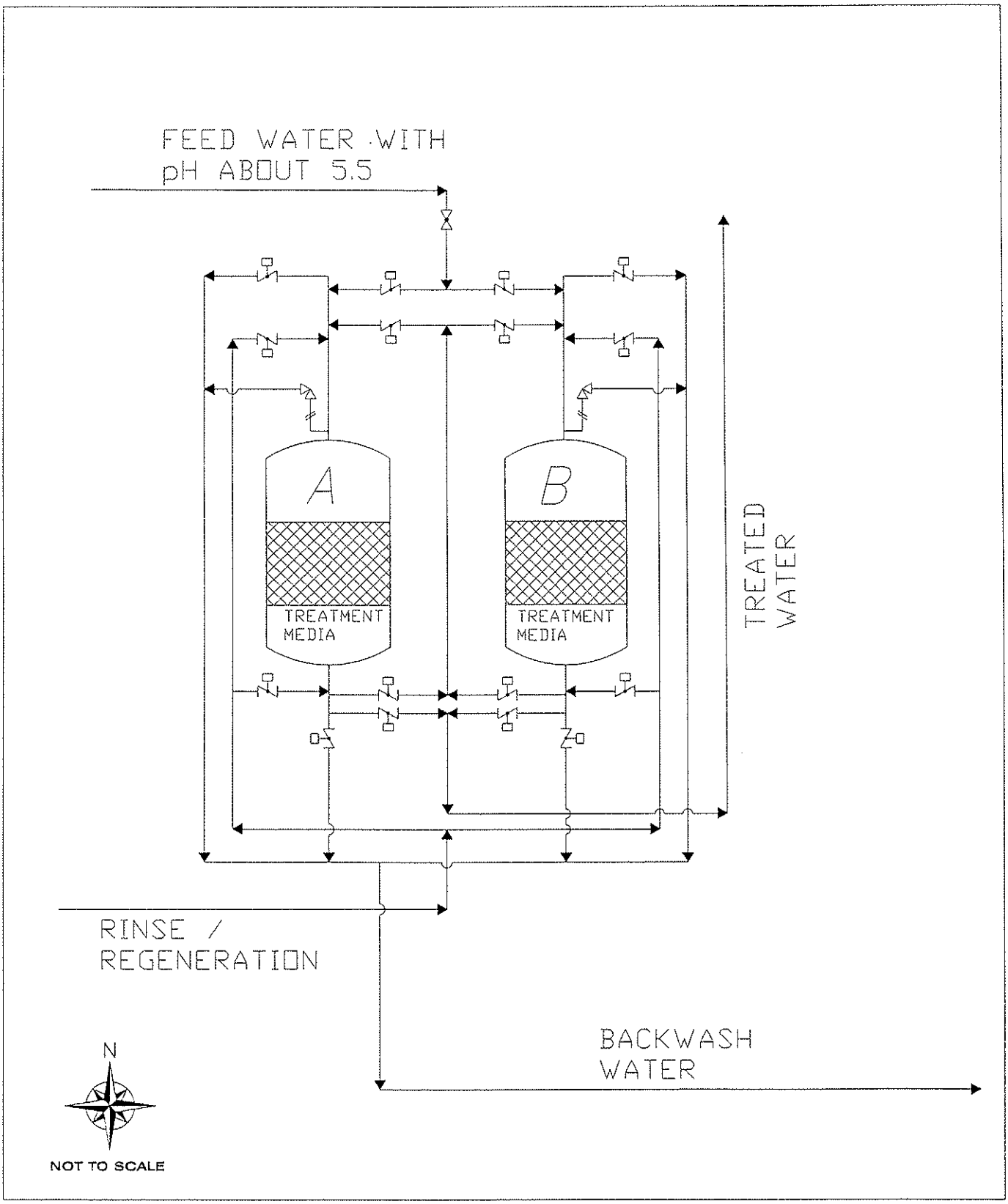
= $7,424,350

Alternative 4 (Directly discharging the activated alumina backwash water to BBARWA without treatment):

Items that save cost as compared to Alternative 1	Decrease in Cost
Item 1: Saving on RO Units	$ 438,000
Item 2: Saving on one 50,000 underground storage tanks	$ 95,000
Item 3: Possible saving on building	$ 60,000
Item 4: Saving on evaporation ponds	$ 256,000
Saving Subtotal:	$ 849,000
Item 5: Saving on construction contingency & engineering	$ 212,250
Total Cost Saving:	$ 1,061,250

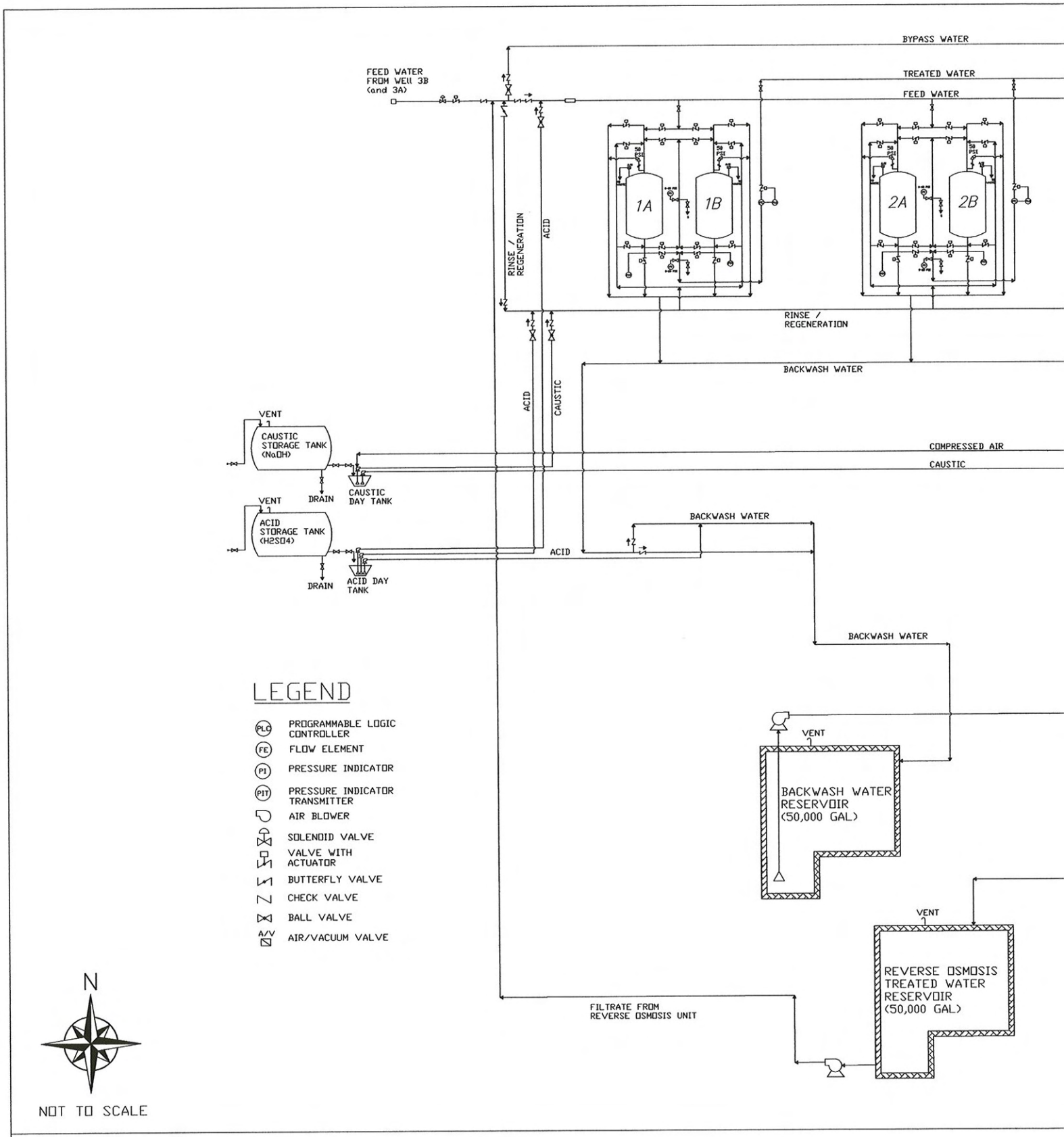
Therefore, the capital project cost for alternative 4 = $ 7,181,850 - $ 1,061,250

= **$ 6,120,600**



PREPARED BY:
So & Associates
Engineers Inc.

FIGURE 1
BIG BEAR CITY COMMUNITY SERVICES DISTRICT
WELL No. 3B FLUORIDE TREATMENT PLANT
TREATMENT VESSEL MODULE

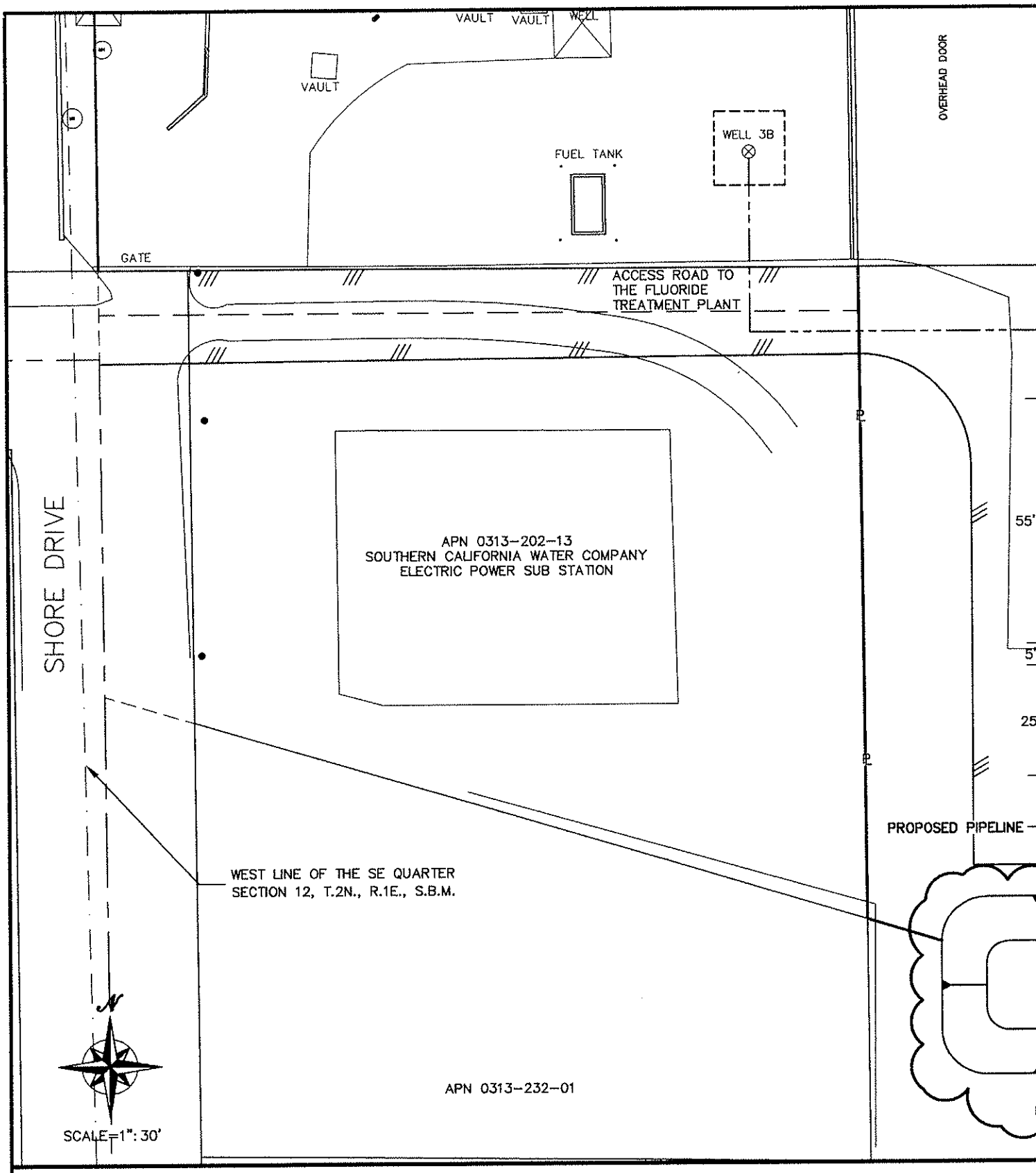


PREPARED BY:

So & Associates Engineers Inc.

* To be discussed / verified.

The above probable project costs do not include the O&M cost and monthly treatment cost to BBARWA. As far as O&M cost is concerned, the major cost for RO system is the membrane replacement and power consumption. Considering the RO treatment of backwash water, the operating costs include $33,000/year on membrane replacement and 76,000 kWH power consumption (assuming 10 hours operation/day). The relatively small cost of piping to connect to BBARWA for Alternative 2, 3 and 4 could be added in the future design phase if any of these alternatives is selected.



PREPARED BY:

So & Associates Engineers Inc.

Appendix I

Pipeline CIP Maps

0 100 200 400 600

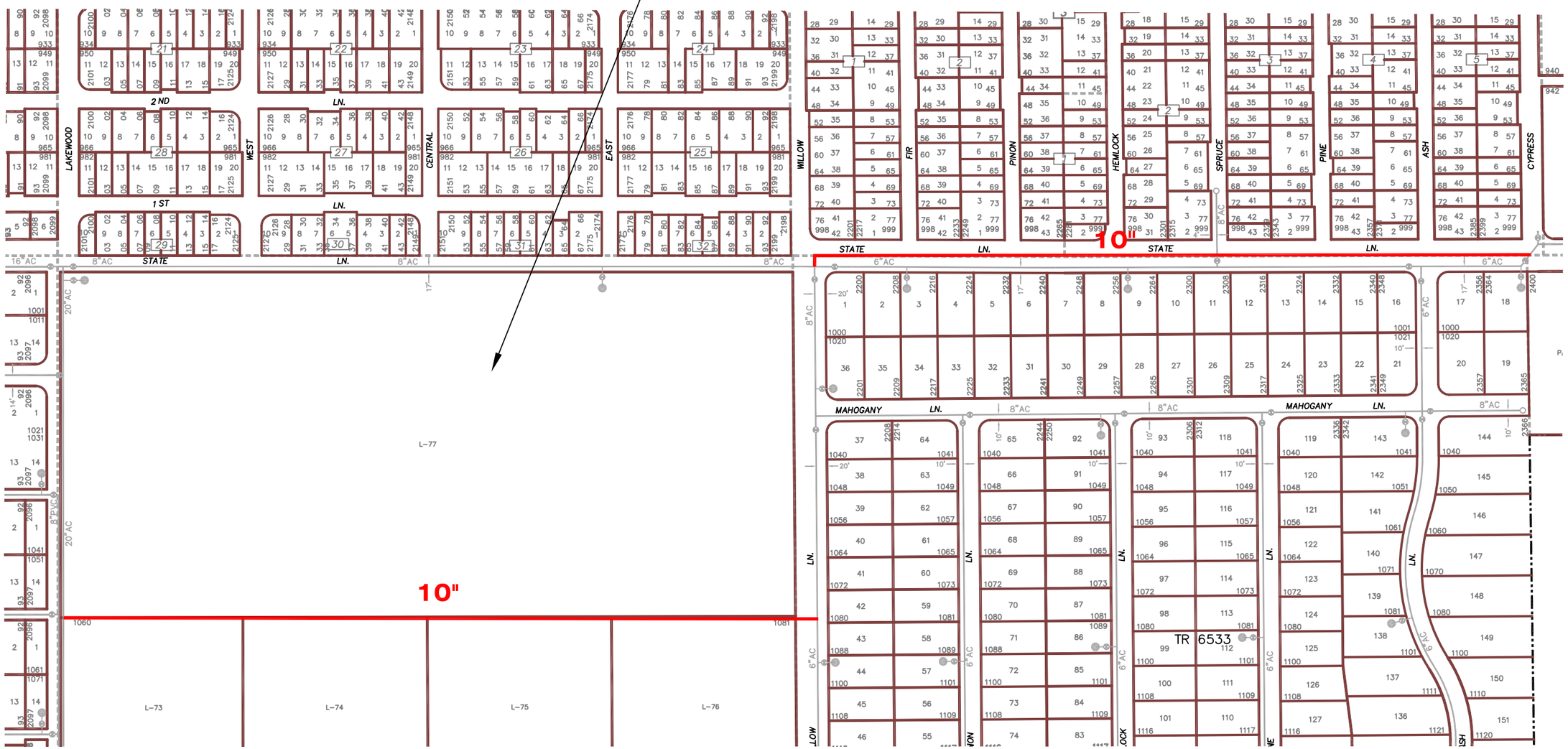
SCALE FEET

SCALE: 1"=200'

PROPOSED PIPELINE

PROPOSED PARALLEL PIPELINE

PROJECTED MEDIUM DENSITY
SINGLE FAMILY RESIDENTIAL (RS-20M)



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2008 WATER MASTER PLAN

PROJECT 1C –STATE

FIGURE

CIP.1C

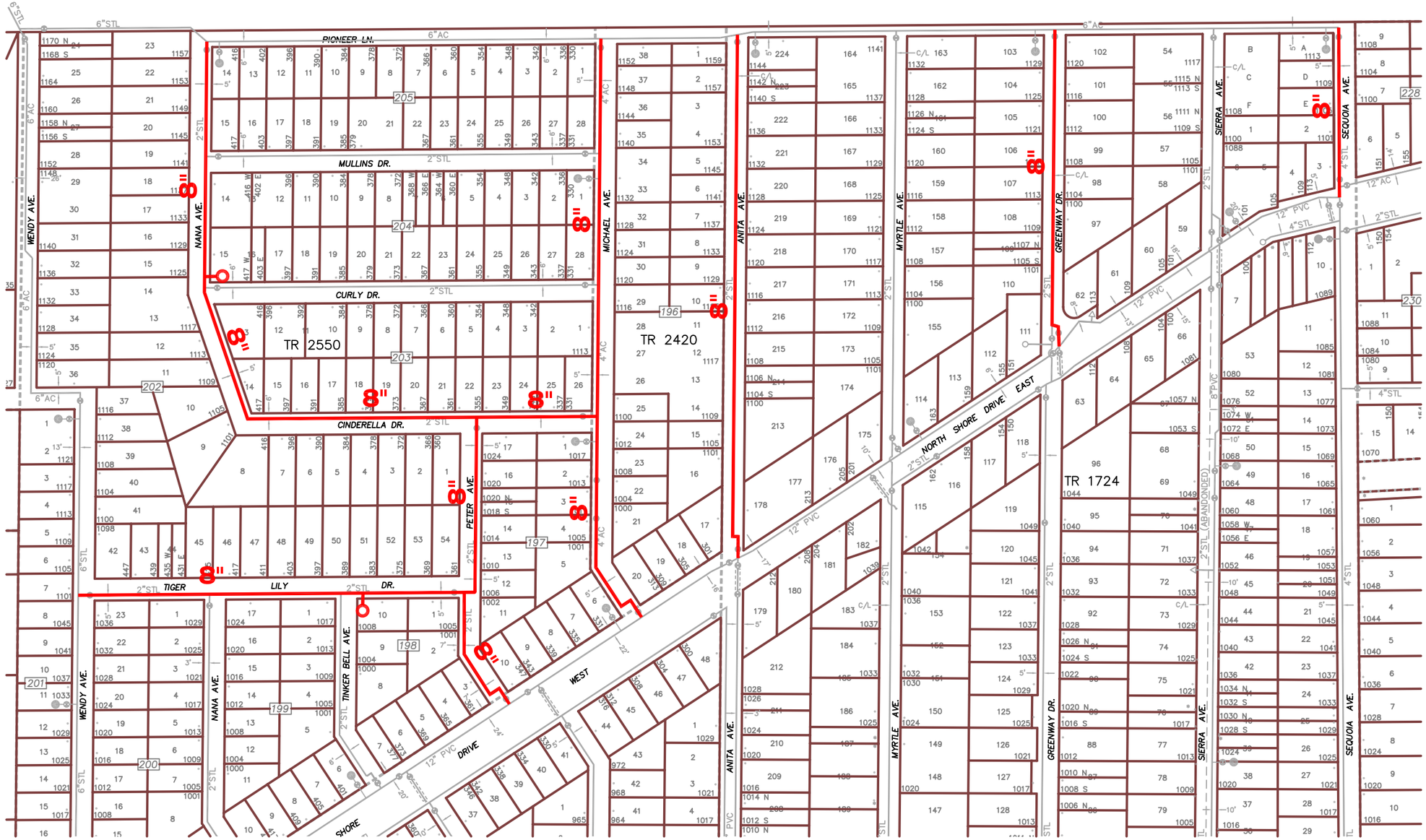
SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

