City of Cheyenne
Board of Public Utilities

Volume 3 – Source Water Supply and Delivery

2013 Cheyenne Water and Wastewater Master Plans

Final

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Abbreviations and Acronyms

ac-ft	Acre-feet
ac-ft/year	Acre-feet per year
ASR	Aquifer Storage and Recovery
BOPU	Board of Public Utilities
cfs	Cubic feet per second
CY	Cubic yards
DTW	Depth to Water
ft	Feet
gal	Gallon
gpd	Gallons per day
gpm	Gallons per Minute
HDPE	High density polyethylene
LCGCA	Laramie County Groundwater Control Area
LS	Lump Sum
mgd	Million Gallons per Day
mi	Miles
NCSA	North Cheyenne Study Area
NMPL	normal maximum pool level
O&M	Operations and Maintenance
PLCC	Planning Level Estimate of Construction Cost
PRV	Pressure reducing valve
psi	pounds per square inch
PVC	Polyvinyl chloride
RCC	Roller compacted concrete
SCADA	Supervisory control and data acquisition
SDF	Sediment Dewatering Facility

SEO	Wyoming State Engineer's Office
SWSS	Surface Water Supply System Model
TCE	trichloroethylene
TRS	Township Range and Section
USACE	US Army Corps of Engineers
USGS	US Geological Survey
WTP	Water Treatment Plant
WYDOT	Wyoming Department of Transportation
WDEQ	Wyoming Department of Environmental Quality
WWDC	Wyoming Water Development Commission
Y/N	Yes or No
yr	Year

3.1 Introduction

Adequate raw water supplies are critical to the future operation of BOPU. This volume of the Master Plan provides an assessment of raw water supply availability from both surface and groundwater sources. Sustainability of the existing raw water sources and delivery infrastructure was evaluated using a variety of data sources including historical hydrological data, current operating policies, well field logs and projected future demands.

3.1.1 Raw Water System Overview

The raw water collection system for BOPU is extensive, involving several surface storage reservoirs, pipelines from multiple watersheds providing surface water, well fields supplying groundwater, and arrangements for water exchanges between watersheds.

Surface water sources come from multiple watersheds. The local Crow Creek Basin provides for roughly 20% of the raw water supply. Crystal Lake Reservoir and Granite Springs Reservoir both collect native water from the Middle Crow Creek Basin, as well as storing non-native water brought through the Stage I and Stage II pipelines from Rob Roy Reservoir. Intake pipelines to the Sherard Water Treatment Plant (WTP) begin at the Crystal Lake Reservoir dam. Smaller reservoirs on the North Fork of Crow Creek (Old North Crow and Upper North Crow reservoirs) and South Fork of the Crow Creek (South Crow reservoir) collect water that can be transmitted to Round Top for use as irrigation water. Pipeline pressures from the smaller reservoirs are too low at the Wye to compete with the transmission pipeline pressures to Sherard WTP.

Additional surface water is imported from the Douglas Creek watershed through the Stage I and II pipeline system. The system starts at Rob Roy Reservoir in the North Platte drainage basin. A minimum flow release of 5.5 cfs to Douglas Creek is required from Rob Roy Reservoir. The remainder of the stored water may be diverted into the Stage I/II pipelines that transfer water to Lake Owen and then onward to Granite Springs and Crystal Lake Reservoirs. A schematic diagram of the surface water collection system is shown in Figure 3-1.

Prior to any diversion, impacts to downstream North Platte basin water right holders must be prevented. BOPU operates a diversion tunnel that transfers water from the Little Snake river watershed in the Yampa-White basin into the North Platte basin. This imported water is stored in Hog Park Reservoir. Hog Park Reservoir passes a minimum flow of 15 cfs downstream, with additional flows released based on natural runoff. If inadequate water is stored in Hog Park Reservoir or the U.S. Forest Service restricts the amount of water that can be released at a given time, it may not be possible to offset Stage I/II diversion impacts on North Platte water users using Hog Park Reservoir alone. BOPU has contracted 15,700 acre-feet of storage space in the U.S. Bureau of Reclamation-operated Seminoe Reservoir as a secondary supply for offsets.