NEW HYDRANT

PROPOSED REPLACEMENT PIPELINE



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2008 WATER MASTER PLAN

PROJECT 2C – MICHAEL

FIGURE

CIP.2C

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

NEW HYDRANT

PROPOSED REPLACEMENT PIPELINE

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PROJECT 3C – CINDERELLA

FIGURE

CIP.3C

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

NEW HYDRANT

PROPOSED REPLACEMENT PIPELINE

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PROJECT 4C – GREENWAY

FIGURE

CIP.4C

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED PIPELINE

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2008 WATER MASTER PLAN

PROJECT 5C – FERN

FIGURE

CIP.5C

NEW HYDRANT

PROPOSED REPLACEMENT PIPELINE

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2008 WATER MASTER PLAN

PROJECT 6C – DIVISION

FIGURE
CIP.6C

SCALE: 1"=200'

0 100 200 400 600

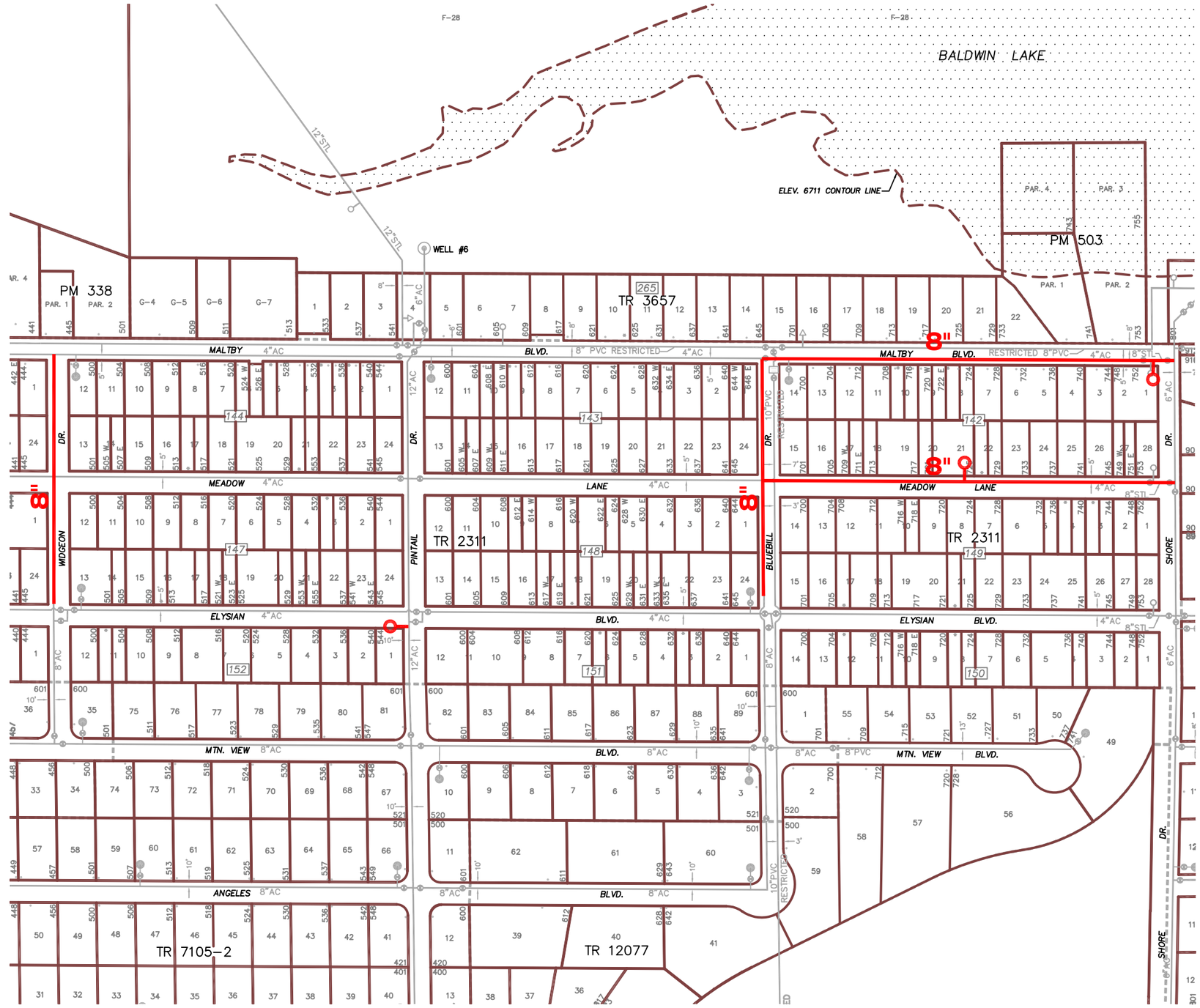
SCALE FEET

NEW HYDRANT

PROPOSED REPLACEMENT PIPELINE

PROPOSED PARALLEL PIPELINE

6"AC



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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 7C – MALTBY

FIGURE

CIP.7C

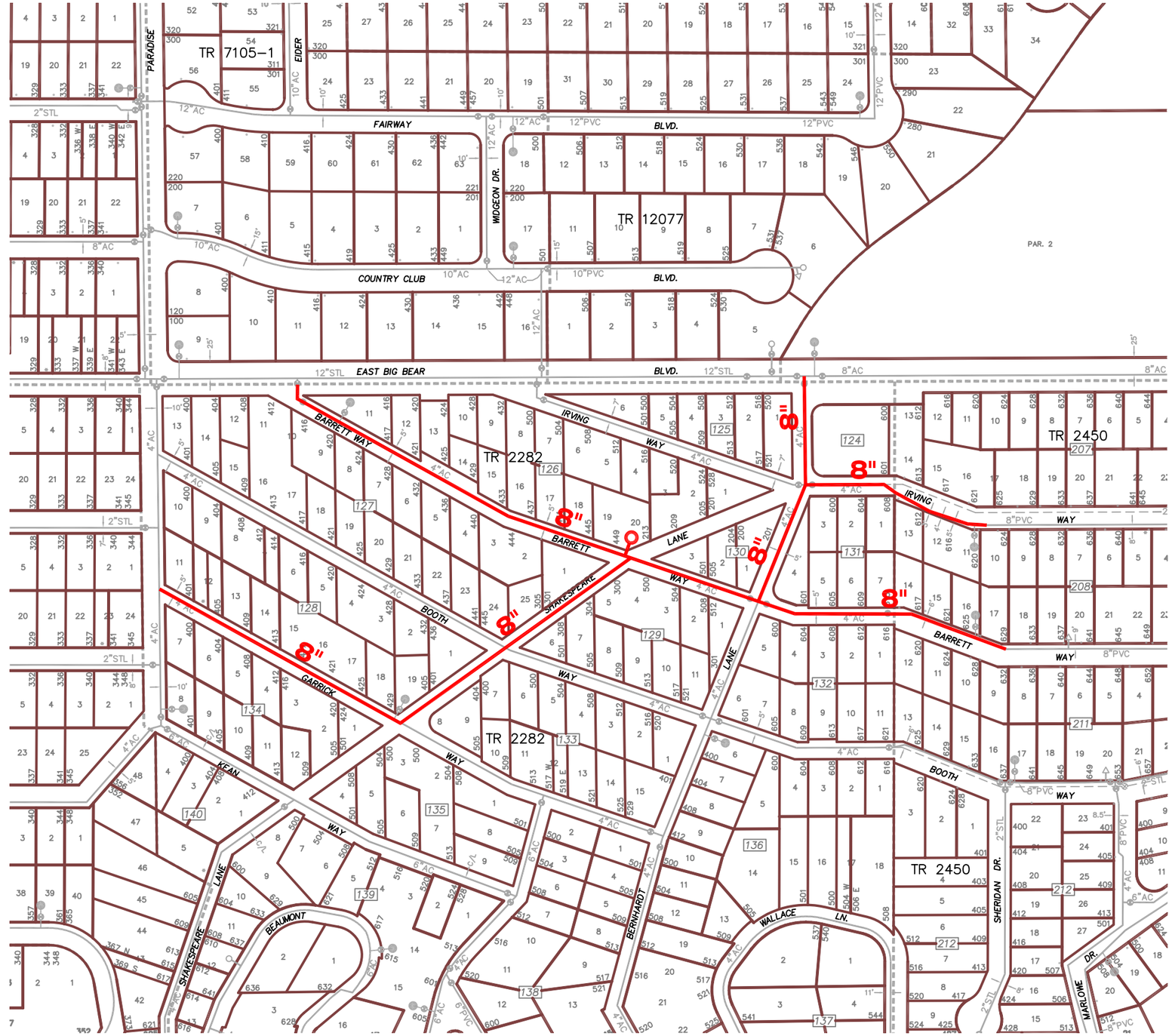
SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

NEW HYDRANT

PROPOSED REPLACEMENT PIPELINE



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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 8C – BARRETT

FIGURE

CIP.8C

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED REPLACEMENT PIPELINE

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2008 WATER MASTER PLAN

PROJECT 9C – MTN. VIEW

FIGURE

CIP.9C

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

CLOSED ZONE VALVE

NEW OR RECONFIGURED HYDRANT

PROPOSED REPLACEMENT PIPELINE

PROPOSED PARALLEL PIPELINE

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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 1G – COHILA, PHASE 1

FIGURE

CIP.1G

CLOSED ZONE VALVE

NEW OR RECONFIGURED HYDRANT

PROPOSED REPLACEMENT PIPELINE

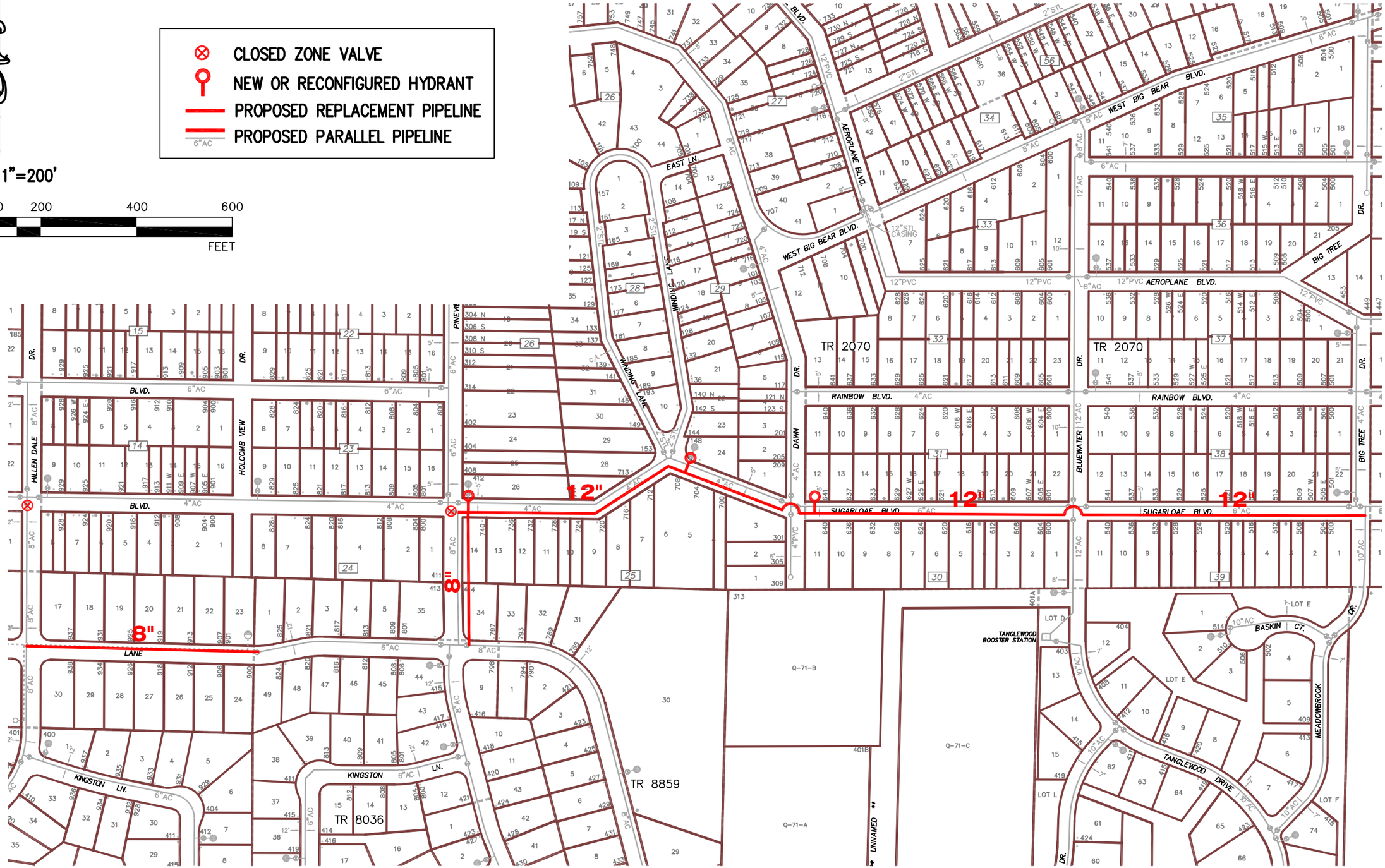
PROPOSED PARALLEL PIPELINE

6"AC

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET



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2008 WATER MASTER PLAN

PROJECT 2G – COHILA, PHASE 2

FIGURE

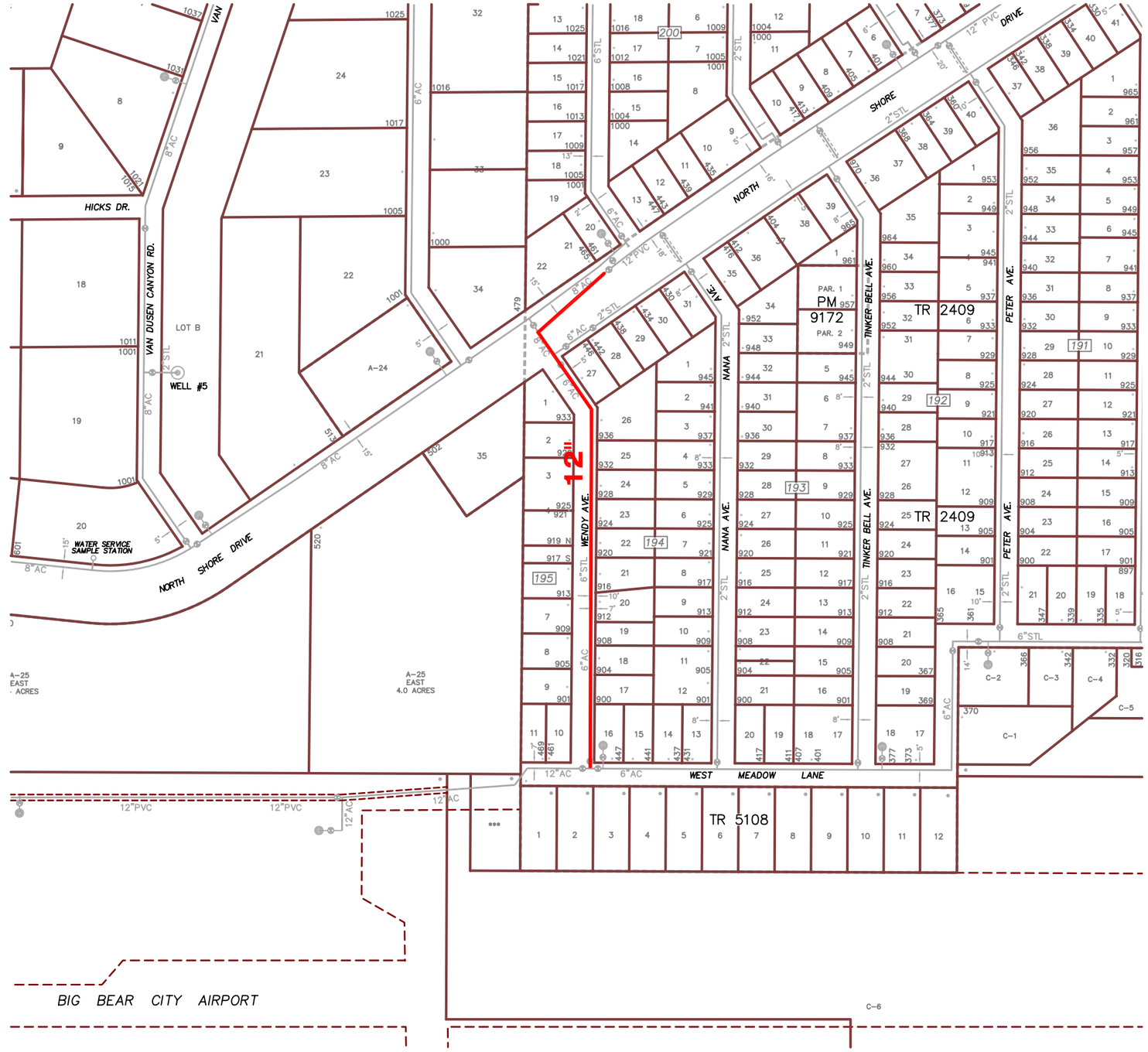
CIP.2G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED REPLACEMENT PIPELINE



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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 3G – WENDY

FIGURE

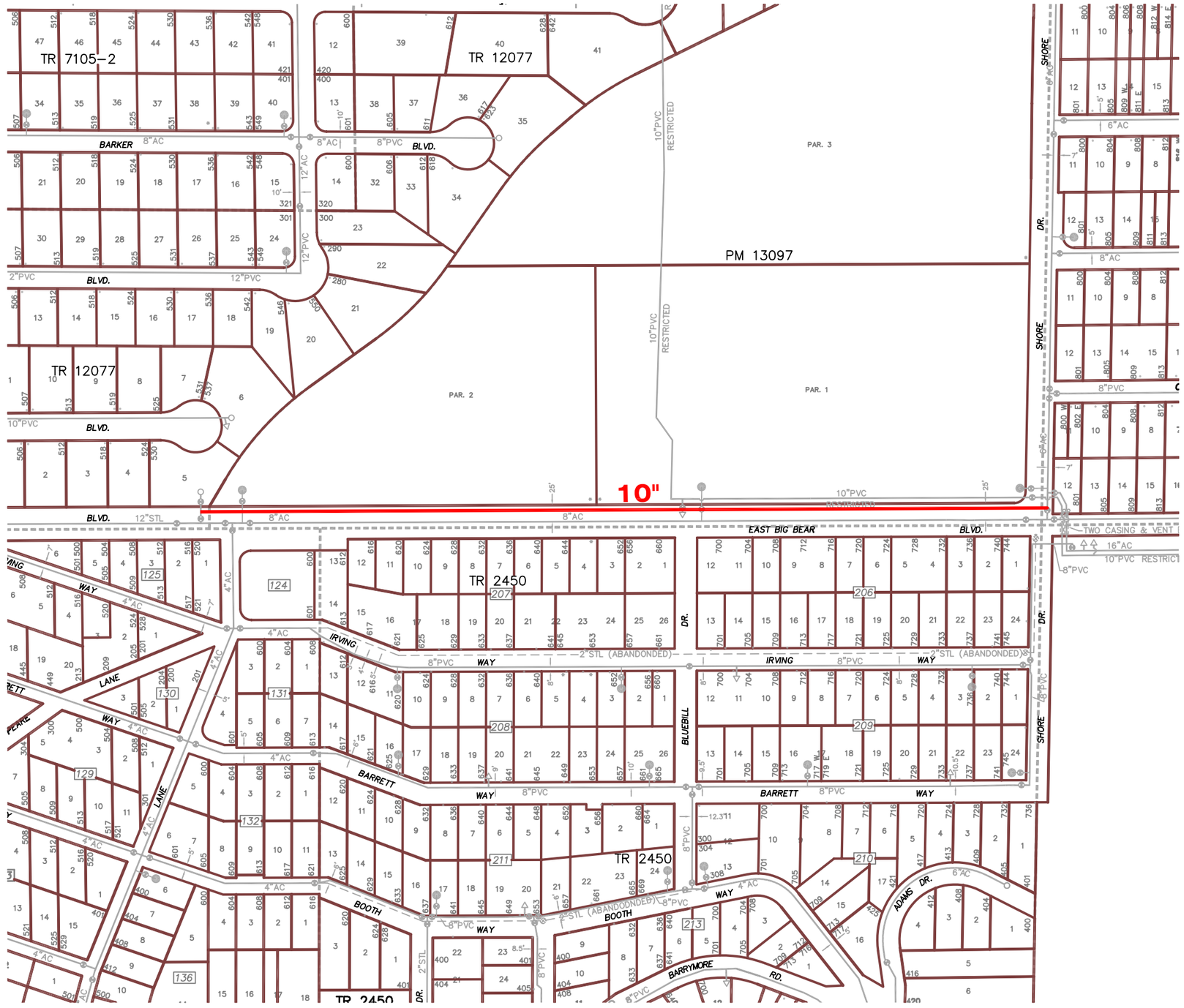
CIP.3G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED PARALLEL PIPELINE