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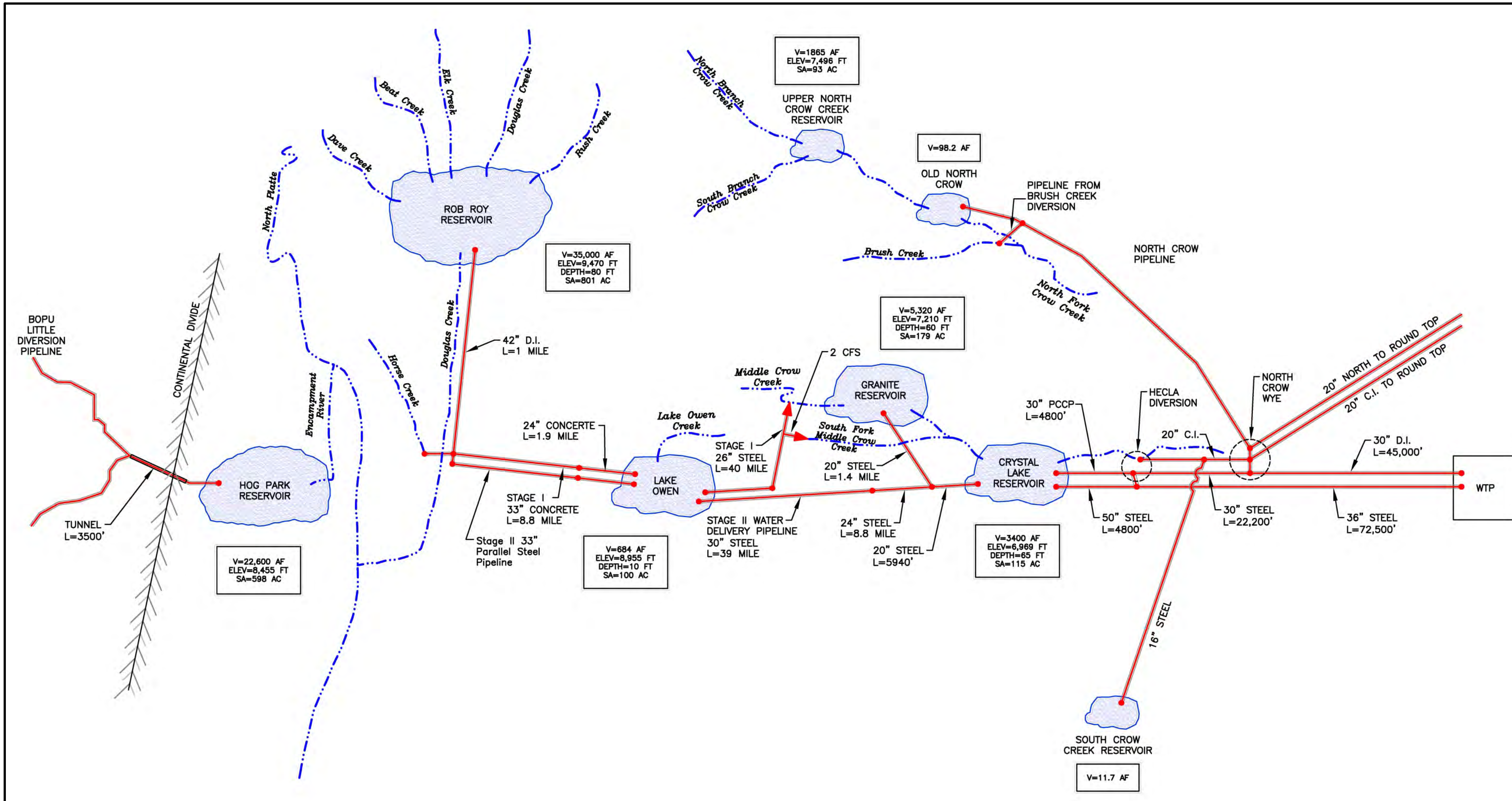
Figure 3-1
Existing Surface Water Supply System Schematic



Groundwater currently makes up approximately 30% of the total raw water supply, sourced primarily from the Ogallala aquifer. A total of 35 wells are located in four well fields: namely the Bell, Happy Jack, Borie, and Federal. Groundwater provides supplemental water for fluctuating surface water availability. Additionally maintaining a minimum blend of groundwater improves water quality in the finished water.

Figure 3-2 shows the contribution from various sources to potable water supply. Groundwater is generally proportional to the total water supply while surface water sources may vary from year-to-year.

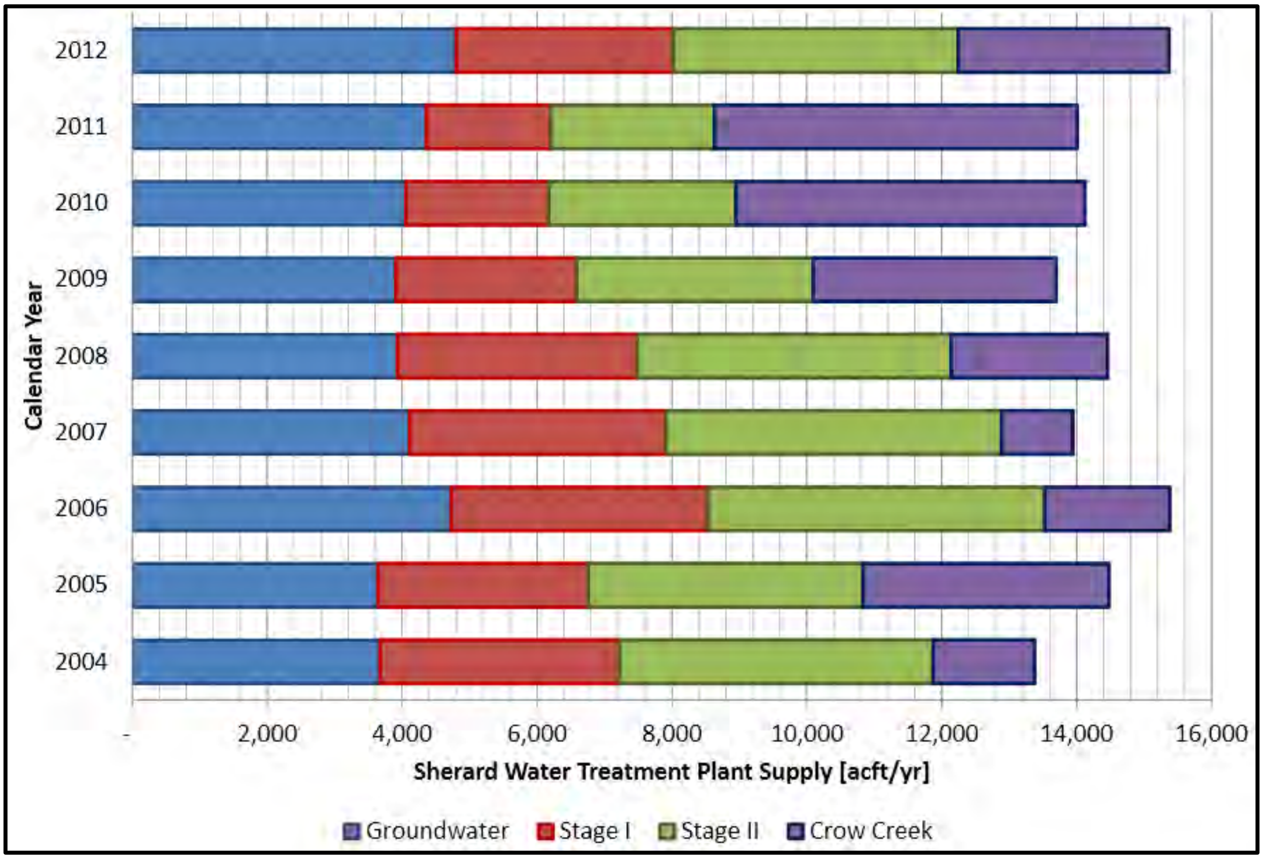


Figure 3-2
Sherard Water Treatment Plant Supply by Source

3.1.2 Evaluation of the Raw Water System

The reliability of the raw water collection system depends on the variability of runoff from the surface water system, the sustainability of the groundwater well fields, and required non-potable and potable demands. Droughts have and will continue to occur in the collection system area. Previous droughts of note include 1950 to 1956 (the "1950s drought"), 1958 to 1967 (the "1960s drought"), and more recently 2000 to 2008 (the "2000s drought"). Runoff in the collection area during these droughts ranged from 13% to 25% below the long term average. Static water levels in some well fields have also been declining. Overall yields over the last ten years have been reduced by approximately 700 gpm.

The variation in surface water supply and trends in groundwater yields affect the raw water supply in meeting demands. The demands themselves, as described in Volume 2, are expected to increase over time. When combined, shortages are likely to occur in the future. Improvements in infrastructure, such as pipelines and expanded storage, may help to provide additional supply along with expansion of existing or new well fields. Infrastructure changes can provide for greater raw water system resiliency and operational flexibility.

In Volume 3, the sustainability of BOPU raw water collection system is evaluated against future potable and non-potable demands in the existing, near-term (2013 to 2023), mid-term (2023 to 2033), and long-term (2033 to 2063) planning periods.

Section 3.2: Existing Raw Surface Water Sources. The extent and magnitude of potential potable supply shortages are identified assuming historic hydrology and watershed yields as well as yields assuming a reduced snowpack. In subsequent sections, potential projects that are intended to reduce potable supply shortages or improve system reliability are examined. These potential projects are described in the following sections of Volume 3

Section 3.3: North Crow Creek Raw Water Collection System Evaluation. The North Crow Creek water collection system represents a significant water resource for the City that is currently under utilized. Options are identified and evaluated for increasing use of the resource.

Section 3.4: Existing Well Field Sustainability Evaluation. The principal water source for the City of Cheyenne has historically been surface water, which has provided about 70% of total demand on average. Groundwater has been used as a supplemental source, for water quality blending, and as an important way to meet peak summer demands. Recent drought conditions have also focused more attention on groundwater usage. As the City faces ever-increasing water supply demands, increasingly limited surface water supplies will require that groundwater become an even more important supply source. This section describes the existing groundwater resources and long term sustainability of groundwater extraction.

Section 3.5: Well Field Development Evaluation. Locations and potential yields of future well fields and expansion of existing well fields are described in this Section.

Section 3.6: Aquifer Recharge Evaluation. Aquifer storage and recovery (ASR) may be a viable way to enhance performance of the Ogallala Aquifer and the corresponding City-owned wells, but only in some areas where the conditions are right. This Section further describes recommendations for implementation of this technology into the City's well fields as appropriate with further testing or study as needed.