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2008 WATER MASTER PLAN

PROJECT 4G – BIG BEAR

FIGURE

CIP.4G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED REPLACEMENT PIPELINE

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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 5G – MEADOW

FIGURE

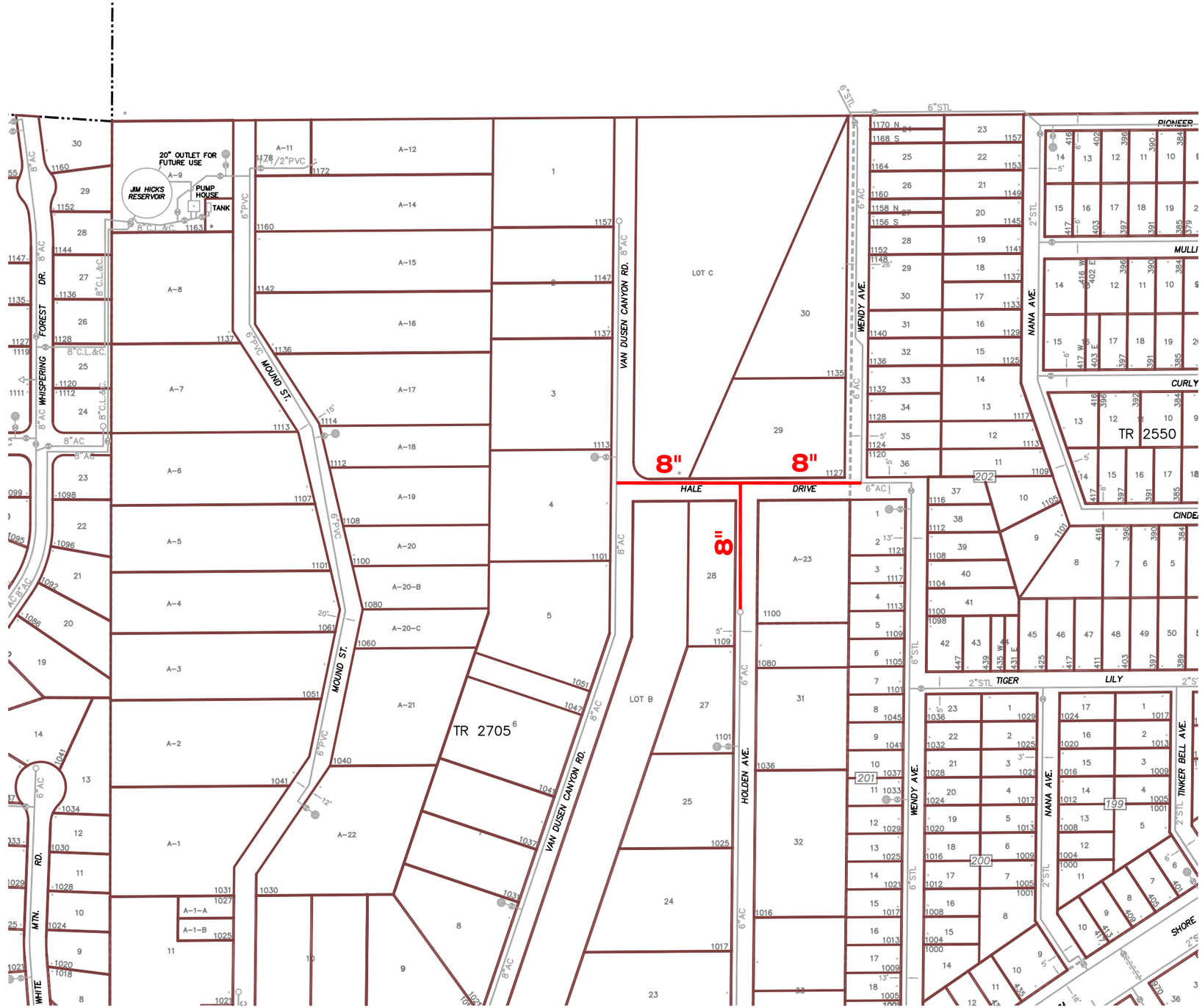
CIP.5G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED PIPELINE



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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 6G – HALE

FIGURE

CIP.6G

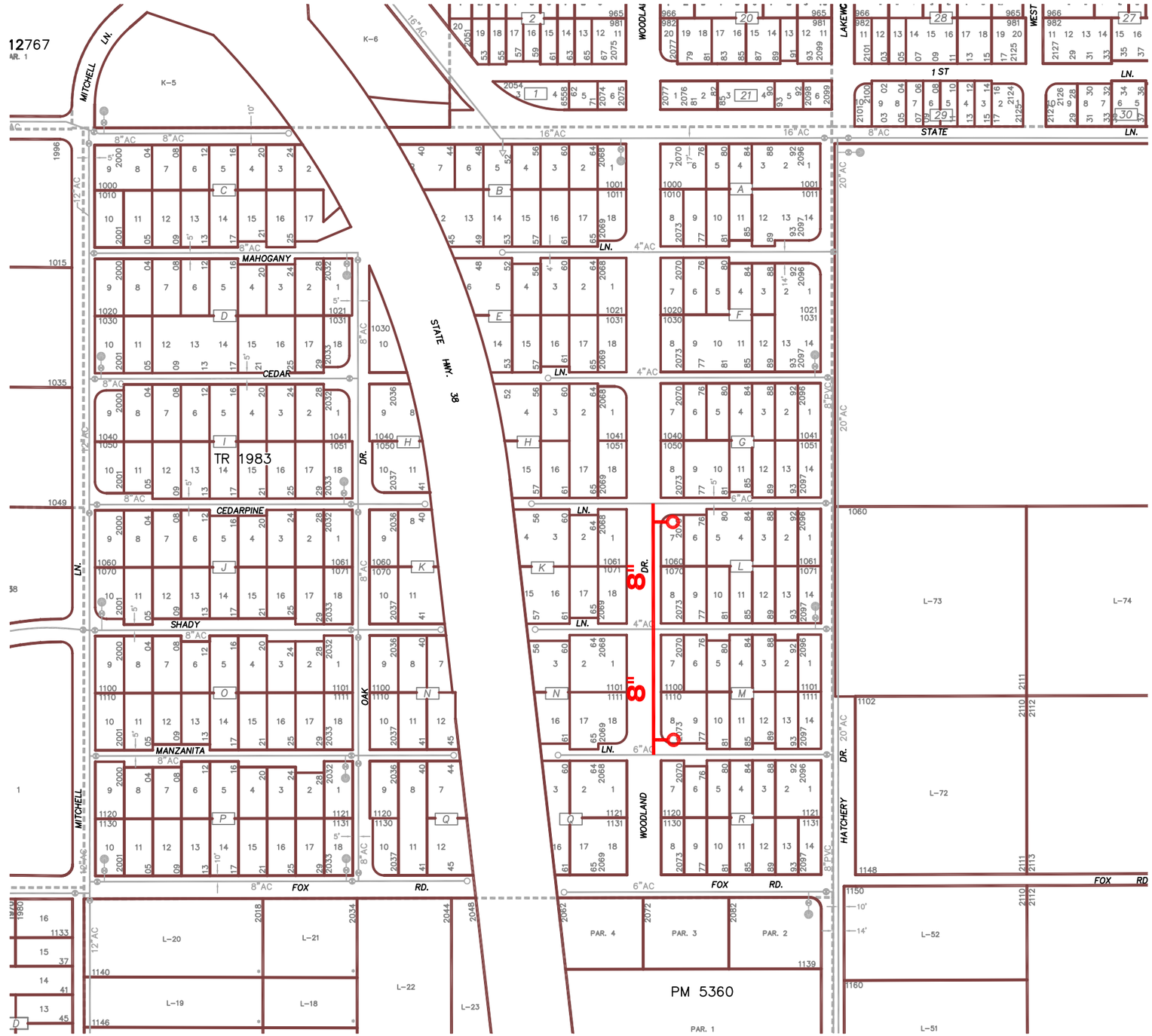
SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

NEW HYDRANT

PROPOSED PIPELINE



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BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 7G – WOODLAND

FIGURE

CIP.7G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED REPLACEMENT PIPELINE

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

CIVILTEC engineering inc.

General Civil, Municipal, Water and Wastewater Engineering, Planning, Construction Management and Surveying

118 West Lime Avenue
Monrovia, Ca. 91016
Phone: (626) 357-0588
Fax: (626) 303-7957

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 8G – ARBOR

FIGURE

CIP.8G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED REPLACEMENT PIPELINE

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

CIVILTEC engineering inc.

General Civil, Municipal, Water and Wastewater Engineering, Planning, Construction Management and Surveying

118 West Lime Avenue
Monrovia, Ca. 91016
Phone: (626) 357-0588
Fax: (626) 303-7957

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 9G – FAIRWAY

FIGURE

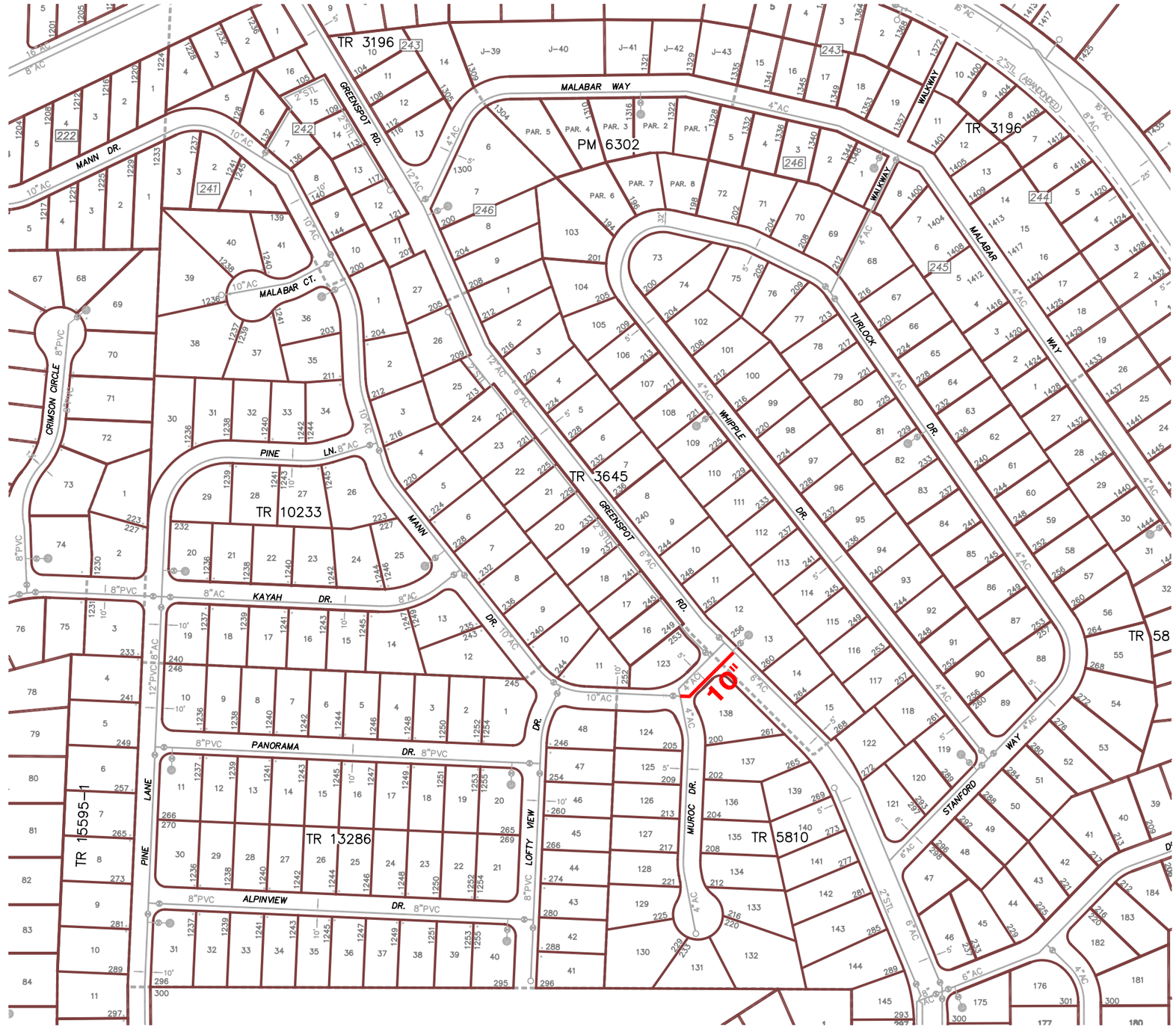
CIP.9G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

6"AC PROPOSED PARALLEL PIPELINE



BIG BEAR CITY COMMUNITY SERVICES DISTRICT

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Phone: (626) 357-0588
Fax: (626) 303-7957

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 10G – GREENSPOT

FIGURE

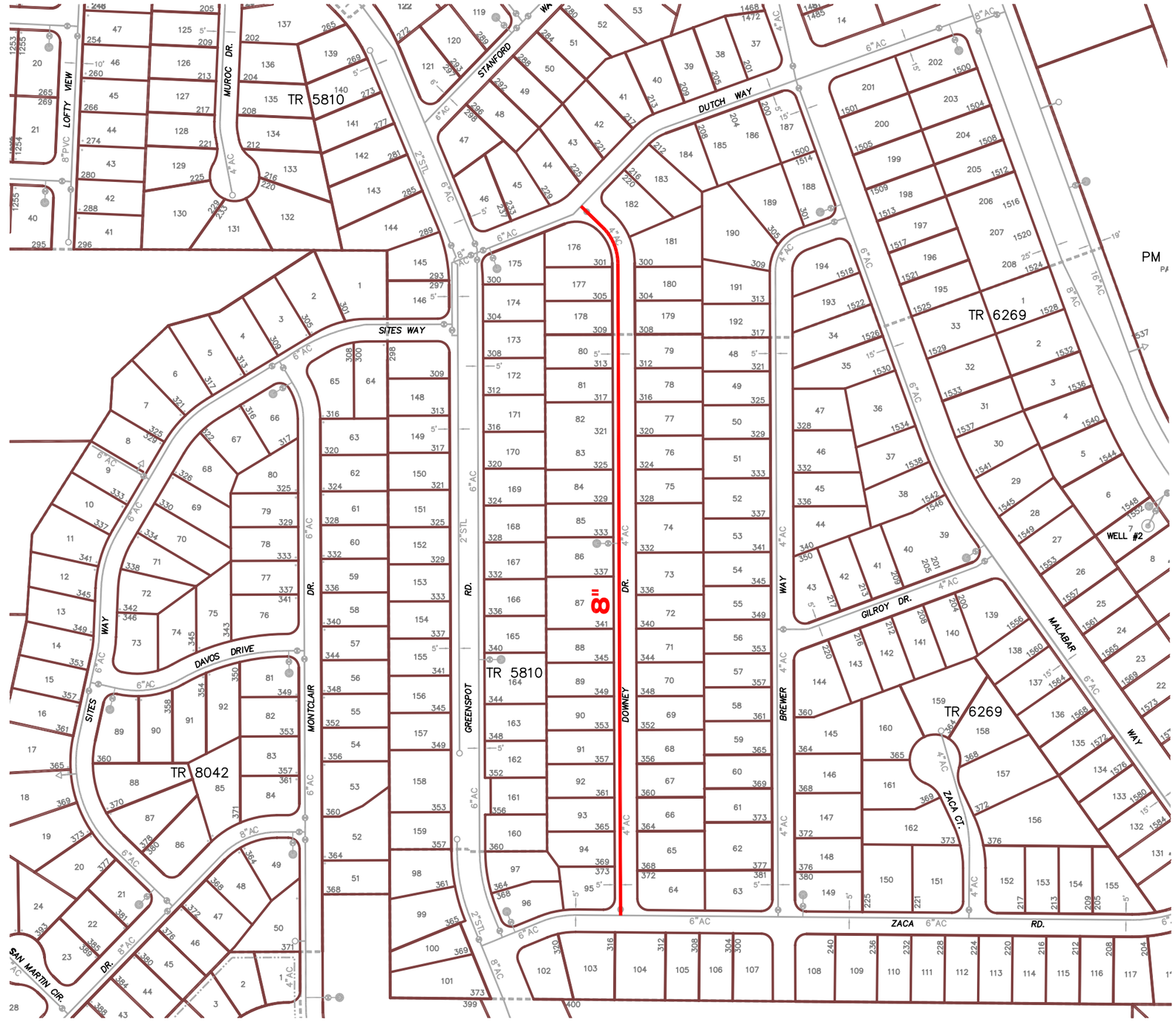
CIP.10G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED REPLACEMENT PIPELINE



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118 West Lime Avenue
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Phone: (626) 357-0588
Fax: (626) 303-7957

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 11G – DOWNEY

FIGURE

CIP.11G

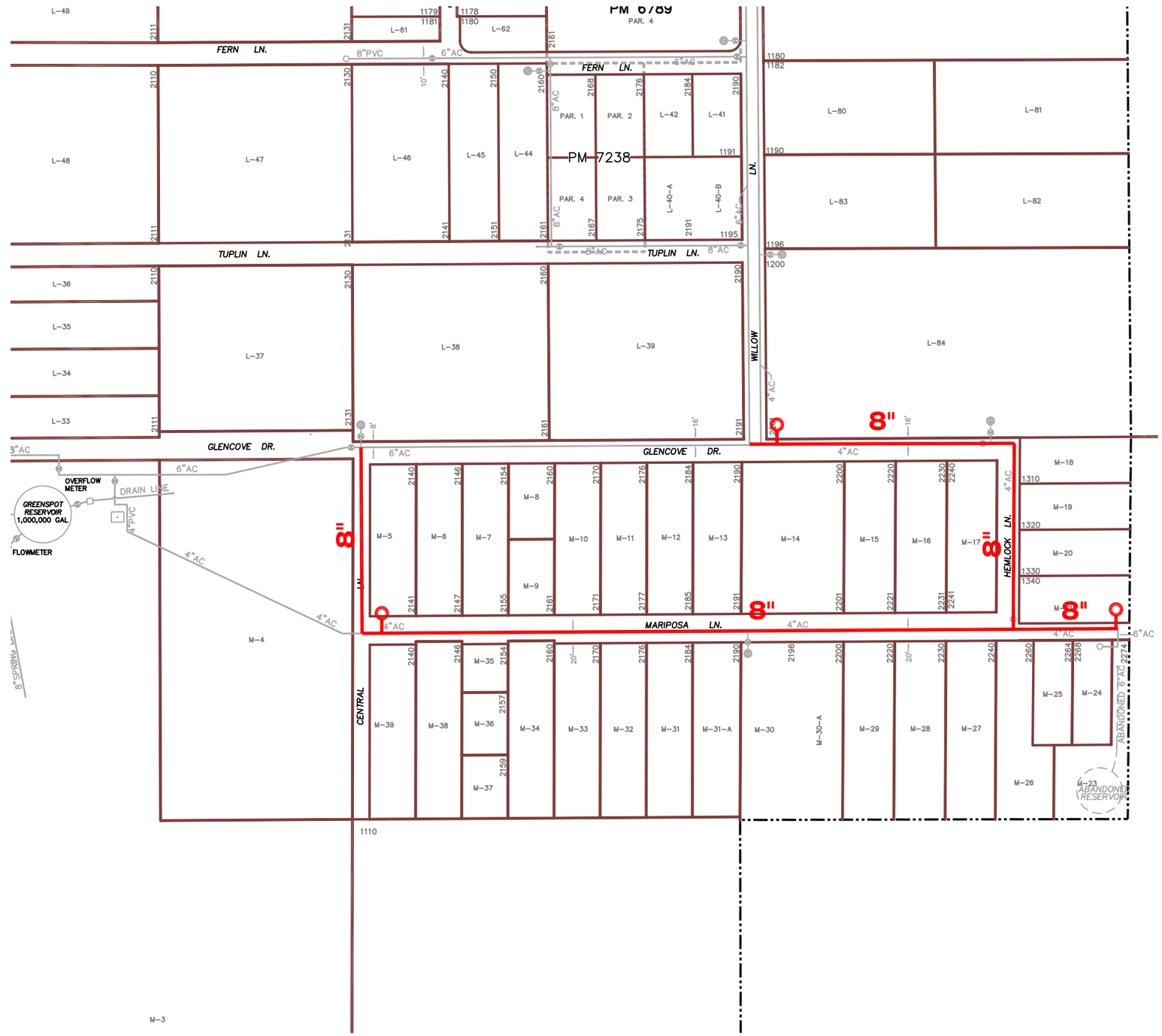
SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

NEW HYDRANT

PROPOSED REPLACEMENT PIPELINE



BIG BEAR CITY COMMUNITY SERVICES DISTRICT

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General Civil, Municipal, Water and Wastewater Engineering, Planning, Construction Management and Surveying

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Monrovia, Ca. 91016
Phone: (626) 357-0588
Fax: (626) 303-7957

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 12G – MARIPOSA

FIGURE

CIP.12G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED REPLACEMENT PIPELINE

General Civil, Municipal, Water and Wastewater Engineering, Planning, Construction Management and Surveying

CIVILTEC engineering inc.

118 West Lime Avenue
Monrovia, Ca. 91016
Phone: (626) 357-0588
Fax: (626) 303-7957

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 13G – MALABAR

FIGURE

CIP.13G

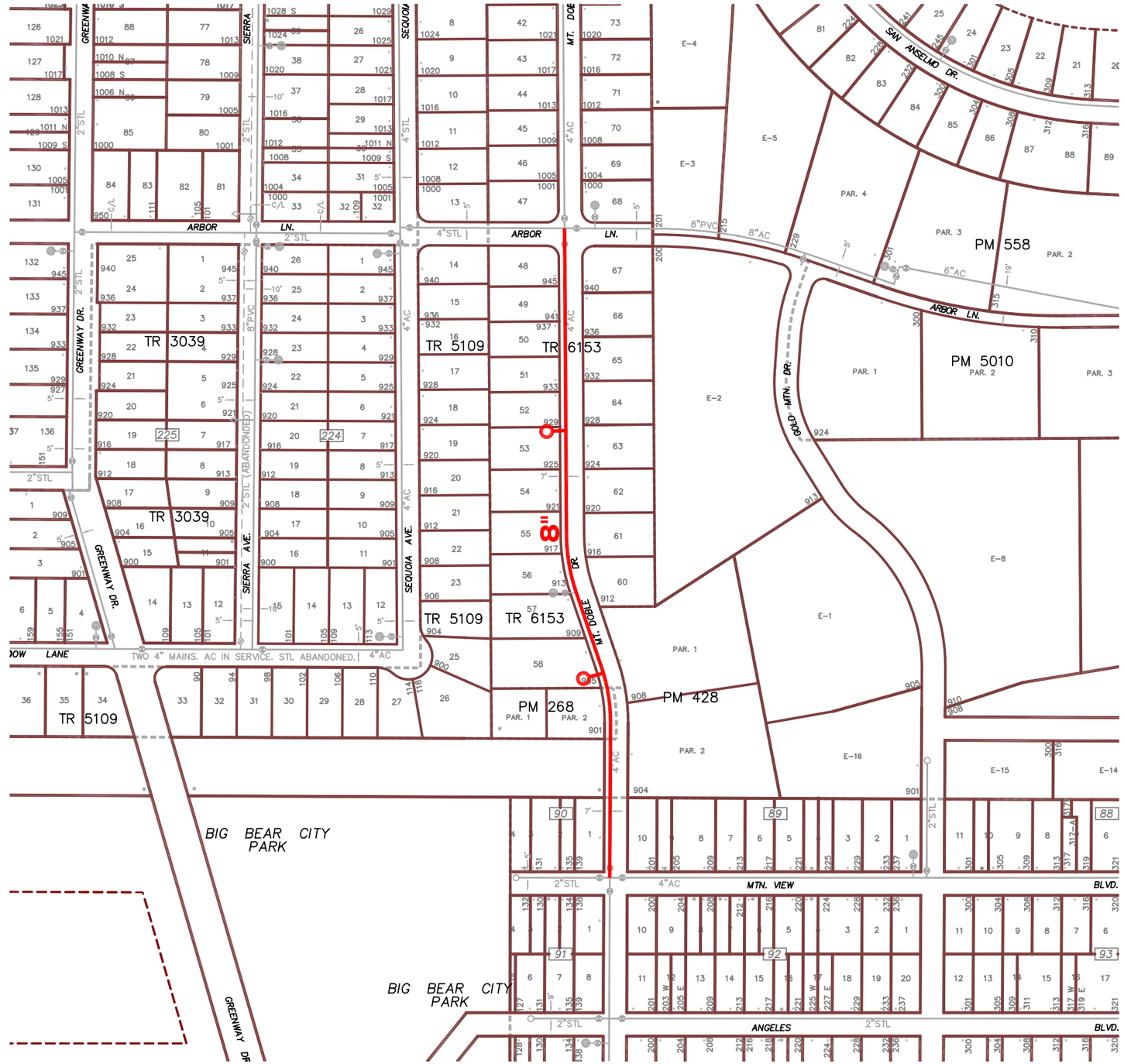
SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

NEW HYDRANT

PROPOSED REPLACEMENT PIPELINE



General Civil, Municipal, Water and Wastewater Engineering, Planning, Construction Management and Surveying

CIVILTEC engineering inc.

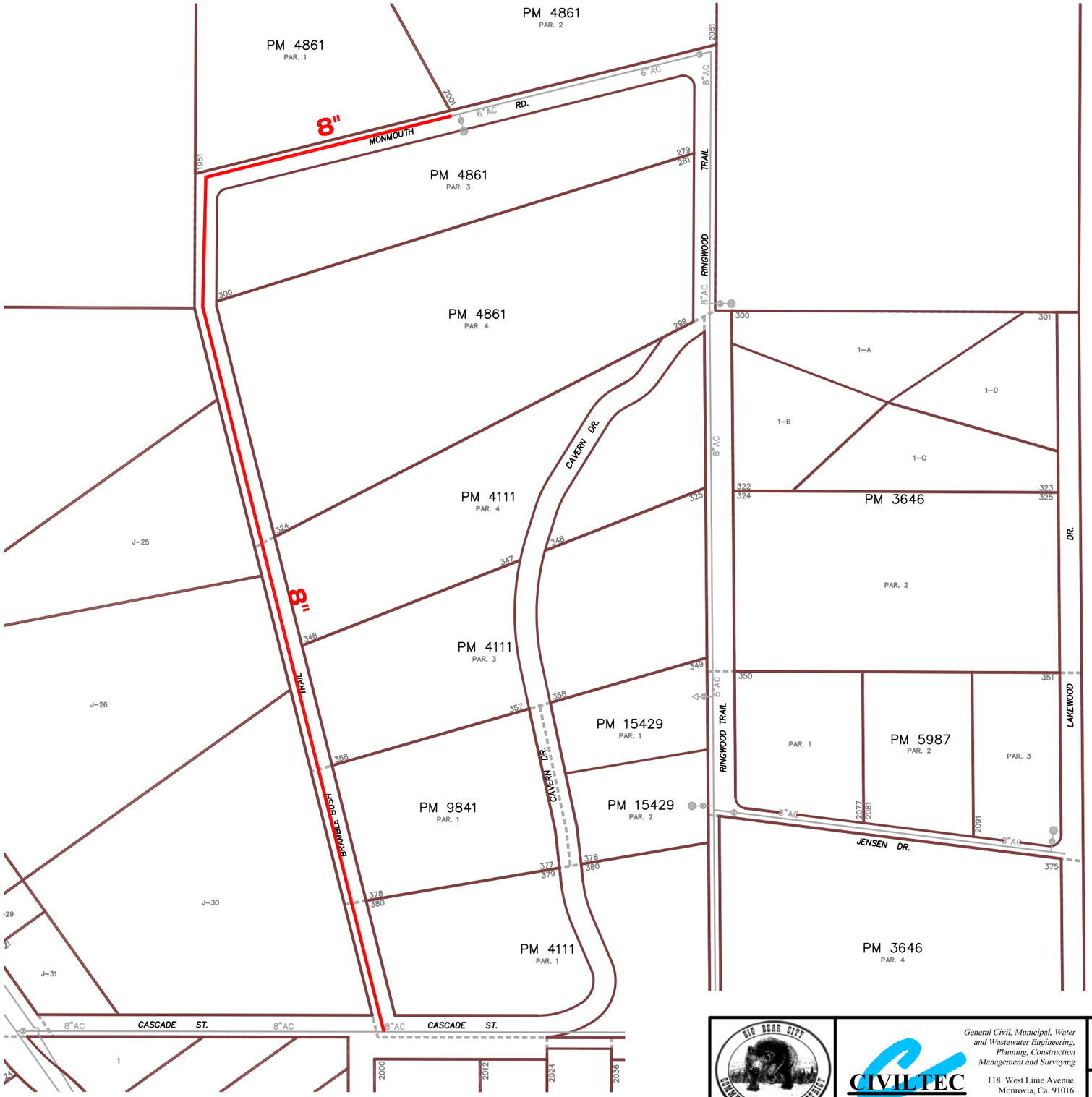
118 West Lime Avenue
Monrovia, Ca. 91016
Phone: (626) 357-0588
Fax: (626) 303-7957

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 14G – MT. DOBLE

FIGURE

CIP.14G



SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED PIPELINE

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

CIVILTEC engineering inc.

General Civil, Municipal, Water and Wastewater Engineering, Planning, Construction Management and Surveying

118 West Lime Avenue
Monrovia, Ca. 91016
Phone: (626) 357-0588
Fax: (626) 303-7957

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
2008 WATER MASTER PLAN

PROJECT 15G – MONMOUTH

FIGURE

CIP.15G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

NEW HYDRANT

PROPOSED NEW PIPELINE

BIG BEAR CITY COMMUNITY SERVICES DISTRICT	CIVILTEC engineering inc. General Civil, Municipal, Water and Wastewater Engineering, Planning, Construction Management and Surveying. 118 West Lime Avenue Monrovia, Ca. 91016 Phone: (626) 357-0588 Fax: (626) 303-7957	BIG BEAR CITY COMMUNITY SERVICES DISTRICT 2008 WATER MASTER PLAN	FIGURE
		PROJECT 16G – E LANE	CIP.16G

SCALE: 1"=200'

0 100 200 400 600

SCALE FEET

PROPOSED PIPELINE

BIG BEAR CITY COMMUNITY SERVICES DISTRICT	CIVILTEC engineering inc. — General Civil, Municipal, Water and Wastewater Engineering, Planning, Construction Management and Surveying. 118 West Lime Avenue, Monrovia, Ca. 91016, Phone: (626) 357-0588, Fax: (626) 303-7957	BIG BEAR CITY COMMUNITY SERVICES DISTRICT 2008 WATER MASTER PLAN PROJECT 17G – LAKEWOOD	FIGURE CIP.17G

Appendix J

B.E.S.T. and Aclara Estimates

THE B.E.S.T. METER CO., INC.
2648 E. Workman Avenue
PMB 426
West Covina, CA 91791-1604
626-915-4402
FAX 626-915-0075

REPRESENTATIVE FOR:

MASTER METER

March 17, 2009

Anthony Herda
Staff Engineer
CIVILTEC Engineering, Inc.
118 W. Lime Avenue
Monrovia, CA 91016

REFERENCE: City of Big Bear Fixed Network Estimate

Dear Anthony:

Thank you for giving us this opportunity to work with you on the City of Big Bear Fixed Network Project/Estimate.

Enclosed, please find an estimate by HD Supply Waterworks based on the information you have given us. As indicated, without a propagation study, the number of repeaters may vary.

Also enclosed, please find the following:

1. Literature on Master Meter's Fixed Network System
2. Line card on HD Supply Waterworks/National Service Group
3. Questionnaire for Benefit Analysis
4. Sample Benefit Analysis

We are looking forward to working with you and the City of Big Bear. Please let us know if you have any questions or need a demonstration on our product.

Ed Amelung of Master Meter, is the local Regional Sales Manager for California, Chris Lambros of HD Supply Waterworks is the local distributor for California and I am the local Representative for Master Meter in Southern California. We all work closely together to prove our commitment to excellence in product and service to our customers.

Sincerely,

Nancy Jenkins
President

enclosures

HD Supply Waterworks
Chris Lambros - Product Specialist
3155 N. Indian Avenue
Perris, CA 92562
Phone Number: 909-376-3141
Fax Number: 559-553-8811

Date 3-16-09

CIVILTEC Engineering, Inc.
Anthony Herda - Staff Engineer
118 West Lime Avenue
Monrovia, CA 91016
Phone: 626-357-0588
Fax: 626-303-7957

Subject: 3G Master Meter Fixed Base System & Meters

Quantity	Description		Each		Extended
	FixedLinx - Repeater Options				
5	**1.** New Site - FixedLinx Repeater with up to 100' Tower (Installed) Customer Furnishes Site. The current range for each repeater is estimated 3 square miles. There is a possibililty of fewer Repeaters.	$	42,000.00	$	**210,000.00**
	OR				
5	**2.** Existing Structure - FixedLinx Repeater if structure height meets Master Meter guidelines to have a reading area of 3 square miles.	$	15,000.00	$	**75,000.00**
	OR				
5	**3.** Existing Structure (but needs a 60' Tower) - Installedf FixedLinx Repeater	$	30,000.00	$	**150,000.00**
	NOTE: 3 to 5 Repeaters, 60' to 100' may vary. Propagation Study Required Customer must provide site and power drop.				
	Concentrator				
1	Site FixedLinx Concentrator	$	15,000.00	$	**15,000.00**
	Software Management Systems				
1	MasterLinx Enterprise Management System Data Center Annual Master Meter Hosting the Software (Web-Based)	$	7,500.00		**7,500.00**
	Training				
1	3 Day Onsite Software Training System Configuration Includes all Travel Expenses	$	7,500.00		**$7,500.00**
	Maintenance				
1	Annual Maintenance Fee - Includes up to 5 Repeaters	$	6,000.00		**Annually**
	Optional Drive-By System				
1	Complete Drive-By Laptop System: includes Panasonic Toughbook 51, DMMR Receiver, USB GPS, Antenna, Vehicle Stand, Software, Cable and Charger - Annual Maintenance Fee after 1st year $1,200.	$	25,000.00	$	**25,000.00**
	3G Radio Meters and Installation		**EACH**		**EXTENDED**
	3/4" Size Meters				
5880	5/8x3/4" Master Meter 3G Radio Meter - **Envirobrass**	$	182.00	$	**1,070,160.00**
120	3/4x9" Master Meter 3G Radio Meter - **Envirobrass**	$	215.00	$	**25,800.00**
	Boosters required for Fixed Base				
6000	Master Meter 3G Boosters in Pit Application (Wet Environment)	$	125.00	$	**750,000.00**
	Installation of Meters and Boosters (Budgetary Only)				
6000	Installation - Cutting 1-3/4" Hole for Booster Installation	$	10.00	$	**60,000.00**
6000	3/4" Meter Installation (Meter Nut to Meter Nut Only) Includes GPS, and Paperless Work Order	$	50.00	$	**300,000.00**
			Subtotal	$	**2,205,960.00**

	Total Option Pricing (Does Not Include Tax on all Above Items)		
Option 1:	New Site for Repeaters (Needs 100' Tower) w/ Meters & Installation	$	2,445,960.00
Option 1a:	Includes Option 1 and Drive-By System	$	2,470,960.00
Option 2:	Existing Site for Repeaters (No Tower Needed) w/ Meters & Installation	$	2,310,960.00
Option 2a:	Includes Option 2 and Drive-By System	$	2,335,960.00
Option 3:	Existing Site for Repeaters (Needs 60' Tower) w/ Meters & Installation	$	2,385,960.00
Option 3a:	Includes Option 3 and Drive-By System	$	2,410,960.00

Note: All meters are quoted with Envirobrass Bodies to meet/exceed AB1953 and Prop 65.

Tax Not Included. - Quote Good for 60 Days
Full Freight Allowed for $50,000.00 +

Mountain States Pipe and Supply

Casey Pike
Mountain States Pipe and Supply
8119 Indian Peaks Avenue
Frederick, Colorado 80516
(303) 678-1300

Civil Tech Estimate

Aclara Star Fixed Network

Aclara Pricing				BUDGET PRICING	
Item Number	**Part Number**	**Description**	**Qty**	**Price Each**	
Fixed Network Star Software					
1	Network Control Computer and NCC Software	**STAR Network Control Server** supplied with all licenses and **Aclara STAR Software** Software Maintenance Fee: $2200 per Year	1	$20,500.00	$20,500.00
Data Collection and Installation Hardware and Software					
2	DCU - Aclara	Cell Control Unit (CCU) Base Unit, 120V (GSM/GPRS) Note: The DCU Unit is available as a self contained unit with Cell Phone connectivity or for this application, AC Power and Phone Line. Phone line is only used in middle of night as a dial out connection. Power, Phone and Phone Service to be supplied by utility	1	$5,000.00	$5,000.00
Aclara MTU Interface Units					
3	MTU	Aclara Single-Port MTU Unit	7000	$118.00	$826,000.00
Severn Trent Water Meters					
4	Severn Trent 20 gpm	Severn Trent solid state water meter with Encoded Register	7000	$112.00	$784,000.00
MTU Programmer					
5	PSION	Psion Programmer Used to program MTU units	1	$2,233.00	$2,233.00
Implementation					
6	Implementation	NCC Installation and Training	2	$2,800.00	$5,600.00
7	Implementation	DCU Installation *Aclara Installation of DCU Units on Towers*	2	$1,500.00	$3,000.00
8	Implementation	STAR System Operaotrs MTU / DCU Training	1	$2,800.00	$2,800.00
FCC Licensing Fee					
9	FCC Application Fee		1	$300.00	$300.00
Union Labor for Installation of Meter and AMR					
10	Labor		7000	$90.00	$630,000.00
Total Cost....					**$2,279,433.00**

Appendix K

BBCCSD Water Services Operating Budget

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

WATER SERVICES

OPERATING BUDGET

SUMMARY: The Water Department services 5,990 connections, 24-hours per day, 365 days per year. The Department provides potable drinking water within an 8 square mile service area. This service area excludes a portion of Whispering Forest, Erwin Lake, and all of Sugarloaf. Department operations are supported by four reservoirs with a total capacity of 6.25 million gallons, 80 miles of main lines, 418 fire hydrants and 1,840 gate valves. The water system also supports fire suppression activities with water flows that range from 500 to in excess of 1,500 gallons per minute.

The minimum base rate for residential water service, irrespective of meter size or consumption, is a bimonthly fee of $40.04. Owners of improved parcels receiving water service pay an additional annual fee of $30 that is included on their property tax bill. Owners of unimproved parcels where water service is available pay an annual fee of $40 that is included on their property tax bill.

Cost comparisons for single-family residences show that District charges are lower on average than those levied by the Crestline Village Water District, City of Big Bear Lake, Running Springs, and the Lake Arrowhead Community Services District.

RESOURCES:	FY 08-09	FY 07-08
PERSONNEL	*9.0*	*9.0*
TOTAL REVENUES/ RESERVE TRANSFERS	*$3,769,782*	*$3,740,500*
TOTAL APPROPRIATIONS	*3,769,782*	*3,740,500*
CASH TO RESERVES	*0*	*0*

GOAL & OBJECTIVES: The goal of the Water Department is to provide the most cost effective water service in the area to our customers. This goal will be accomplished through work plans that focus on installation, maintenance, repair activities and future improvements to the potable water system to enhance the level of service provided.