Section 3.7: Federal and Bell Well Field Connection Evaluation. Currently groundwater from the Federal Well Field can only be conveyed to the Round Top tank. This Section investigates the feasibility and logistics of a new interconnecting pipeline between the Federal groundwater wells and the King II tank.

Section 3.8: Granite Springs and Crystal Lake Bypass Pipeline Evaluation. Currently the only delivery path for Stage I and Stage II water to the Sherard WTP is through Crystal Lake and Granite Springs Reservoirs. Should either reservoir be rendered unusable either by infrastructure failure or contamination, or taken out of service for maintenance, the impact on BOPU's ability to deliver adequate water to its customers during high demand times would be significant. A set of bypass pipelines is discussed in this Section as an alternate means for delivery of this water to Sherard WTP.

Section 3.9: Crystal Lake Dredging Evaluation. Reclamation of lost Crystal Lake storage volume may be possible though the removal of accumulated sediment. In this Section, alternatives and probable costs are discussed for dredging Crystal Lake Reservoir for the purpose of increasing water storage capacity. The options include in the dredging evaluation include various dredging methods, sediment placement and dewatering methods, and water quality impacts.

Section 3.10: Granite Springs and Crystal Lake Enlargement Evaluation. The combined Granite Springs and Crystal Lake reservoirs have a storage capacity of around 8,700 acre-feet. Expansion of either or both dams could increase BOPU's overall raw water storage capacity. This Section examines the extent of potential dam enlargement and the dam regulatory and safety issues associated with structures the age of the Crystal Lake and Granite Springs reservoir dams.

Recommendations on planning and implementation programs for the existing and near-term, mid-term, and long-term are presented in Section 3.11.

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3.2 Existing Raw Surface Water Sources

The surface water collection system was evaluated using a reconstructed set of natural watershed flows from water years 1933 to 2012. Variations in runoff over this time frame included multiple drought events, most notably the 1950s drought and the 2000s drought. System performance is calculated against the historic flows and existing and future demands. The runoff estimated from the 2003 Master Plan was updated to include the last ten years of BOPU SCADA flow and reservoir data. The same methodology used in the 2003 Master Plan was applied throughout.

Table 3-1 lists the sources of natural runoff data. A historic source of data uses direct measurements of flow or as reconstructed using water budgets. Synthesized runoff estimates were developed from regression equations and drainage-area ratios.

Table 3-2 lists the updated average, minimum, and maximum runoff at various locations. The average watershed runoff has decreased since assessment in the 2003 Master Plan, due to the 2000s drought. However, the year of maximum runoff occurred in water years 2011 and 2012 in the North Platte basin.

Table 3-1
Summary of Flow Data

Location	Period of Record (1933-2012)	
	Historic	Synthesized
Little Snake River Basin		
Stage I/II Diversions to Hog Park Reservoir	1991 to 2012	1932 to 1990
Hog Park Reservoir Natural Inflow (Hog Park Creek)	1988 to 2012	1932 to 1987
Douglas Creek Basin		
Rob Roy Reservoir Inflow (Douglas Creek)	1955 to 1965 1990 to 2012	1933 to 1954 1966 to 1989
Horse Creek Flow	--	1933 to 2012
Other Douglas Creek Diversions	--	1933 to 2012
Lake Owen Inflows	--	1933 to 2012
Crow Creek Basin		
Granite Springs Reservoir Natural Inflow	1991 to 2012	1933 to 1990
Crystal Reservoir Natural Inflow	1991 to 2012	1933 to 1990
Upper North Crow Reservoir Inflow	1970 to 2012	1933 to 1970
Brush Creek Flow	--	1933 to 2012
Upper North Crow Reservoir Inflow	--	1933 to 2012
South Crow Creek Reservoir Inflow	--	1933 to 2012

Table 3-2
Watershed Annual Statistics (1933-2012)

Location	Average [acre-feet/year]	Maximum [ac-ft/yr] (Year)	Minimum [ac-ft/yr] (Year)
Little Snake River Basin			
Stage I/II Diversions to Hog Park Reservoir	18,662	23,250 (1984)	3,180 (1934)
Hog Park Reservoir Natural Inflow (Hog Park Creek)	18,533	34,308 (2011)	7,110 (1977)
Douglas Creek Basin			
Rob Roy Reservoir Inflow	24,287	46,024 (2011)	1,420 (2002)
Horse Creek Flow	2,973	5,634 (2011)	840 (2002)
Other Douglas Creek Diversions	1,540	2,918 (2011)	440 (2002)
Lake Owen Inflows	393	843 (2012)	100 (2002)
Crow Creek Basin			
Granite Springs Reservoir Natural Inflow	3,830	9,510 (1965)	660 (1954)
Crystal Lake Reservoir Natural Inflow	498	1,300 (1965)	90 (1954)
Upper North Crow Reservoir Inflow	1,713	5,240 (1965)	240 (1974)
Brush Creek Flow	468	1,430 (1965)	70 (1974)
Upper North Crow Reservoir Inflow	269	820 (1965)	40 (1974)
South Crow Creek Reservoir Inflow	940	2,860 (1942)	320 (1954)

Appendix 3-A describes the natural runoff methodology and monthly estimates of runoff for each inflow location.