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
WATER SERVICES

	ADOPTED BUDGET 7/01/07-6/30/2008	ADOPTED BUDGET 7/01/08-6/30/2009	*FORECASTED BUDGET 7/01/09-6/30/2010*
Revenues and reserve transfers:			
Charges for services	$ 2,100,000	$ 2,150,000	$ 2,200,000
Turn on/off charges	26,000	28,000	28,000
Standby charges	220,000	233,000	230,000
Connection fees	38,500	20,000	27,000
Delinquent assessments	30,000	31,000	31,000
Other income	-	-	-
Total revenues	2,414,500	2,462,000	2,516,000
Reserve transfers	1,326,000	1,307,782	-
Total revenues/reserve transfers	***3,740,500***	***3,769,782***	***2,516,000***
Operations and maintenance (O&M):			
Salaries and benefits	796,003	856,439	920,843
Source of supply	110,000	115,000	115,000
Transmission and distribution	102,000	100,000	100,000
Utilities	18,000	19,000	19,000
Fiscal agent fees	1,000	1,000	1,000
Insurance	20,000	22,000	22,000
Engineering	10,000	15,000	15,000
Vehicle expense	50,000	50,000	50,000
Education and training	7,000	7,000	7,000
Memberships, subscriptions and dues	13,000	13,000	13,000
Permits and fees	12,500	18,500	18,500
SCADA system maintenance	10,000	10,000	10,000
Stickleback maintenance	1,000	5,000	5,000
General services	519,035	481,274	504,640
Safety compliance	-	28,586	30,278
General fund reserves	50,000	50,000	-
Other operating expenses	58,916	57,000	57,000
Total O&M appropriations	1,778,454	1,848,799	1,888,261
Debt service:			
Citizens Business Bank loan	118,441	118,369	118,294
Fluoride blending project COP's	222,585	216,925	225,000
Retrofit loan	38,020	38,020	38,020
Total debt service	379,046	373,314	381,314
Capital appropriations-CIP	257,000	239,887	246,425
Capital appropriations-Reserves	1,326,000	1,307,782	-
Total appropriations	***3,740,500***	***3,769,782***	***2,516,000***
Cash to reserves	***$ -***	***$ -***	***$ -***
Rate increase	0.0%	0.0%	0.0%

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

WATER SERVICES - PHASE 1

DEBT SERVICE SCHEDULE

In May 2003, the District refinanced Water Revenue Bonds originally issued through the California Statewide Communities Development Authority. The original loan was for the construction of a project which enabled the District to meet safe drinking water standards. Interest and principal payments are due as follows:

PAYMENT DATE	PRINCIPAL AMOUNT	INTEREST RATE	INTEREST AMOUNT	PERIOD TOTAL	FISCAL YEAR TOTAL
7/15/2008	$ 46,562	3.97%	$ 13,556	$ 60,118	$ 118,369
1/15/2009	$ 47,486	3.97%	$ 12,632	$ 60,118	$ -
7/15/2009	$ 48,429	3.97%	$ 11,689	$ 60,118	$ 118,294
1/15/2010	$ 49,390	3.97%	$ 10,728	$ 60,118	$ -
7/15/2010	$ 50,371	3.97%	$ 9,747	$ 60,118	$ 118,217
1/15/2011	$ 51,371	3.97%	$ 8,747	$ 60,118	$ -
7/15/2011	$ 52,390	3.97%	$ 7,728	$ 60,118	$ 118,135
1/15/2012	$ 53,430	3.97%	$ 6,688	$ 60,118	$ -
7/15/2012	$ 54,491	3.97%	$ 5,627	$ 60,118	$ 118,051
1/15/2013	$ 55,572	3.97%	$ 4,546	$ 60,118	$ -
7/15/2013	$ 56,676	3.97%	$ 3,442	$ 60,118	$ 117,964
1/15/2014	$ 57,801	3.97%	$ 2,317	$ 60,118	$ -
7/15/2014	$ 58,948	3.97%	$ 1,170	$ 60,118	$ 58,948
TOTAL	$ 682,917				$ 767,978

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

WATER SERVICES - FLUORIDE BLENDING PROJECT

DEBT SERVICE SCHEDULE

In November 1996, the District issued Refunding Certificates of Participation, 1996 Series Z to refinance the construction costs of a fluoride blending facility. Interest and principal payments are due as follows:

PAYMENT DATE	PRINCIPAL AMOUNT	INTEREST RATE	INTEREST AMOUNT	PERIOD TOTAL	FISCAL YEAR TOTAL
8/1/2008	$ 205,000	5.20%	$ 11,292	$ 216,292	$ 216,925
2/1/2009	$ -		$ 5,963	$ 5,963	
8/1/2009	$ 225,000	5.30%	$ 5,962	$ 230,962	$ 225,000
TOTAL	$ 430,000				$ 441,925

$177,266 Certificate of Participation requirement

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

WATER SERVICES - RETROFIT LOAN

DEBT SERVICE SCHEDULE

In 1989, the Department of Water Resources approved a water conservation construction loan to the District. The proceeds of the loan were used for the installation of ultra-low flush toilets, low flow showerheads, and water meters. Interest and principal payments are due as follows:

PAYMENT DATE	PRINCIPAL AMOUNT	INTEREST RATE	INTEREST AMOUNT	PERIOD TOTAL	FISCAL YEAR TOTAL
10/1/2008	$ 16,593	3.44%	$ 2,424	$ 19,017	$ 38,020
4/1/2009	$ 16,887	3.44%	$ 2,130	$ 19,017	$ -
10/1/2009	$ 17,164	3.44%	$ 1,853	$ 19,017	$ 38,020
4/1/2010	$ 17,468	3.44%	$ 1,549	$ 19,017	$ -
10/1/2010	$ 17,761	3.44%	$ 1,257	$ 19,017	$ 38,020
4/1/2011	$ 18,072	3.44%	$ 945	$ 19,017	$ -
10/1/2011	$ 18,378	3.44%	$ 639	$ 19,017	$ 38,111
4/1/2012	$ 18,695	3.44%	$ 322	$ 19,017	$ -
TOTAL	$ 141,019				$ 152,171

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
5 YEAR CAPITAL IMPROVEMENT PROGRAM
FISCAL YEAR ENDING 6/30/09
WATER DEPARTMENT

Projects	2008/09	2009/10	2010/11	2011/12	2012/13	Totals
Mainline Improvements	$ -	$ 201,425	$ 220,000	$ 220,000	$ 220,000	$ 861,425
SCADA Maint./Upgrade & Training	10,000	10,000	10,000	10,000	10,000	50,000
Well Pump Replacements	10,000	10,000	10,000	10,000	10,000	50,000
Purchase future Well Site	76,987					76,987
AQMD Generator Test & Retrofit	40,000					40,000
Fence Van Dusen Property	8,000					8,000
Portable Pump	40,000					40,000
Replace 4 Well Site Heaters	5,000					5,000
Warning Light Kits for Vehicles	4,500					4,500
Laptop Computer	3,000					3,000
Jackhammers	2,900					2,900
Bobcat purchase (50%)	30,000					30,000
Steel Plates (50%)	2,500					2,500
Well 3B Video & Pump	7,000					7,000
Paving of Well Sites		25,000				25,000
Greenspot Main				60,000		60,000
Totals	$ 239,887	$ 246,425	$ 240,000	$ 300,000	$ 240,000	$1,266,312

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

PROJECTED WATER FUND RESERVES

	VEHICLE REPLACEMENT	FACILITIES AND EQUIPMENT	DEBT SERVICE RESERVE (Note 1)	EMERGENCY OPERATING & CONTINGENCY	TOTAL RESERVES
BEGINNING BALANCE	$ 521,000	$ 3,051,734	$ 177,266	$ 250,000	$ 4,000,000
ADDITIONS:					
Interest Income	-	101,000	-	-	101,000
Cash from Operations	-	-	-	-	-
EXPENDITURES:					
Replace Unit # 123	(32,000)	-	-	-	(32,000)
Paradise Yard Paving	-	(36,782)	-	-	(36,782)
Well # 8 - Flouride Blend Line *(carryover)*	-	(655,000)	-	-	(655,000)
Barranca/Country Club Mainline Improve. Programs *(carryover)*	-	(364,000)	-	-	(364,000)
Water Master Plan Update *(carryover)*	-	(220,000)	-	-	(220,000)
6/30/09 BALANCE	$ 489,000	$ 1,876,952	$ 177,266	$ 250,000	$ 2,793,218

Note 1:Certificates of Participation requirement. Amount is not available to the District until 8/1/09.

BIG BEAR CITY COMMUNITY SERVICES DISTRICT
VEHICLE REPLACEMENT
FISCAL YEAR ENDING 6/30/09
WATER SERVICES

Vehicle/Equipment	Replacement Cost *	Useful Life	Fiscal Year Scheduled
Unit #123 - 3/4 Ton Pickup 1998	$ 32,000	10 Years	2008/09
Unit #117 - 3/4 Ton Pickup 2001	32,000	10 Years	2011/12
Unit #101 - Chevy Trailblazer 2002	30,000	10 Years	2012/13
Unit #116 - 2 Ton Utility 2000	75,000	15 Years	2015/16
Unit #126 - 3/4 Ton Utility 2006	35,000	10 Years	2016/17
Unit #127 - 3/4 Ton Utility 2006	35,000	10 Years	"
Unit #128 - 3/4 Ton Utility 2006	35,000	10 Years	"
Unit #162 - Power-Vac 2002	45,000	15 Years	2017/18
Unit #124 - 3/4 Ton Pickup 2007	40,000	10 Years	"
Unit #125 - CAT Backhoe 2004	100,000	15 Years	2019/20
Unit #902 - F-600 Dump Truck 2006 (A)	30,000	25 Years	2031/32
Totals	**$ 489,000**		

* Prices are based on current replacement costs.
(A) Replacement cost represents 50% of total required (shared with Sewer Dept.)

WATER SERVICES

Fiscal Year: 2008-09

PROJECT OBJECTIVES

1. PURCHASE FUTURE WELL SITE

Project Manager: Tim Moran, Water Superintendent.

Project Team: Michael Mayer, and Jerry Griffith.

Scope of Work: Negotiate price, and purchase approximately ½ acre of property east of wells 1& 1B. There currently is not enough room on District Property to drill Replacement wells to replace the oldest wells in the District.

Anticipated Result: Create a buffer zone as property is currently used to graze animals. Provide adequate space to drill replacement wells in the future.

Budget: $76,987

Start Date: July 2008

Target Completion Date: June 2009

Current Status: Discussion with current owners.

2. AQMD GENERATOR EMISSIONS TESTING & RETROFIT

Project Manager: Jerry Griffith, Water Foreman.

Project Team: Rachael Jahn, and Bryan Cavanagh

Scope of Work: Test 2 Water Department standby generators for emissions. Retrofit exhausts systems if tests indicate excessive emissions.

Anticipated Result: Reduce emissions and comply with AQMD rules.

Budget: $40,000

Start Date: July 2008

Target Completion Date: December 2008

Current Status: Requesting quotes from qualified testing companies.

3. FENCE VAN DUSEN PROPERTY

Project Manager: Jerry Griffith, Water Department Foreman.

Project Team: Ralph Musella.

Scope of Work: Install approximately 760 feet of fencing on District Property at the intersection of Van Dusen Canyon Road and Hale Drive.

Anticipated Result: Reduce liability by denying access to motorized vehicles on District Property

Budget: $8,000

Start Date: July 2008

Target Completion Date: December 2008

Current Status: Received 1 quote from LEOCO Fence Company.

Appendix L

Water Rate Review
(Bartle Wells, 1995)

BIG BEAR CITY COMMUNITY SERVICES DISTRICT

WATER RATE REVIEW

Board of Directors
Robert Buhrle, *President*
Bob Colven, *Director*
Stew Hanson, *Director*
Dr. Len Holguin, *Director*

Water Rate Review Committee
Bob Colven, *BBCCSD Director*
Dr. Len Holguin, *BBCCSD Director*
Robert Poe, *Member, Citizens Advisory Committee*
Ralph Olsen, *Member, Citizens Advisory Committee*
Phil Schuyler, *Whispering Pines Estates*

District Staff
Ray Griest, *General Manager*
Gary Keller, *Water Superintendent*
Mike Mayer, *Finance Officer*

April 1995

BARTLE WELLS ASSOCIATES
Independent Public Finance Advisors
1636 Bush Street
San Francisco CA 94109

Tel. 415/775-3113

Bartle Wells Associates
1636 Bush Street
San Francisco 94109
415/775-3113 FAX 415/775-4123

Charter member
National Association of
Independent Public Finance Advisors

April 4, 1995

Big Bear City Community Services District
PO Box 558
Big Bear City CA 92314

Attn: Gary Keller
Water Superintendent

Re: Water Rate Review

We are pleased to present our Water Rate Review. The report presents our findings and recommendations as developed over the previous three months. Much of our work and deliberations were presented publicly at three public meetings of the Water Rate Review Committee and at a meeting of the District's Board of Directors.

The patience, goodwill, and constructive criticism of the Water Rate Review Committee is greatly appreciated. We are especially indebted to District Water Superintendent Gary Keller for his good advice and timely efforts on our behalf. We also want to thank General Manager Ray Griest and Finance Officer Mike Mayer for their assistance during our study.

We remain available to assist the District in implementing our recommendations.

Very truly yours,

BARTLE WELLS ASSOCIATES

Thomas E. Gaffney, CIPFA
President

Douglas R. Dove, CIPFA
Project Consultant

TEG/DRD:mt
Enc.

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FINDINGS AND RECOMMENDATIONS

- District revenues are adequate to fund operating expenses and ongoing capital expenses.
- Capital expenses should be funded at $300,000 per year based on the District's five-year capital plan.
- Overall, the District's current rate structure is equitable, encourages conservation, and is well-suited for a mountain resort community.
- No changes to the volume rates are recommended.
- Adjustment of the bimonthly meter charge is recommended, to more equitably allocate fixed costs.
- Bimonthly meter charges should be proportional to water use based on meter size. This reflects the customer's potential demand on the system.
- The standard meter charge should remain at $32.50 for residential customers.
- Residential customers are generally served by 5/8" or 3/4" meters with a flow rating of 25 gallons per minute. The flow ratings are based on AWWA standards.
- Charges for larger meters should be proportional to the flow rating for larger meters.
- Standby charges on undeveloped parcels should be increased by $10, to $40 per parcel per year, to fully reflect the benefits of water availability.
- Currently all parcels (developed and undeveloped) are charged a standby charge of $30 per year per parcel. The increase is projected to generate $20,000 per year in additional fixed revenues.
- To maintain equity, tier breakpoints for larger meters should be increased in proportion to the meter charge—e.g., if a 1" meter is charged two times the residential meter charge, then the corresponding tier breakpoints should be doubled.
- All District water customers should be charged on the same basis.
- The mobile home park is a 6" meter customer and should be charged a proportional meter charge and water rates.
- Customers with oversized meters would be reviewed on a case-by-case basis to equate meter size with actual water use. Any adjustments of meter charges made during the initial phase-in to maintain equity should be based on factual information available to the District.

INTRODUCTION

Big Bear City Community Services District has retained Bartle Wells Associates to prepare a water rate study and develop rate alternatives to meet the following District goals:

- Preserve equity among customer classes
- Provide sufficient revenues to fund ongoing operational as well as capital expenses
- Promote water conservation
- Provide incentives for full-time water use rather than peak season use

The District appointed a water rate study committee to review our progress and to meet periodically to review and discuss water rate alternatives. The committee consists of two Board members, two members of the citizens advisory committee, and a representative from the Whispering Pines Mobile Home Park, the District's largest water customer.

The water rate review committee met monthly on three separate occasions during the project development period. Each of these meetings was a duly noticed public hearing, open to the public, the press, any interested local citizens, and District water customers. In addition, the District wrote letters to high-consumption water users informing them of the potential impacts on water rates as a result of the study's recommendations. Much information and many tables were presented at the public water rate review committee meetings. Not all of the material discussed at the meetings is contained in this report. The material discussed in the meetings which is not contained in this report is a part of the District's record and is available for further public inspection and review in the offices of the Water Department.

This report presents the results of our findings and recommendations. The recommendations have already been presented to the water rate review committee and are subject to further deliberations at regular meetings of the District's Board of Directors. This report and our findings and recommendations will be further reviewed by the District as needed.

DISTRICT WATER UTILITY

Big Bear City Community Services District is located in the San Bernardino Mountains, about 50 miles east of the City of San Bernardino. Elevation within the District ranges from 6,700 to 7,100 feet. The District serves a population of about 10,500 and provides water, sewer, fire protection, and refuse collection services.

The District serves about 5,275 water connections, of which about 5,100 are active. The water service area covers about 8 square miles. Major facilities include 73 miles of pipelines ranging from 1½ to 20 inches in diameter, 10 vertical wells, 2 slant wells, 4 tank reservoirs which store a total of 6.25 million gallons of water, and 6 water pressure booster systems. Table 1 shows the current and projected water requirements. This information was developed in the District's *Water Master Plan,* dated June 1991 and prepared by So & Associates Engineers Inc. The District's capital plans include provision for supplying the required water through the planning period.

Table 2 shows a summary of District water customers by meter size. All water customers are served metered water. For various reasons, some meters are larger than a customer may actually require. For example, certain residential customers were connected to the system through 2" meters, while the normal

TABLE 1 ■ **Big Bear City Community Services District Water Rate Review Water Requirements**

	1990	1994	1996	2002
Connections	5,250	5,375	5,825	6,400
Average daily demand (mgd)	1.05	1.02	1.17	1.28
Annual production (acre-feet)	1,100	1,068	1,225	1,340

TABLE 2 ■ **Big Bear City Community Services District Water Rate Review Summary of Customers by Meter Size**

Meter Size	Customers	Percent of Total
5/8"	5,096	96.6%
3/4"	67	1.3
1"	64	1.2
1-1/2"	22	0.4
2"	20	0.4
3"	2	0.0
4"	3	0.1
6"	1	0.0
Totals	5,275	100.0%

size meter for such a customer is a 5/8" or 3/4" meter. Certain other water customers may also be assigned a larger meter than may be required by their water consumption. Because the District levies a uniform monthly service charge to all water customers except the mobile home park, this doesn't currently affect a customer's water bill. The table includes about 175 inactive meters. Inactive meters are not subject to bimonthly water bills, but must apply to the District and pay a reconnection fee before water service is restored.

Small meters (5/8" and 3/4") constitute the bulk of District water customers (97.9 percent). These meters serve residential and small commercial users. As may be expected, Table 3 shows that the larger meters use proportionately more water than the smaller meters. The largest water user, the mobile home park, is served through a 6" meter and consumes about 5 percent of total water sold within the District. The five 3" or 4" meters together also consume about 5 percent of total water sold.

Table 4 shows actual water consumption for seven of the largest meter customers in the District. This information demonstrates the variations in seasonal water use of several water users. Of the group, the 4" high school meter registers bimonthly water use ranging from a low of 137 hundred cubic feet (ccf) to a high of over 5,600 ccf during the summer peak-use period. The District must construct facilities to serve the high summertime peak use, but during the winter only receives payment for between 140 to 300 ccf's of water sales. Note that the 2" high school meter uses only a nominal amount of water. This account is assigned too large a meter for water actually used.

Table 5 summarizes water use over the six bimonthly billing periods. Water use peaks during summer months when outdoor water use is greatest. Winter use is slightly higher than spring use because of the area's winter sports activities. Summer use averages about twice the winter average use. The District's peak use determines the sizing and cost of capital facilities.

TABLE 3 ■ **Big Bear City Community Services District Water Rate Review Metered Water Sales, 1994**

Meter Size	Water Use (acre-feet)	Percent of Total
5/8"	754.3	81.2%
3/4"	17.9	1.9
1"	22.1	2.4
1-1/2"	28.5	3.1
2"	13.0	1.4
3"	3.5	0.4
4"	42.9	4.6
6"	46.5	5.0
Totals	928.7	100.0%

TABLE 4 ■ Big Bear City Community Services District Water Rate Review
Water Consumption Patterns - Large Meters (ccf)

Billing Period	Acct 15-0110 High School 4" Mtr	Accounts 15-0090 / 15-0100 High School 2" Mtr	Accounts 15-0090 / 15-0100 High School 3" Mtr	Accounts 003-5030 / 005-7251 Athletic Facility 4" Mtr	Accounts 003-5030 / 005-7251 Athletic Facility 2" Mtr	Acct 10-1935 Public District 3" Mtr	Acct 11-5315 Park 4" Mtr
1994 Nov-Dec	305	10	10	0	0	149	59
Sep-Oct	2,031	20	460	114	110	199	566
Jul-Aug	5,379	0	1,160	798	770	239	957
May-Jun	4,920	10	160	898	1,320	280	1,341
Mar-Apr	760	10	20	196	350	205	21
Jan-Feb	315	10	0	35	0	267	0
Total	13,710	60	1,810	3,041	2,550	1,339	2,944
1993 Nov-Dec	205	10	20	23	0	214	31
Sep-Oct	3,345	20	800	126	40	262	389
Jul-Aug	5,609	0	1,200	979	1,010	237	967
May-Jun	4,413	10	670	851	730	188	663
Mar-Apr	1,585	10	10	519	660	90	52
Jan-Feb	137	10	0	29	0	298	6
Total	15,294	60	2,700	2,547	2,440	1,289	2,108

TABLE 5 ■ Big Bear City Community Services District Water Rate Review
Summary of 1994 Water Use by Season

Billing Period	Bimonthly Water Use (acre-feet)
January/February	113.5
March/April	101.3
May/June	218.0
July/August	230.4
September/October	154.2
November/December	111.3
Total	928.7
Bimonthly average	154.8
Summer average (May – August)	224.2
Winter average (November – February)	112.4

Current water rates are shown in Table 6. All water customers except the mobile home park pay a bimonthly fixed charge of $32.50 regardless of the amount of water used or the meter size. Water customers also pay a water quantity rate based on the metered amount of water used. Quantity rates vary depending on the amount of water used within each of three tiers. For the first 24 ccf of bimonthly water consumption the rate is $1.20 per ccf; for the next 16 ccf (up to 40 ccf total) the rate is $1.50 per ccf; all water use above 40 ccf is billed at $1.80 per ccf.

For high-use water customers, much water use is billed at the highest rate, the third tier. This form of quantity rate structure is known as an inclining block rate. It is widely used by public water agencies throughout California, because it economically encourages water conservation. The more water a customer uses, the higher (within three tiers) the water rate.

To equate mobile home users to other residential users, the District assigned a bimonthly fixed charge to each mobile home space within the mobile home park. To account for less than average use, a factor of 85 percent was applied to the fixed charge resulting in a bimonthly charge of $27.62 per occupied mobile home space. The mobile home park provides 248 spaces so the bimonthly charge to the mobile home park is about $6,850. The quantity rate tiers were adjusted to reflect the amount of water in each tier for the adjusted number of residences served.

Table 7 shows a summary of the District's standby charge which is charged to every real property parcel. This charge is authorized by California Government Code §61765.9. The code allows the District to collect up to $40 per parcel or per acre of land. Revenues from the charge over the first $10 must be used for capital improvements or to fund debt service. Annual revenues from the charge total about $219,000 while the District's current annual debt service totals about $423,000. Therefore the charge meets the legal use test.

TABLE 6 ■ **Big Bear City Community Services District Water Rate Review**
Current Water Rate Structure (Effective July 1, 1993)

Residential/Commercial	
Bimonthly fixed charge	$32.50
Bimonthly quantity rates (per ccf):	
1-24 ccf	1.20
25-40 ccf	1.50
Over 40 ccf	1.80
Mobile Home Parks	
Bimonthly fixed charge per occupied space	27.62
Bimonthly quantity rates:	
Same as residential except tier boundaries multiplied by number of spaces	
Late Fee	10%
Facilities Charge	
One-time buy-in charge (per connection)	$2,606

TABLE 7 ■ **Big Bear City Community Services District Water Rate Review Summary of Standby Charges**

Current Charge: $30/year per parcel; charge is collected with property taxes.

Projected 1994/95 Revenues: $219,000

Number of Parcels: About 7,300

Parcels Assessed: All improved and unimproved parcels within District's water service area.

Maximum Charge Permitted: Under Government Code §61765.9, District can levy standby charges of up to $40 per acre or parcel per year.

Other Limitations: Proceeds of charges over $10 per acre or parcel must be used for water system improvements (capital).

Usually, standby charges are levied more heavily on undeveloped properties than on developed property. Undeveloped properties receive a benefit from the availability of water facilities for which they make no contribution unless a standby charge is collected. Developed properties pay for capital costs in their bimonthly water bills. The District should establish some differential in the standby charge between developed and undeveloped properties.

A tool for determining the revenues generated from quantity rates is shown in Table 8. The table summarizes the 1994 water consumption for the entire District. About 87 percent of the bimonthly water bills included water sales only within the first tier. In fact, about 17 percent of the water bills registered no measured water sales during a two month billing period. Such a situation is typical for a mountain resort community. Many of the water connections are second homes and not permanent residences. Table 8 also shows that about 8 percent of the bills use water in the second tier and about 5 percent of the bills reach the third tier.

Mountain resort communities present special considerations when developing water rates. Some of the considerations that apply in the District's case include the following:

- Relatively small community
- Many part-time residents
- Significant percentage of nonresident homeowners
- Fixed costs substantially higher than variable costs
- Seasonal usage peaks (winter and summer)
- Relatively low water use per customer
- Primarily residential development
- Limited water supplies

Some of these factors influence water rates. Because many bills include little water consumption, the District must rely on fixed charges to recover a reasonable share of revenues from part-time users. Seasonal peaks and low water use per customer also favor more reliance on fixed charges to develop stable and reliable revenues.

TABLE 8. BIG BEAR CITY COMMUNITY SERVICES DISTRICT
SUMMARY OF 1994 WATER CONSUMPTION
TOTAL DISTRICT

BARTLE WELLS ASSOCIATES
bcon.wk1 3/08/95

BIMONTHLY CONSUMPTION BLOCK (CCF)	TOTAL BILLS IN BLOCK	CUMULATIVE TOTAL BILLS	USAGE IN BLOCK (CCF)	PERCENT OF BILLS AT OR BELOW BLOCK		AVG USE IN TIER (CCF)
0	5,307	5,307	0	17.32%		
3	7,649	12,956	22,947	42.28%		
5	2,495	15,451	12,475	50.42%		
8	2,572	18,023	20,576	58.82%		
10	1,486	19,509	14,860	63.67%		
12	1,456	20,965	17,472	68.42%		
15	1,955	22,920	29,325	74.80%		
18	1,562	24,482	28,116	79.89%	TIER 1	7.06
19	475	24,957	9,025	81.44%		
20	423	25,380	8,460	82.82%		
21	396	25,776	8,316	84.12%		
23	648	26,424	14,904	86.23%		
25	544	26,968	13,600	88.01%		
27	455	27,423	12,285	89.49%		
30	615	28,038	18,450	91.50%		
35	673	28,711	23,555	93.70%	TIER 2	31.45
40	472	29,183	18,880	95.24%		
45	344	29,527	15,480	96.36%		
50	261	29,788	13,050	97.21%		
55	165	29,953	9,075	97.75%		
60	132	30,085	7,920	98.18%		
65	98	30,183	6,370	98.50%		
70	77	30,260	5,390	98.75%		
80	123	30,383	9,840	99.15%		
90	79	30,462	7,110	99.41%		
100	45	30,507	4,500	99.56%		
120	53	30,560	6,360	99.73%		
150	35	30,595	5,250	99.84%		
200	24	30,619	4,800	99.92%		
250	6	30,625	1,500	99.94%	TIER 3	89.48
500	6	30,631	3,000	99.96%		
1000	3	30,634	3,000	99.97%		
2000	4	30,638	8,000	99.98%		
3000	2	30,640	6,000	99.99%		
4000	1	30,641	4,000	99.99%		
5000	2	30,643	10,000	100.00%		
TOTALS	30,643		403,891			

DISTRICT FINANCES

Revenues and Expenses

Table 9 summarizes water system revenues and expenses from 1992/93 through the 1995/96 draft budget. Budgeted revenues for 1995/96 total $1,922,000. The District's largest source of revenues is the bimonthly base rate which generates about $1 million annually. Metered water charges generated $508,000 in 1993/94, but are conservatively estimated at $495,000 in 1995/96. Revenues from the standby charge are budgeted at $239,000 for 1995/96, reflecting a $20,000 rate increase.

Operating expenses for 1995/96 are budgeted at $1,622,000, including debt service of $421,000. Capital expenses are budgeted at $181,000 and an operating surplus of $119,000 is projected.

Five-Year Capital Budget

Table 10 summarizes the District's draft five-year capital plan for the period 1995/96 through 1999/00. The five-year capital budget totals $1,574,875. Water main upgrade and replacement accounts for $982,500, or about 62 percent of the total. Wells are the next-largest item, at $327,500, or about 21 percent of the budget. The budget indicates that the District's annual capital funding requirement is about $300,000.

TABLE 9 ■ **Big Bear City Community Services District Water Rate Review Summary of Revenues and Expenses**

	1992/93	1993/94	Budget 1994/95	Draft Budget 1995/96
Revenues				
Base rate		$ 995,000	$ 995,000	$1,015,000
Metered water	$1,523,000	508,000	472,000	495,000
User fees MHP		63,000	63,000	40,000
Standby charges	218,000	218,000	219,000	239,000
Interest	86,000	79,000	100,000	66,000
Other	1,000	43,000	38,000	28,000
Facilities charge	27,000	32,000	39,000	39,000
Total revenues	1,855,000	1,895,000	1,926,000	1,922,000
Expenses*				
Source of supply	72,000			
Transmission & distribution	188,000	1,265,000	1,212,000	1,201,000
General	580,000			
Administrative	285,000			
Debt service	356,000	288,000	423,000	421,000
Total expenses	1,481,000	1,553,000	1,635,000	1,622,000
Available for reserves/capital	374,000	324,000	291,000	300,000
Capital Expenses				
Total capital expenses			251,000	181,000
Annual reserves			$ 40,000	$ 119,000

*Not including depreciation.

TABLE 10 ■ Big Bear City Community Services District Water Rate Review
Five-Year Capital Plan Draft

Water Projects	1995/96	1996/97	1997/98	1998/99	1999/00	Totals
Mainline replacement	$162,500	$100,000	$100,000	$100,000	$100,000	$ 562,500
New services	13,875	13,875	13,875	13,875	13,875	69,375
Wells Nos. 1 and 1B emergency power	2,500	--	--	--	--	2,500
Replace linoleum, refuse building*	1,250	--	--	--	--	1,250
Utility trailer*	1,250	--	--	--	--	1,250
Water Department office	--	83,000	--	--	--	83,000
Equipment storage yard	--	35,000	--	--	--	35,000
Greenspot transmission main	--	75,000	--	--	--	75,000
Van Dusen main	--	50,000	50,000	50,000	50,000	200,000
Well No. 3	--	--	175,000	--	--	175,000
Well No. 1B	--	--	--	150,000	--	150,000
Division main	--	--	--	--	220,000	220,000
Totals	$181,375	$356,875	$338,875	$313,875	$383,875	$1,574,875

*Water Department's share of costs.

Revenue Requirement From Rates

Table 11 summarizes the water system's revenue requirement for 1995/96. The total revenue requirement for 1995/96 is $1,922,000. The revenue requirement includes operation and maintenance costs, debt service costs, and an allowance of $300,000 for ongoing capital costs. Costs related to the assessment district are funded separately and not included in the revenue requirement.

The total revenue requirement from Water charges (bimonthly meter charges, volume charges, standby charges) is $1,789,000. Revenues from facilities charges, interest on reserve funds, and meter payback and new service hookups are estimated at $133,000.

TABLE 11 ■ **Big Bear City Community Services District Water Rate Review Revenue Requirement, 1995/96 Budget**

Uses of Funds	
Operation & maintenance	$1,201,000
Debt service	421,000
Capital replacements	300,000
Total uses	1,922,000
Source of Funds	
Facilities charge	39,000
Interest	66,000
New services and turn-on/turn-off	28,000
Subtotal	133,000
Required from rates	1,789,000
Budget 1995/96 (proposed)	
Service charges	1,023,000
Metered water	527,000
Standby charges	239,000
Revenue from rates	$1,789,000

Reserve Fund Balances

The District maintains separate reserves within the Water Department for vehicle replacement, system improvements, and facilities charges. The estimated fund balances as of July 1, 1995 are as follows:

Vehicle replacement fund	$ 134,000
System improvements	576,000
Facilities charges	590,000
Total	$1,300,000

The $590,000 facilities charge reserve is limited in use to capital improvements and system expansion, and cannot be used for operation, maintenance, or replacement purposes. Other reserves total about $710,000, or about 43 percent of the District's 1994/95 operating budget.

Fixed vs. Variable Expenses

Table 12 summarizes the District's fixed and variable O&M expenses based on the 1994/95 budget. Like most water agencies, the District's fixed costs far exceed the variable costs. The District's fixed costs are estimated at 96 percent of operating costs.

TABLE 12 ■ Big Bear City Community Services District Water Rate Review Fixed and Variable O&M Expenses

	Budget 1994/95		Percent of Total O&M Expenses	
			Fixed	Variable
Wages & salaries	$ 325,000	- fixed	19.9%	-- %
Payroll overhead	124,000	- fixed	7.6	--
Insurance:				
Workers' compensation	23,000	- fixed	1.4	--
General	36,000	- fixed	2.2	--
Laboratory tests	16,000	- fixed	1.0	--
Medical fees	1,000	- fixed	0.1	--
Maintenance and repair:				
Source of supply	24,000	- fixed	1.5	--
Lines	66,000	- fixed	4.0	--
General	18,000	- fixed	1.1	--
Supplies:				
Operating	13,000	- fixed/variable	0.4	0.4
Operating inventory	33,000	- fixed	2.0	--
Water conservation	7,000	- var increase	--	0.4
Gas & oil	11,000	- fixed	0.7	--
Utilities	53,000	- variable	--	3.2
Professional fees	64,000	- fixed	3.9	--
Mechanic contribution	27,000	- fixed	1.7	--
Safety contribution	14,000	- fixed	0.9	--
Rents & leases	1,000	- fixed	0.1	--
Memberships & education	9,000	- fixed	0.5	--
Miscellaneous	1,000	- fixed	0.1	--
Administrative contribution	346,000	- fixed	21.2	--
Debt service	423,000	- fixed	25.0	--
Total	$1,635,000	- fixed	96.0%	4.0%

Fixed vs. Variable Charges

Table 13 summarizes the District's revenues from fixed and variable charges based on the 1994/95 budget. As previously mentioned, water charges are the District's primary source of revenues. Fixed charges provide 65.3 percent of the District's revenues. Fixed charges provide a stable, dependable revenue source regardless of usage levels or seasonal factors. Variable water charges provide 25.6 percent of the District's revenues. Variable charges increase or decrease with water usage and provide an economic incentive to conserve. Other revenues including interest earnings provide the remaining 9.1 percent of revenues.

TABLE 13 ■ **Big Bear City Community Services District Water Rate Review Fixed and Variable Charges, 1994/95 Budget**

			Percent of Total Revenues		
			Fixed Charges	Variable Charges	Other
Base rate	$ 995,000	- fixed	52.7%	-- %	-- %
Metered water ...	472,000	- variable	--	24.5	--
User fees MHP ...	63,000	- fixed/variable ...	2.2	1.1	--
Standby chgs	219,000	- fixed	11.4	--	--
Interest	100,000	- variable	--	--	5.2
Other	38,000	- fixed/variable ...	--	--	1.9
Facilities chg ...	39,000	- variable	--	--	2.0
Totals	$1,926,000		65.3%	25.6%	9.1%

WATER RATE ALTERNATIVES

Meter Charges

Monthly or bimonthly meter charges are considered "availability" or "readiness-to-serve" charges for providing a customer with access to water at all times. These charges constitute a fixed portion of the customers water bill because they are levied whether or not water is used. This means, for example, that a customer who leaves his residence for an extended period will still pay some charge for water availability. The bimonthly meter charge compensates the Water Department for the costs it incurs in making the water available upon demand.

Bimonthly meter charges provide a steady, predictable revenue stream to water agencies, and reflect the fixed nature of a portion of operation and maintenance costs. Fixed costs are those that the utility has to pay whether or not usage materializes. They include: labor (salaries and benefits), capital costs associated with facilities, debt service, customer service costs, and other fixed O&M expenses. In the District's case, fixed costs constitute about 96 percent of total costs.

Some water agencies recover all of their costs through fixed charges. Fixed rate structures are attractive because they provide the water agency a guaranteed revenue stream, but because they do not charge based on consumption, they cannot use cost to influence water use. Ideally, water rates should contain a fixed charge to recover a portion of fixed costs and a quantity charge that is proportional to consumption. In mountain resort communities such as the District, the fixed component of customers' bills (i.e., bimonthly meter charges and standby charges) should be used to recover between one-half and two-thirds of operating expenses. The remaining costs should be recovered through consumption charges which provide cost incentives to encourage conservation.

Under the existing rates, all customers except the mobile home park pay the same bimonthly meter charge regardless of meter size.

Many water agencies levy bimonthly meter charges based on meter size. A base rate established by meter size recognizes that the water system's exposure varies according to meter size, with the larger size meter placing a potentially larger demand on the system. Actual water consumption will vary greatly between individual meters of the same size, depending on the property use, number of occupants, types of appliances, outside irrigation, and other factors.

An equitable and commonly accepted method of scaling meter charges by meter size is the American Water Works Association (AWWA) capacity ratios for maximum flow rate for continuous operations of meters. Table 14 summarizes the AWWA capacity ratios of larger meters to the 5/8" meter, and to the 5/8" and 3/4" meters.

By combining the 5/8" and 3/4" meters as the standard residential meter, a maximum flow rate of 25 gallons per minute (gpm) is derived. Use of the 25 gpm flow as the standard is preferred because a number of the District's residential customers are on 3/4" meters. If District households were served exclusively by 5/8" meters, then a standard residential flow of 20 gpm would be appropriate.

TABLE 14 ■ Big Bear City Community Services District Water Rate Review
Meter Charge Ratios*

BASED ON 5/8" METER

Meter Size	Meter Rating (gpm)	Ratio To 5/8" Meter
5/8"	20	1.0
3/4"	30	1.5
1"	50	2.5
1-1/2"	100	5.0
2"	160	8.0
3"	300	15.0
4"	500	25.0
6"	1,000	50.0

BASED ON 5/8" & 3/4" METERS
(Recommended)

Meter Size	Meter Rating (gpm)	Ratio To 5/8"&3/4" Meter
5/8" & 3/4"	25	1.0
1"	50	2.0
1-1/2"	100	4.0
2"	160	6.4
3"	300	15.0
4"	500	20.0
6"	1,000	40.0

*Based on AWWA capacity ratios for maximum flow rate for continuous operations of meters.

Uniform Block Rates

A uniform block rate is a single rate per unit of consumption charged for all volume use. Uniform block rates are typically used when simplicity of rate structure is desired or when water supply is not limited. Uniform block rates provide less incentive to conserve than inclining block rates.

Inclining Block Rates

An inclining block rate incorporates a unit charge that increases with increasing consumption. In California, where water supplies are limited, the use of inclining block rates is common. The primary purpose of implementing inclining block rates is to encourage conservation.

Due to its limited supplies, the District has already implemented a three-tier inclining block rate. Some water agencies have adopted inclining block rates with five, ten, or even more increasing tiers. The advantages of a three-tier structure are that (1) customers have a good indication as to whether their water use is relatively low (Tier 1), medium (Tier 2), or high (Tier 3); and (2) it is relatively simple to administer.

Seasonal Rates

Most water agencies experience peak demand for water in the summertime when outdoor use is prevalent. Some water agencies levy seasonal rates whereby higher rates are charged for water use during the peak season. The primary advantage of seasonal rates is that they provide year-round incentive to conserve. The main disadvantage of these rates is that they can be complicated and difficult to understand. Seasonal rates are typically used by agencies that experience extreme variations between peak and off-peak water use. Consumption data shows that the District's water use is about twice as high in the summer as in the winter.

Standby Charges

Standby charges are availability charges generally levied on a per-acre or per-parcel basis, and are collected on the property tax bill. The justification for the standby charge is that the utility incurs costs regardless of whether direct service is being rendered. The property owner has derived an immediate benefit from availability of water through an increase in property value attributable to that availability. The District has substantial investment in plant facilities and incurs cost in maintaining the availability of water service. Under Government Code §61765.9 the District can levy standby charges of up to $40 per acre or parcel per year. The Government Code stipulates that proceeds of the charges over $10 per acre or parcel must be used for water system improvements or debt service.

Most water agencies levy standby charges on undeveloped parcels only. The District, however, levies the charge on both developed and undeveloped parcels. This reflects the need to allocate fixed costs to and generate revenues from all users, particularly those who are part-time residents.

Fireflow Charges

Some water agencies assess a special fireflow charge for the benefits associated with provision of adequate fireflow protection. Based on AWWA guidelines, fireflow-related costs may be as high as 20 percent of total operating costs for a utility the size of the District. Most water agencies include the fireflow cost in their basic water rates. Some agencies levy special fireflow charges on customers with dedicated fire meters. The District has only one dedicated fire meter—at the airport.