The calculation of system performance was made using the Surface Water Supply System (SWSS) model. The original version of SWSS was developed for the 1993 Master Plan and subsequently updated for the 2003 Master Plan. The SWSS model is a rule-based expert system. The model simulates operation of the reservoirs, Stage I and II pipelines, and collection systems to meet water rights, minimum flows, and deliveries to Sherard WTP and irrigation locations. Several operating changes since the 2003 Master plan were reflected in the model. The Seminoe Account was expanded to a total of 15,700 acre-feet. Groundwater contributions to potable water demands was decreased from 4,000 acre-feet per year to 3,200 acre-feet per year, as described in Section 3.4.7 "Estimates of Sustainable Production from BOUP Well fields". The combined pipeline capacity between Crystal Lake and Sherard WTP is modeled with a 58 mgd capacity.

System demands were simulated for the existing (current conditions), near-term (10 years), mid-term (20 years), and long-term (50 years). Potable demand projections are listed in Table 2-18 and non-potable demands in Table 2-23 of Volume 2. The potable demand ranges reflect various estimates of future population. For this analysis the average population estimate is used for the potable demands. Table 3-3 lists the total, nonpotable, and potable demands that were utilized in this analysis.

Table 3-3
Demand Projections

Year	Planning Period	Projected Total Usage [ac-ft/yr]	Projected Non-Potable Usage [ac-ft/yr]	Projected Potable Usage [ac-ft/yr]
2013	Existing	18,378	780	17,598
2023	Near-Term	21,056	880	20,176
2033	Mid-Term	24,753	990	23,763
2063	Long-Term	34,459	1,280	33,179

The methodology used in Volume 2 reviewed the last ten years (calendar years 2003 to 2012) of Sherard WTP production records. Figure 3-3 provides these annual demands. Demands ranged from 12,492 ac-ft/year in 2003 to 16,638 ac-ft/year in 2012, with a median demand of 14,435 ac-ft/year. During this timeframe, the service population increased from 66,552 in 2003 to 73,836 in 2012, for an average increase of 1.1% per year. The existing planning period assumes a service population between 74,400 as a lower estimate to 75,000 on the upper estimate. To account for changes in service population, the historic water use was converted to per capita rates and multiplied by the existing planning period population. Figure 3-4 shows this adjustment, where the blue bars are the historic water use and the green bars are the population-adjusted water use. When adjusting for the existing planning period population, the range of water demands was from 13,314 acft/year (2009) to 17,930 acft/year (2006), with a median use of 15,019 acft/year.

The adjusted water demands were ranked from highest to lowest, shown in Figure 3-5. The highest water use year is 2006 and the lowest water use year is 2009. An additional 0.5 mgd (approximately 560 acre-feet/year) is added to the adjusted historical demands to reflect potential additional demands from large water users for the Existing Condition Planning Period.

The Existing Condition Planning Period potable usage is a risk-based assessment representing the ranked water use with a 10% chance of usage exceeding the levels shown. This places the potable Existing Condition Planning Period demand between the population-adjusted demands from the highest demand year (2006) and second-highest demand year (2012).