Consumption Block Analysis

Consumption block analysis is a method of assessing water use patterns and projecting revenues for certain user groups. Table 15 presents a consumption block analysis for all users except the mobile home park. The consumption block is based on AWWA's bill tabulation methodology presented in the appendix of Water Rate Manual M1. Water Rate Manual M1 is a part of the District's record and is available for public inspection in the Water Department.

TABLE 15. BIG BEAR CITY COMMUNITY SERVICES DISTRICT WATER RATE REVIEW
SUMMARY OF BIMONTHLY BILLED USAGE FOR: 1994
USER GROUP ANALYZED: ALL USERS EXCEPT MOBILE HOME PARK

BARTLE WELLS ASSOCIATES
BLK-USE1.WK1

23-MAR-95

(1) BIMONTHLY USAGE BLOCK (CCF)	(2) NUMBER OF BILLS IN BLOCK	(3) CUMULATIVE BILLS THROUGH BLOCK	(4) TOTAL USE OF BILLS STOPPING IN BLOCK (CCF)	(%)	(5) CUMULATIVE USE OF BILLS STOPPING IN BLOCK (CCF)	(6) TOTAL USE TO THIS BLOCK OF ALL BILLS PASSING THROUGH (CCF)	(7) CUMULATIVE BILLED USAGE (CCF)	(%)	(8) TOTAL USAGE IN BLOCK (CCF)	(%)	REVENUE PROJECTION EXISTING RATES ($)	REVENUE PROJECTION VOLUME REVENUES EXISTING RATES ($)
TIER 1												
0	5,307	30,637	0	0.0%	0	0	0	0.0%	0	0.0%	1.20	0
3	7,649	25,330	22,947	6.2%	22,947	53,043	75,990	20.4%	75,990	20.4%	1.20	91,188
5	2,495	17,681	12,475	3.3%	35,422	75,930	111,352	29.9%	35,362	9.5%	1.20	42,434
8	2,572	15,186	20,576	5.5%	55,998	100,912	156,910	42.1%	45,558	12.2%	1.20	54,670
10	1,486	12,614	14,860	4.0%	70,858	111,280	182,138	48.8%	25,228	6.8%	1.20	30,274
12	1,456	11,128	17,472	4.7%	88,330	116,064	204,394	54.8%	22,256	6.0%	1.20	26,707
15	1,955	9,672	29,325	7.9%	117,655	115,755	233,410	62.6%	29,016	7.8%	1.20	34,819
18	1,562	7,717	28,116	7.5%	145,771	110,790	256,561	68.8%	23,151	6.2%	1.20	27,781
19	475	6,155	9,025	2.4%	154,796	107,920	262,716	70.5%	6,155	1.7%	1.20	7,386
20	423	5,680	8,460	2.3%	163,256	105,140	268,396	72.0%	5,680	1.5%	1.20	6,816
21	396	5,257	8,316	2.2%	171,572	102,081	273,653	73.4%	5,257	1.4%	1.20	6,308
23	648	4,861	14,904	4.0%	186,476	96,899	283,375	76.0%	9,722	2.6%	1.20	11,666
											Tier 1 Revs	340,050
TIER 2												
25	544	4,213	13,600	3.6%	200,076	91,725	291,801	78.3%	8,426	2.3%	1.50	12,639
27	455	3,669	12,285	3.3%	212,361	86,778	299,139	80.2%	7,338	2.0%	1.50	11,007
30	615	3,214	18,450	4.9%	230,811	77,970	308,781	82.8%	9,642	2.6%	1.50	14,463
35	673	2,599	23,555	6.3%	254,366	67,410	321,776	86.3%	12,995	3.5%	1.50	19,493
40	472	1,926	18,880	5.1%	273,246	58,160	331,406	88.9%	9,630	2.6%	1.50	14,445
											Tier 2 Revs	72,047
TIER 3												
45	344	1,454	15,480	4.2%	288,726	49,950	338,676	90.8%	7,270	1.9%	1.80	13,086
50	261	1,110	13,050	3.5%	301,776	42,450	344,226	92.3%	5,550	1.5%	1.80	9,990
55	165	849	9,075	2.4%	310,851	37,620	348,471	93.5%	4,245	1.1%	1.80	7,641
60	132	684	7,920	2.1%	318,771	33,120	351,891	94.4%	3,420	0.9%	1.80	6,156
65	98	552	6,370	1.7%	325,141	29,510	354,651	95.1%	2,760	0.7%	1.80	4,968
70	77	454	5,390	1.4%	330,531	26,390	356,921	95.7%	2,270	0.6%	1.80	4,086
80	123	377	9,840	2.6%	340,371	20,320	360,691	96.7%	3,770	1.0%	1.80	6,786
90	79	254	7,110	1.9%	347,481	15,750	363,231	97.4%	2,540	0.7%	1.80	4,572
100	45	175	4,500	1.2%	351,981	13,000	364,981	97.9%	1,750	0.5%	1.80	3,150
120	53	130	6,360	1.7%	358,341	9,240	367,581	98.6%	2,600	0.7%	1.80	4,680
150	35	77	5,250	1.4%	363,591	6,300	369,891	99.2%	2,310	0.6%	1.80	4,158
200	24	42	4,800	1.3%	368,391	3,600	371,991	99.8%	2,100	0.6%	1.80	3,780
250	18	18	4,500	1.2%	372,891	0	372,891	100.0%	900	0.2%	1.80	1,620
											Tier 3 Revs	74,673
Totals	30,637		372,891	100.0%					372,891	100.0%		486,770

Revenue Projection Adjustment (5%) 24,338

AVERAGE BIMONTHLY USE (CCF): 12.2

Total Projected Revenues $511,108

The consumption block shows the District's actual water use for calendar year 1994. As shown in the left-hand column of the table, District billing information is categorized into 30 bimonthly usage blocks ranging from 0 to 250 ccf. The total usage in each block is calculated and shown in Column 8. Based on the total usage in each block and the volume rates, volume revenues are calculated and shown in the far right-hand column. The model tends to underestimate total volume revenues because all bills in each block are multiplied by the minimum use in each block (e.g., all 24 bills in the 200-to-250-ccf block are assumed to use 200 ccf). Because of this, a revenue adjustment of 5 percent has been added to the total projected volume revenues. This yields total projected volume revenues of about $511,000 per year, which agrees well with prior audited water sales revenue.

Table 16 summarizes the results of the consumption block analysis. Tier 1 generates an estimated 70 percent of the District's volume revenues, while Tiers 2 and 3 each generate about 15 percent of volume revenues.

TABLE 16 ■ Big Bear City Community Services District Water Rate Review Summary of Consumption Block Analysis

Average bimonthly use per customer ..... 12.2 ccf

Tiered Usage	Percent of Water Bills	Percent of Water Use	Percent of Revenues
Tier 1	87%	77%	70%
Tier 2	8	13	15
Tier 3	5	10	15

RECOMMENDED RATES AND CHARGES

Meter Charges

Rather than use the same bimonthly charge for every water user regardless of the amount of water used, the District should rely on a bimonthly charge proportional to the amount of water used. The most commonly used method of estimating the amount of water used, is by a ratio of meter sizes serving each water customer.

A bimonthly fixed charge reflecting water consumption more equitably relates the charge to the demand a customer puts on the water system. The District must be ready 24 hours a day—every day—to serve users big and small. Big users, however, require more facilities than smaller users and should pay proportionately more.

Table 17 shows the meter size ratios of the various meters in current use within the District. These ratios should be applied to the fixed bimonthly charge to determine the bimonthly charge to large meters.

Because the size of a meter previously issued to customers didn't impact water rates, some adjustments must be made for customers whose water consumption is not accurately reflected by meter size. The following general rules should apply regarding the fixed bimonthly charge:

- All single-family residences without special considerations such as a second unit or large irrigation demands should pay the basic fixed bimonthly charge.
- Businesses and commercial users assigned large meters, but using small quantities of water should likewise pay based on a smaller meter size.
- Users with wide swings of seasonal use should be charged at the larger meter size because the water facilities must be sized to provide for peak water use.

TABLE 17 ■ **Big Bear City Community Services District Water Rate Review Bimonthly Meter Charge Revenue Model**

	Current Charges			Recommended Charges		
Meter Size	Active Meters	Bimonthly Charge	Annual Revenue	Capacity Ratio	Bimonthly Charge	Annual Revenue
5/8"	5,006	$ 32.50	$ 976,200	1.0	$ 32.50	$ 976,200
3/4"	67	32.50	13,100	1.0	32.50	13,100
1"	14	32.50	2,700	2.0	65.00	5,500
1-1/2"	4	32.50	800	4.0	130.00	3,100
2"	4	32.50	800	6.4	208.00	5,000
3"	2	32.50	400	12.0	390.00	4,700
4"	2	32.50	400	20.0	650.00	7,800
6"	1	6,851.00	41,100	40.0	1,300.00	7,800
Totals	5,100		$1,035,500			$1,023,200

Users with large meters would be, in effect, reserving a greater share of District facilities and water supply. For this reason, such users are entitled to more water within each tier. For the large meters, the amount of water available within each tier would be the standard amount times the meter size ratio. For example, a 1½" meter would be assigned a bimonthly meter charge 4 times that of the base 5/8" and 3/4" meters. The 1½" meter would be entitled to four times the base tier quantities of water for each tier (24 ccf x 4 = 96 ccf in the first tier).

Quantity Rates

Except for the large meter adjustments, the District's quantity rates do not require any modification at this time. Tiers provide a conservation incentive for customers to reduce water bills by saving water. The three tier structure allows the second tier to serve as a form of alert that water use is above the minimum billing level, but not yet at the highest rate. The second tier rate is 25 percent higher and the third tier is 50 percent higher than the base rate. These rate jumps give the conservation message without unduly penalizing higher water use. Many high-use customers may already be conserving water.

Because of the large meter adjustments for proportionately more water at the lower tiers, slightly less water (districtwide) will be sold at the higher tiers. This will be offset to a large degree by the fact that the mobile home park will now have less water at the lower tiers than before.

Standby Charges

The District should adjust the standby charge for unimproved parcels from $30 per parcel to $40 per parcel. No adjustment in the standby charge to improved parcels is recommended. The water rate review committee considered eliminating the standby charge on improved properties, but the result would be a loss in revenues of about $150,000 which would have to be funded by the very same customers in some other charge increase.

The increase in the standby charge to unimproved parcels represents the only charge differential between such lots and improved lots. Generally the purpose of a standby charge is to generate a separate revenue stream from future customers that aren't paying for facilities they may decide to use in the future. Standby charge revenues from unimproved properties will increase by about $20,000 if the recommendation is approved.

Rate Impacts by User Group

The recommended rate adjustments will impact only large water users. Residential customers and small users on 5/8" and 3/4" meters will not be affected. Tables 18 through 20 summarize the estimated impacts to larger meters. Customers classified as having a 1", 1½", or 2" meter will typically see a 14 to 29 percent increase in annual costs. Users that have large seasonal water use fluctuations will see the largest percentage increase. Customers on 3" meters are projected to see a 41 to 49 percent increase due to their large seasonal water use fluctuations. Customers on 4" meters would see increases varying from 8 to 49 percent. The mobile home park, with a 6" meter, would see a reduction in costs of about 39 percent, due to lower fixed charges.

TABLE 18. BIG BEAR CITY COMMUNITY SERVICES DISTRICT WATER RATE REVIEW
TYPICAL RATE IMPACTS FOR 1", 1-1/2" AND 2" METERS

1" METER

Meter Size: 1"
Ratio to 5/8" Meter: 2

	Current Rates	Current CCF	Proposed Rates	Proposed CCF
Tier 1:	$1.20	24	$1.20	48
Tier 2:	1.50	40	1.50	80
Tier 3:	1.80	>40	1.80	>80

Billing Period	1994 Usage (ccf)	Current Charges: Bimonthly Serv Chg	Current Charges: Volume Charge	Current Charges: Total Bill	Proposed Charges: Bimonthly Serv Chg	Proposed Charges: Volume Charge	Proposed Charges: Total Bill	Amount	Percent
1	30	$32.50	$37.80	$70.30	$65.00	$36.00	$101.00	$30.70	
2	27	32.50	33.30	65.80	65.00	32.40	97.40	31.60	
3	59	32.50	87.00	119.50	65.00	74.10	139.10	19.60	
4	62	32.50	92.40	124.90	65.00	78.60	143.60	18.70	
5	42	32.50	56.40	88.90	65.00	50.40	115.40	26.50	
6	30	32.50	37.80	70.30	65.00	36.00	101.00	30.70	
Total	250	195.00	344.70	539.70	390.00	307.50	697.50	157.80	29.2%

1-1/2" METER

Meter Size: 1-1/2"
Ratio to 5/8" Meter: 4

	Current Rates	Current CCF	Proposed Rates	Proposed CCF
Tier 1:	$1.20	24	$1.20	96
Tier 2:	1.50	40	1.50	160
Tier 3:	1.80	>40	1.80	>160

Billing Period	1994 Usage (ccf)	Current Charges: Bimonthly Serv Chg	Current Charges: Volume Charge	Current Charges: Total Bill	Proposed Charges: Bimonthly Serv Chg	Proposed Charges: Volume Charge	Proposed Charges: Total Bill	Amount	Percent
1	122	$32.50	$200.40	$232.90	$130.00	$154.20	$284.20	$51.30	
2	109	32.50	177.00	209.50	130.00	134.70	264.70	55.20	
3	235	32.50	403.80	436.30	130.00	346.20	476.20	39.90	
4	248	32.50	427.20	459.70	130.00	369.60	499.60	39.90	
5	166	32.50	279.60	312.10	130.00	222.00	352.00	39.90	
6	120	32.50	196.80	229.30	130.00	151.20	281.20	51.90	
Total	1,000	195.00	1,684.80	1,879.80	780.00	1,377.90	2,157.90	278.10	14.8%

2" METER

Meter Size: 2"
Ratio to 5/8" Meter: 6.4

	Current Rates	Current CCF	Proposed Rates	Proposed CCF
Tier 1:	$1.20	24	$1.20	153.6
Tier 2:	1.50	40	1.50	256
Tier 3:	1.80	>40	1.80	>256

Billing Period	1994 Usage (ccf)	Current Charges: Bimonthly Serv Chg	Current Charges: Volume Charge	Current Charges: Total Bill	Proposed Charges: Bimonthly Serv Chg	Proposed Charges: Volume Charge	Proposed Charges: Total Bill	Amount	Percent
1	146	$32.50	243.60	$276.10	$208.00	175.20	$383.20	$107.10	
2	131	32.50	216.60	249.10	208.00	157.20	365.20	116.10	
3	282	32.50	488.40	520.90	208.00	384.72	592.72	71.82	
4	298	32.50	517.20	549.70	208.00	413.52	621.52	71.82	
5	199	32.50	339.00	371.50	208.00	252.42	460.42	88.92	
6	144	32.50	240.00	272.50	208.00	172.80	380.80	108.30	
Total	1,200	195.00	2,044.80	2,239.80	1,248.00	1,555.86	2,803.86	564.06	25.2%

TABLE 19. BIG BEAR CITY COMMUNITY SERVICES DISTRICT WATER RATE REVIEW
RATE IMPACTS TO 3" METERS

Name:	High School		Current		Proposed	
			Rates	CCF	Rates	CCF
Meter Size:	3"	Tier 1:	$1.20	24	$1.20	288
Ratio to 5/8" Meter:	12	Tier 2:	1.50	40	1.50	480
		Tier 3:	1.80	>40	1.80	>480

Billing Period	1994 Usage (ccf)	Current Charges			Proposed Charges			Increase/(Decrease)	
		Bimonthly Serv Chg	Volume Charge	Total Bill	Bimonthly Serv Chg	Volume Charge	Total Bill	Amount	Percent
1	10	$32.50	$12.00	$44.50	$390.00	$12.00	$402.00	$357.50	
2	460	32.50	808.80	841.30	390.00	603.60	993.60	152.30	
3	1160	32.50	2,068.80	2,101.30	390.00	1,857.60	2,247.60	146.30	
4	160	32.50	268.80	301.30	390.00	192.00	582.00	280.70	
5	20	32.50	24.00	56.50	390.00	24.00	414.00	357.50	
6	0	32.50	0.00	32.50	390.00	0.00	390.00	357.50	
Total	1,810	195.00	3,182.40	3,377.40	2,340.00	2,689.20	5,029.20	1,651.80	48.9%

Name:	Athletic Facility		Current		Proposed	
			Rates	CCF	Rates	CCF
Meter Size:	3"	Tier 1:	$1.20	24	$1.20	288
Ratio to 5/8" Meter:	12	Tier 2:	1.50	40	1.50	480
		Tier 3:	1.80	>40	1.80	>480

Billing Period	1994 Usage (ccf)	Current Charges			Proposed Charges			Increase/(Decrease)	
		Bimonthly Serv Chg	Volume Charge	Total Bill	Bimonthly Serv Chg	Volume Charge	Total Bill	Amount	Percent
1	0	$32.50	$0.00	$32.50	$390.00	$0.00	$390.00	$357.50	
2	114	32.50	186.00	218.50	390.00	136.80	526.80	308.30	
3	798	32.50	1,417.20	1,449.70	390.00	1,206.00	1,596.00	146.30	
4	898	32.50	1,597.20	1,629.70	390.00	1,386.00	1,776.00	146.30	
5	196	32.50	333.60	366.10	390.00	235.20	625.20	259.10	
6	35	32.50	45.30	77.80	390.00	42.00	432.00	354.20	
Total	2,041	195.00	3,579.30	3,774.30	2,340.00	3,006.00	5,346.00	1,571.70	41.6%

TABLE 20. BIG BEAR CITY COMMUNITY SERVICES DISTRICT WATER RATE REVIEW
RATE IMPACTS TO 4" AND 6" METERS

4" METERS

Name:	High School		Current		Proposed	
			Rates	CCF	Rates	CCF
Meter Size:	4"	Tier 1:	$1.20	24	$1.20	480
Ratio to 5/8" Meter:	20	Tier 2:	1.50	40	1.50	800
		Tier 3:	1.80	>40	1.80	>800

Billing Period	1994 Usage (ccf)	Current Charges: Bimonthly Serv Chg	Current Charges: Volume Charge	Current Charges: Total Bill	Proposed Charges: Bimonthly Serv Chg	Proposed Charges: Volume Charge	Proposed Charges: Total Bill	Increase/(Decrease): Amount	Increase/(Decrease): Percent
1	305	$32.50	$529.80	$562.30	$650.00	$366.00	$1,016.00	$453.70	
2	2031	32.50	3,636.60	3,669.10	650.00	3,271.80	3,921.80	252.70	
3	5379	32.50	9,663.00	9,695.50	650.00	9,298.20	9,948.20	252.70	
4	4920	32.50	8,836.80	8,869.30	650.00	8,472.00	9,122.00	252.70	
5	760	32.50	1,348.80	1,381.30	650.00	996.00	1,646.00	264.70	
6	315	32.50	547.80	580.30	650.00	378.00	1,028.00	447.70	
Total	13,710	195.00	24,562.80	24,757.80	3,900.00	22,782.00	26,682.00	1,924.20	7.8%

Name:	Park		Current		Proposed	
			Rates	CCF	Rates	CCF
Meter Size:	4"	Tier 1:	$1.20	24	$1.20	480
Ratio to 5/8" Meter:	20	Tier 2:	1.50	40	1.50	800
		Tier 3:	1.80	>40	1.80	>800

Billing Period	1994 Usage (ccf)	Current Charges: Bimonthly Serv Chg	Current Charges: Volume Charge	Current Charges: Total Bill	Proposed Charges: Bimonthly Serv Chg	Proposed Charges: Volume Charge	Proposed Charges: Total Bill	Increase/(Decrease): Amount	Increase/(Decrease): Percent
1	59	$32.50	$87.00	$119.50	$650.00	$70.80	$720.80	$601.30	
2	566	32.50	999.60	1,032.10	650.00	705.00	1,355.00	322.90	
3	957	32.50	1,703.40	1,735.90	650.00	1,338.60	1,988.60	252.70	
4	1341	32.50	2,394.60	2,427.10	650.00	2,029.80	2,679.80	252.70	
5	21	32.50	25.20	57.70	650.00	25.20	675.20	617.50	
6	0	32.50	0.00	32.50	650.00	0.00	650.00	617.50	
Total	2,944	195.00	5,209.80	5,404.80	3,900.00	4,169.40	8,069.40	2,664.60	49.3%

6" METER

Name:	Mobile Home Park		Current		Proposed	
			Rates	CCF	Rates	CCF
Meter Size:	6"	Tier 1:	$1.20	24	$1.20	960
Ratio to 5/8" Meter:	40	Tier 2:	1.50	40	1.50	1600
		Tier 3:	1.80	>40	1.80	>1600

Billing Period	1994 Usage (ccf)	Current Charges: Bimonthly Serv Chg	Current Charges: Volume Charge	Current Charges: Total Bill	Proposed Charges: Bimonthly Serv Chg	Proposed Charges: Volume Charge	Proposed Charges: Total Bill	Increase/(Decrease): Amount	Increase/(Decrease): Percent
1	2173	$6,851.00	2,607.60	$9,458.60	$1,300.00	3,143.40	$4,443.40	($5,015.20)	
2	2561	6,851.00	3,073.20	9,924.20	1,300.00	3,841.80	5,141.80	(4,782.40)	
3	4332	6,851.00	5,198.40	12,049.40	1,300.00	7,029.60	8,329.60	(3,719.80)	
4	5056	6,851.00	6,067.20	12,918.20	1,300.00	8,332.80	9,632.80	(3,285.40)	
5	3598	6,851.00	4,317.60	11,168.60	1,300.00	5,708.40	7,008.40	(4,160.20)	
6	2608	6,851.00	3,129.60	9,980.60	1,300.00	3,926.40	5,226.40	(4,754.20)	
Total	20,328	41,106.00	24,393.60	65,499.60	7,800.00	31,982.40	39,782.40	(25,717.20)	−39.3%

Appendix M

Drought Contingency Plan

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Big Bear City Community Services District
Drought Contingency Plan

1. Declaration of Purpose and Principles

The Big Bear City Community Services District (the "District") provides water service to persons within the District. The District's potable water resources are limited and should be preserved, especially during times of declared water shortages, and therefore, the Board has determined that it is necessary to adopt a water conservation program.