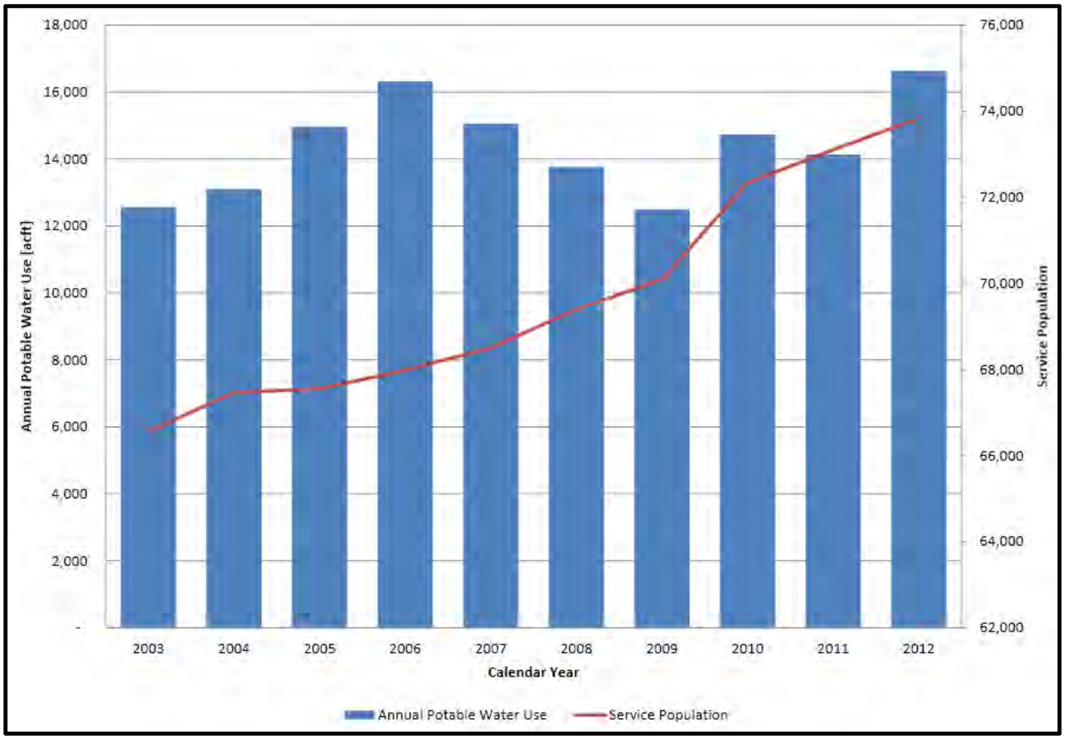

Figure 3-3
Historic Potable Water Use and Service Population

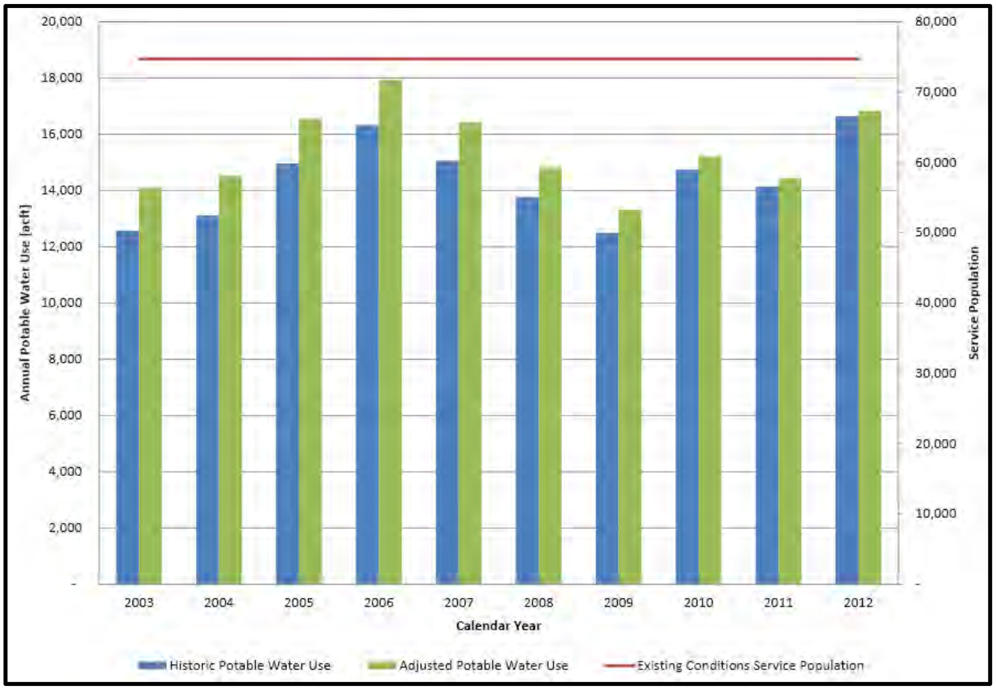

Figure 3-4
Historic Potable Water Use Adjusted for Existing Condition Population

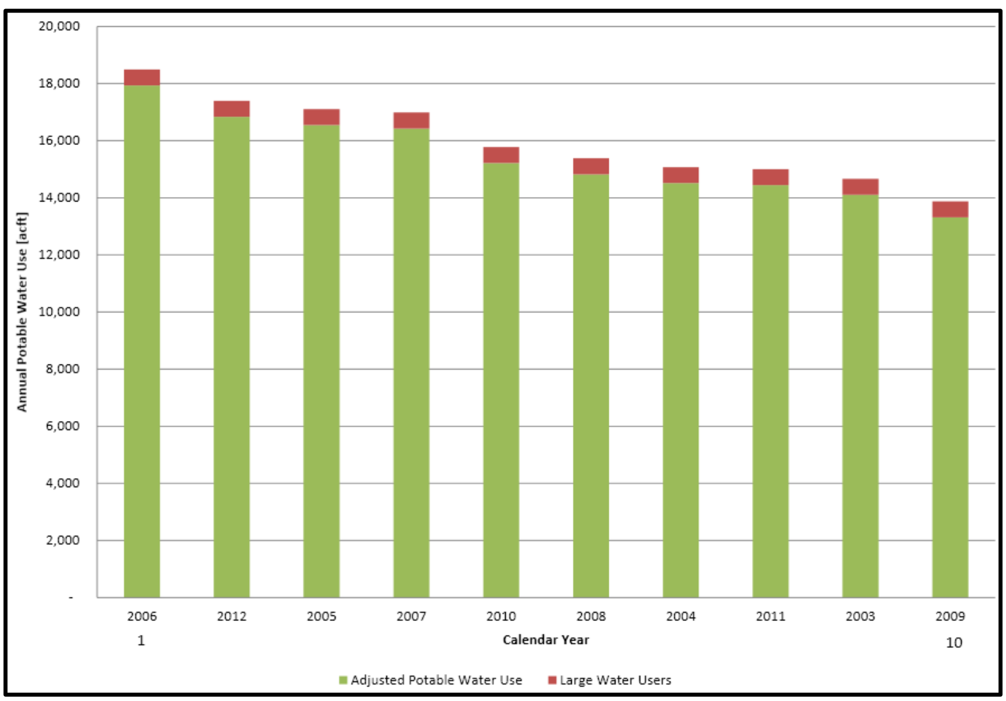

Figure 3-5
Ranked Existing Condition Potable Water Use

System performance is evaluated by determining the potable supply shortages and the risk of shortage. For months with simulated shortages, the average shortage is tabulated along with the worst shortage in a single year. Each month is also assigned a risk of shortage based on BOPU drought level designation. The total system storage for each month is divided by the annual potable demand. The resulting supply index is in years. A supply threshold of 2 or more years is rated Level 1 (No Drought). A supply index of 1.5 to 2 years is a Level 2 (Mild Drought). Level 3 (Moderate Drought) is 1 to 1.5 years. A Level 4 (Severe Drought) is a supply index of less than 1 year. A Level 5 (Emergency) is similar to Level 4, except that the potable supply is experiencing shortages in delivery. Figure 3-6 illustrates the Drought Level calculation.

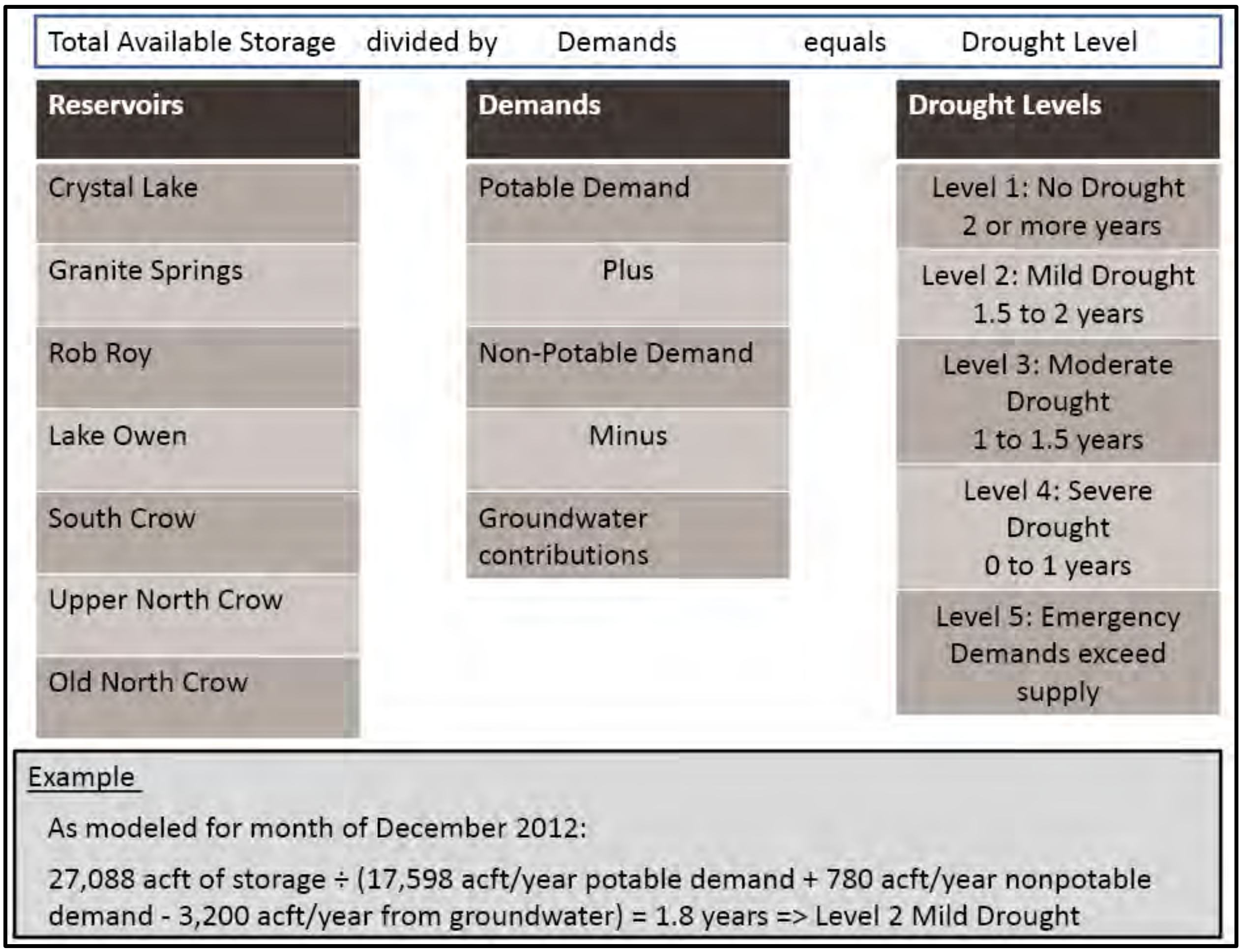

Figure 3-6
Drought Level Calculation

The number of months and frequency in each drought level is tabulated. For 2013 (existing) conditions, there are no simulated shortages of the potable deliveries. Table 3-4 summarizes the drought level risks. Of the 948 months in the simulation (water years 1933 to 2012) the majority (919 months) are at Level 1 (No Drought). The remaining 29 months are at Level 2 (Mild Drought). In the 2023 planning period (Table 3-5), water demands have increased which result in additional stresses to the surface water supply system. The majority of months are still Level 1, although only 613 months are at this level. Level 2 (Mild Drought) occurs more frequently (254 months). The remaining are Level 3 and Level 4.

Table 3-4
2013 Drought Frequency Based on Model Results and Existing Conditions

Drought Level	Number of Months	Frequency (percent)
Level 1: No Drought	919	97%
Level 2: Mild Drought	29	3%
Level 3: Moderate Drought	0	0%
Level 4: Severe Drought	0	0%
Level 5: Emergency	0	0%

Table 3-5
2023 Drought Frequency Based on Model Results and Existing Conditions

Drought Level	Number of Months	Frequency (percent)
Level 1: No Drought	613	65%
Level 2: Mild Drought	254	27%
Level 3: Moderate Drought	76	8%