The District is authorized by Water Code Section 350 et seq. to declare a water shortage emergency and by Water Code Sections 375-378 to adopt water conservation programs. The District declared a water shortage within the District after a public hearing at a regular meeting of the District on April 15, 2002 in Ordinance No. 205. Under State law the District is authorized after declaration of a water shortage emergency to restrict the use of District water and to prohibit the waste or use of District water during such periods for any purpose other than domestic use, sanitation, fire protection or such other uses as may be determined by the District to be necessary.

The District finds and determines that the adoption of water conservation rules and regulations are necessary to:

1. Protect the health, safety, and welfare of the inhabitants of the District,

2. Assure the maximum beneficial use of the water supplies of the District, and

3. Assure that there will be sufficient water supplies to meet the basic needs of human consumption, sanitation, and fire protection.

The District finds that the specific rules, regulations, and restrictions established herein are necessary in the event of a water supply shortage.

2. Public Education

The District will periodically provide the public with information about the Plan, including conditions under which stage of the Plan is to be initiated or terminated and the conservation response

measures to be implemented in each stage. This information will be provided by means of public events, website, press releases, bill inserts, etc.

3. Coordination with Regional Water Planning Groups

In the event that a potential water shortage is projected, the District will make efforts to reasonably communicate with and, as appropriate, collaborate with local, county, regional, state, and federal water planning groups and agencies.

4. Shortage Declaration Process

The following section discusses the District's four water supply shortage stages and their declaration processes.

The District shall monitor the supply and demand for water regularly during mandatory compliance water supply shortage stages to determine the level of conservation required by the implementation or termination of each stage. Each declaration of the Board implementing or terminating a water conservation stage shall be published at least once in a newspaper of general circulation, and shall remain in effect until the Board so otherwise declares.

A. Stage 1 - Threatened Water Shortage

The Board may, following a noticed public hearing, declare a Stage 1 Threatened Water Shortage based on an evaluation of conditions that have the potential to impair the District's ability to meet the water demands of its customers such as depth to water in supply wells, the ability to blend or treat supply water to below the State Maximum Contaminant Level (MCL), duration of drought, time of year, and the potential for existing recharge. Declaration of a Stage 1 Water Shortage Alert will be recited in a motion, resolution or ordinance declaring a Threatened Water Shortage. No public hearing shall be required in the event of a Water Supply Shortage Emergency.

B. Stage 2 - Water Shortage Alert

The Board may, following a noticed public hearing, declare a Stage 2 Water Shortage Alert based on based on an evaluation of conditions that have the potential to further impair the District's ability to meet the water demands of its customers such as (as a minimum) depth to water in supply wells, the ability to blend or treat supply water to below the State Maximum Contaminant Level (MCL), duration of drought, time of year, and the potential for existing recharge Declaration of a Stage 2 Water Shortage Alert will be recited in a motion, resolution

or ordinance declaring a Water Shortage Alert. No public hearing shall be required in the event of a Water Supply Shortage Emergency.

C. Stage 3 - Water Shortage Warning

The Board may, following a noticed public hearing, declare a Stage 3 Water Shortage Alert based on based on an evaluation of conditions that have the potential to greatly impair the District's ability to meet the water demands of its customers such as (as a minimum) depth to water in supply wells, the ability to blend or treat supply water to below the State Maximum Contaminant Level (MCL), duration of drought, time of year, and the potential for existing recharge Declaration of a Stage 3 Water Shortage Alert will be recited in a motion, resolution or ordinance declaring a Water Shortage Warning. No public hearing shall be required in the event of a Water Supply Shortage Emergency.

D. Stage 4 - Water Supply Shortage Emergency

The Board may, following a noticed public hearing, declare a Stage 4 Water Shortage Alert based on based on an evaluation of conditions that have the potential to impair the District's ability to meet the water demands of its customers such that a water supply shortage is threatened. Conditions that will be evaluated include (as a minimum) depth to water in supply wells, the ability to blend or treat supply water to below the State Maximum Contaminant Level (MCL), duration of drought, time of year, and the potential for existing recharge. Declaration of a Stage 4 Water Shortage Alert will be recited in a motion, resolution or ordinance declaring a Threatened Water Supply Shortage Emergency. No public hearing shall be required in the event of an Emergency Water Supply Shortage.

5. Criteria for Terminating Water Shortage Contingency Stages

The Board is authorized to terminate any of the water shortage contingency stages (Stages 1 through 4) based upon an appropriate change in the conditions evaluated at the onset of the water shortage contingency stages. The order to terminate any water supply shortage stage shall be published a minimum of one time in a newspaper of general circulation in the District.

6. Water Supply Shortage Stages Measures and Restrictions

The following section discusses the measures and restrictions for each of the District’s four water supply shortage stages.

A. Stage 1 Restrictions. The following restrictions and prohibitions on water use shall be imposed and complied with by the District Water Users during Stage 1. During Stage 1, the District may impose any or all of the following restrictions or conditions on the use of water, as the circumstances may require:

1. *Landscape Irrigation.*

   a. Landscape watering with sprinklers between 9:00 a.m. and 6:00 p.m. is prohibited. This prohibition extends to all nurseries, golf courses, and public facilities.

      1. Landscape irrigation must be limited to what is necessary and shall not be excessive.

      2. This prohibition shall not extend to those facilities using subsurface or drip irrigation.

      3. Water shall not run off into the streets or other impervious surfaces.

   b. Turf irrigation is prohibited between November 1 and April 1 of each year.

2. *Aesthetic uses.*

   a. Decorative ponds, fountains and waterways having a capacity in excess of twenty (20) gallons shall not be filled with water from the District's water system.

3. *Other outdoor uses.*

   a. Waste is prohibited.

   b. Sidewalks, driveways, buildings, and windows shall not be washed off with hoses, except as required for sanitary purposes. Reduction of washing of other impervious surfaces is recommended.

   c. Noncommercial washing of automobiles, trucks, trailers, boats, airplanes and other mobile equipment shall be done with a handheld hose or bucket. Handheld hoses shall be equipped with an automatic shutoff nozzle.

4. *Commercial and industrial uses.*

   a. Washing of automobiles, trucks, trailers, boats, airplanes and other mobile equipment is permitted.

   b. Restaurants shall not to serve water to customers except when specifically requested by the customer.

c. Voluntary reduction of water use in commercial and industrial processes is recommended.

d. All new commercial car, airplane, and boat washes shall be constructed with recycling water systems.

5. *Domestic use.*

Voluntary reduction for indoor domestic use is recommended by any means available.

6. *Essential and utility uses.*

a. *Fire fighting.* No restrictions.

b. *Medical use by health care facilities.* No restrictions.

7. No water shall be used for private construction purposes, including water used for dust control or compaction. Use of reclaimed or reused water is exempt from this reduction measure.

8. All water leaks shall be repaired immediately.

B. Stage 2 Restrictions. During Stage 2, the following restrictions and prohibitions on water uses shall be imposed on and complied with by all Water Users and are in addition to, and include, all water uses restricted or prohibited in Stage 1.

1. The District recommends that all existing customers with landscaping presently in place reduce their turf area and/or replace it with low water use plants and drip irrigation, or native plants that need little or no watering.

2. Any person installing new landscaping shall not install turf in excess of twenty five (25) percent of the available landscape area and shall not install more than one thousand (1,000) square feet of turf and shall use low water plants. Drip irrigation is highly recommended for use in new landscaping projects.

3. Persons may irrigate on Designated Days only.

C. Stage 3 Restrictions. In addition to the mandatory requirements set forth in Stages 1 and 2, the following restrictions shall be imposed on and complied with by all Water Users during Stage 3.

1. No residences shall use in excess of one hundred (100) gallons per day. This amount may, upon petition to the District, be increased by fifty (50) gallons per day

per person for residences which the District determines based on the facts set forth in petition have more than two (2) persons living full time in a single residence.

2. Mandatory water rationing levels shall be determined for commercial establishments by the District's General Manager on a case-by-case basis and such determination. Once such a determination is made, commercial establishments shall not exceed use limitations so established.

3. All Water Users shall reduce turf to twenty-five (25) percent of the available landscape area which shall not exceed more than one thousand (1,000) square feet of turf on a parcel or lot.

4. No Water User shall install any new landscaping.

5. Any Water User who exceeds the one hundred (100) gallons per day per residence limitation, as adjusted for additional residence members, or the commercial water usage limitation as determined by the District, shall pay a water surcharge equal to twice the amount of the Water User's entire water bill at standard rates.

D. Stage 4 Restrictions. When Stage 4 is declared, the following restrictions and prohibitions shall be imposed and complied with. Such restrictions are in addition to all restrictions imposed in Stages 1, 2, and 3.

1. No Water User may irrigate turf or landscape plants.

2. No Water User may irrigate trees and shrubs except by drip irrigation or hand watering.

3. No new landscaping plants shall be installed, except such landscaping plants as are required for erosion control by the County Building and Safety Department.

4. In addition to the surcharge charged under Stage 3, the District also may install a flow restrictor on property where a Water User is wasting water and/or shut-off water service for those water users who repeatedly exceed the District's mandatory water use limitation of 100 gallons per day per residence, as adjusted for additional residence members. The provisions for a shut-off of water service are defined in Ordinance No. 75, adopted by the Board of Directors on October 13, 1987.

5. All washing of cars, airplanes, and boats or any other vehicles is prohibited. This prohibition does not include washing by a commercial establishment that uses solely reclaimed water to wash the vehicles.

6 In industrial, commercial and other uses, additional restrictions may be imposed if conditions warrant.

7. Expected Water Savings due to Water Supply Shortage Stages

The following section discusses the expected water savings due to each of the District's four water supply shortage stages.

A. The District expects to realize a goal of a five (5) to ten (10) percent reduction in overall water usage after a Stage 1 condition is declared by the Board.

B. The District expects to realize a goal of a fifteen (15) percent reduction in overall water usage after a Stage 2 condition is declared by the Board.

C. The District expects to realize a goal of a twenty-five (25) percent reduction in overall water usage after a Stage 3 condition is declared by the Board.

D. The District shall impose water conservation measures that will result in a forty (40) percent reduction in overall water use after a Stage 4 condition is declared by the Board.

8. Application

The water shortage contingency measures of this Plan shall apply to all persons, customers, and properties utilizing potable water provided by the District. The terms "person" and "customer" as used in the Plan include individuals, corporations, partnerships, agencies, associations, and all other legal entities.

9. Definitions

For the purposes of this Plan, the following definitions shall apply:

A. "Aesthetic use" means the use of water for fountains, waterfalls and landscape lakes and ponds where the use is entirely ornamental and serves no other functional purpose.

B. "Board" means the Board of Directors of the District.

C. "Commercial and industrial uses" means the use of water integral to the production of primary goods and services provided by industrial or commercial facilities. Industrial facilities include facilities that perform the process-specific activities including cooling, oiler feed, cleaning and washing, pollution control, extraction and separation of desirable material from products and waste materials and the incorporation of water into final products. Commercial facilities include, but are not limited to, food service facilities, hotels, retail facilities and nursery operations.

D. "Designated Days" means a water use restriction limiting the days on which water may be used for landscape irrigation. The following system applies to determine the day within each calendar week that the use is permitted:

1. Residences, buildings and premises with even-numbered street addresses may use water as described in this Chapter on even numbered calendar days.

2. Residences, buildings and premises with odd-numbered street addresses: may use water as described in this Chapter on odd numbered calendar days.

E. "District" means the Big Bear City Community Services District.

F. "Domestic use" means the use of water, other than outdoor uses, for personal needs or for household purposes, including drinking, bathing, cooling, heating, cooking, sanitation or cleaning, whether the use occurs in a residence or in a commercial or industrial facility.

G. "Drip Irrigation" means a method of irrigation whereby water is applied directly to the base or roots of the plant without the loss of water to surrounding dirt or vegetation.

H. "Existing facility" means a swimming pool, hot tub or any similar facility, including residential and private facilities, installed during any period for which a water conservation plan stage has not been declared or during a stage which, together with all preceding and succeeding stages, has been rescinded. This term does not include pools specifically maintained to provide habitat for aquatic life.

I. "Existing landscaping plant" means a landscaping plant planted during any period for which a water conservation plan stage has not been declared or during a stage which, together with all preceding and succeeding stages, has been rescinded.

J. "Handheld hose" means a hose attended by one person, fitted with a manual or automatic shutoff nozzle.

K. "Health care facility" means any hospital, clinic, nursing home or other health care or medical research facility.

L. "Impervious surface area" means any structure or any street, driveway, sidewalk, patio or other surface area covered with brick, paving, tile or other impervious material.

M. "Landscape irrigation" means the application of water to grow landscaping plants.

N. "Landscaping plant" means any plant, including any tree, shrub, vine, herb, flower, succulent, ground cover or grass species, that is used for landscaping purposes or for the support of intensive recreational areas including playgrounds and playing fields.

O. "New facility" means a swimming pool, hot tub or any similar facility, including residential and private facilities, installed during any water conservation plan stage. When that stage, together with all other stages which precede or succeed that stage in a continuous time period, is rescinded, the new facility will be treated thereafter as an existing facility. This term does not include wading pools or pools specifically maintained to provide habitat for aquatic life.

P. "New landscaping plant" means a landscaping plant planted during any water conservation plan stage. When that stage, together with all other stages which precede or succeed that stage in a continuous time period, is rescinded, the new landscaping plant will be treated thereafter as an existing landscaping plant.

Q. "Other outside use" means the use of water outdoors for the maintenance, cleaning and washing of structures and mobile equipment, including automobiles and boats, and the washing of streets, driveways, sidewalks, patios and other similar areas.

R. "Reclaimed Water" or "Reclaimed or Reused Water" means any water which, as a result of the treatment of domestic wastewater, is suitable for a direct beneficial use or a controlled use that would not otherwise occur.

S. "Turf" means any landscaped area that sustains primarily varieties of grasses.

T. "Water" means water supplied by the District.

U. "Waste" means any unreasonable or nonbeneficial use of water, or any unreasonable method or use of water, including, but not limited to, the specific uses prohibited and restricted by this Plan as hereinafter set forth. Waste includes, but is not limited to, allowing water to run off into a gutter, ditch or drain or failing to repair a controllable leak.

V. "Water Supply Shortage Emergency" means any Water Supply Shortage caused by an earthquake, loss of electrical power, pipeline breakage, or any other threatened or existing water shortage caused by a disaster or facility failure which results in the District's inability to meet the water demands of its customers.

W. "Water Supply Shortage" means a water shortage such that the ordinary demands and requirements of water customers cannot be satisfied without depleting the water supply to the extent that there would be insufficient water for human consumption, sanitation, and fire protection.

X. "Water User" means any person, firm, partnership, association, corporation or political entity using water obtained from the water system of the District.

10. Public Notification Procedures

When the District determines that a potable water shortage condition exists, any or all of the following notification procedures may be implemented:

A. Notify the general public and influential local decision-makers what the situation is, actions to be taken, what the customers are intended to achieve, and how these actions are to be implemented.

B. The public at large will be informed of the situation and what must be done. Contact can be made through billing inserts, special mailings, telephone contact, e-mail, roadway signage, water conservation booths, speaker's bureau, community association meetings, newsletters, and education programs, etc. Literature should be provided on the potable water shortage condition, conservation methods, and water-saving devices.

C. Use of media in all its available forms will be employed as appropriate. This would include public service announcements on radio and cable television as well as press releases in local newspapers.

D. Posting of all pertinent information on the District's web page.

11. Enforcement

Failure to comply with the provisions of this Plan shall constitute a misdemeanor punishable under Water Code Section 377.

A person who knowingly or intentionally violates or causes the violation of any restriction or prohibition on the use of water or related activities contained in any stage of this Plan shall be deemed guilty of a misdemeanor and, upon conviction, shall be punished by a fine as follows:

1. First Offense: Fifty dollars ($50.00)

2. Second Offense: One hundred dollars ($100.00)

3. Third Offense: Two hundred fifty dollars ($250.00)

4. Fourth Offense and Subsequent Offenses: Five hundred dollars ($500.00) and/or by imprisonment in the county jail for not more than thirty (30) days.

In addition to the provisions of this Section, the District reserves the right to take such civil enforcement action or other action as may be available or appropriate to compel compliance with the provisions of this Plan.

12. Exceptions

A. Application for Exception Permit. The District may grant permits for uses of water otherwise prohibited under this Plan if it finds and determines that special circumstances make compliance impossible or that restrictions herein would either:

1. Cause an unnecessary and undue hardship to the Water User or the public; or

2. Cause an emergency condition affecting the health, sanitation, fire protection or safety of the Water User or the public.

B. Such exceptions may be granted only upon application therefore. Upon granting any such exception permit, the District may impose any conditions it determines to be just and proper.

C. The District may also grant a one-time extension in the following circumstances:

1. Use of water to establish lawn or other vegetation requiring seed germination.

2. Use of water blasting as preparation for house painting.

3. Use of water to prepare for driveway sealing.

13. Reservations of Rights

The rights of the District hereunder shall be cumulative to any other right of the District to discontinue service. All monies collected by the District pursuant to any of the penalty provisions of this Plan shall be deposited in the Operating Fund as reimbursement for the District's costs and expenses of administering and enforcing this Plan.

14. Water Connection Limitations

A. Stages 1 and 2. Under Water Supply Shortage Stages 1 and 2, the District will allow a maximum of twenty-four (24) connections per quarter not to exceed ninety-six (96) new water connections within each twelve (12) month period following the determinations that Stages 1 and 2 water conditions exist. Under Stages 1 and 2, the District will allow each customer a maximum of three (3) new service connections per quarter.

B. Stage 3. Under Water Supply Shortage Stage 3, the District will allow a maximum of no more than eighteen (18) new water connections per quarter, not to exceed seventy-two (72) new water connections within each twelve (12) month period following the determination that Stage 3 water conditions exist. Under Stage 3, the District will allow each customer a maximum of two (2) new service connections per quarter. In Stage 3, the Board may limit connections to less than seventy-two (72) if it determines that conditions so warrant. In Stage 3 the actual reduction in new connections shall be determined by the Board at a noticed public hearing.

C. Stage 4. Under Water Supply Shortage Stage 4, the District will reduce new water connections to a maximum of no more than fifteen (15) new water connections per quarter not to exceed sixty (60) new water connections within each twelve (12) month period following the determination that Stage 4 conditions exist, if it determines that conditions so warrant. Under Stage 4, the District will allow each customer a maximum of one (1) new service connection per quarter. In Stage 4 the actual reduction in new connections shall be determined by the Board at a noticed public hearing.

D. For purposes of this Section, the term quarter means the three-calendar-month period ending on March 31, June 30, September 30, or December 31 of any year.