Level 4: Severe Drought	5	1%
Level 5: Emergency	0	0%

The 2033 planning period (Table 3-6) is the first period where delivery shortages are simulated. A total of 11 months have shortages. The majority of months (245) are Level 4 (Severe Drought, with less than one year of surface water storage).

Table 3-6
2033 Drought Frequency Based on Model Results and Existing Conditions

Drought Level	Number of Months	Frequency (percent)
Level 1: No Drought	53	6%
Level 2: Mild Drought	362	38%
Level 3: Moderate Drought	277	29%
Level 4: Severe Drought	245	26%
Level 5: Emergency	11	1%

In Figure 3-7, the total annual shortages for Level 5 years are shown. Most of these years are during the 2000s drought. In Figure 3-8 the statistical distribution of the annual Level 5 year shortages is shown. The exceedance probability is the percent of Level 5 years where the shortage exceeds a given amount. For a 50% exceedance probability, half of Level 5 years have shortages greater than 693 acre-feet per year. For a 10% exceedance probability, 1 out of 10 of Level 5 years exceed 2,534 acre-feet per year. The single worst year shortage is 3,238 acre-feet per year.

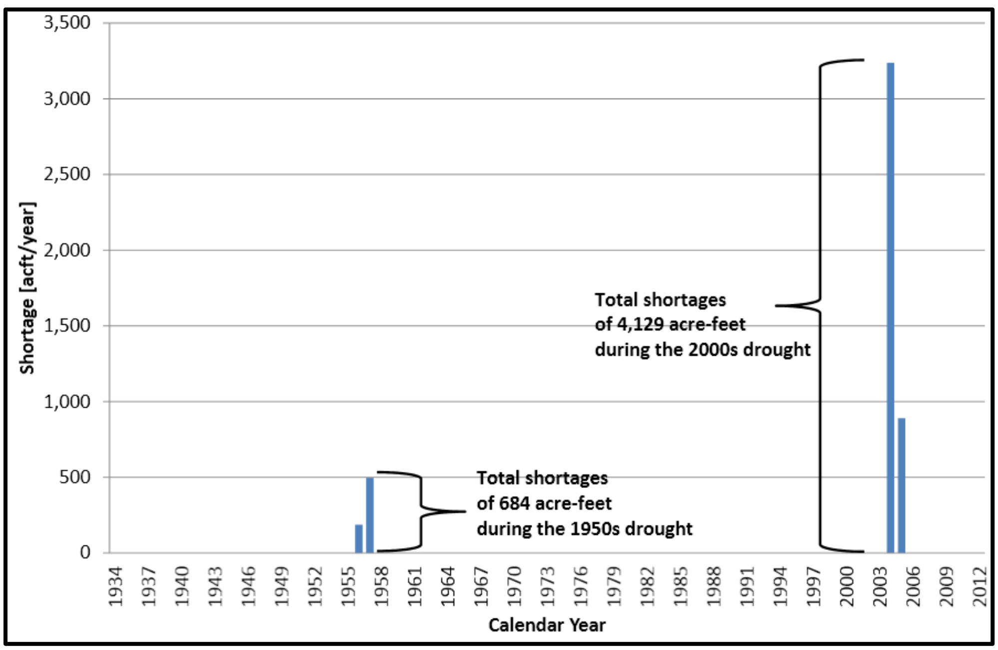

Figure 3-7
Potable Water Demand Shortage, Existing Conditions, Year 2033 Demands

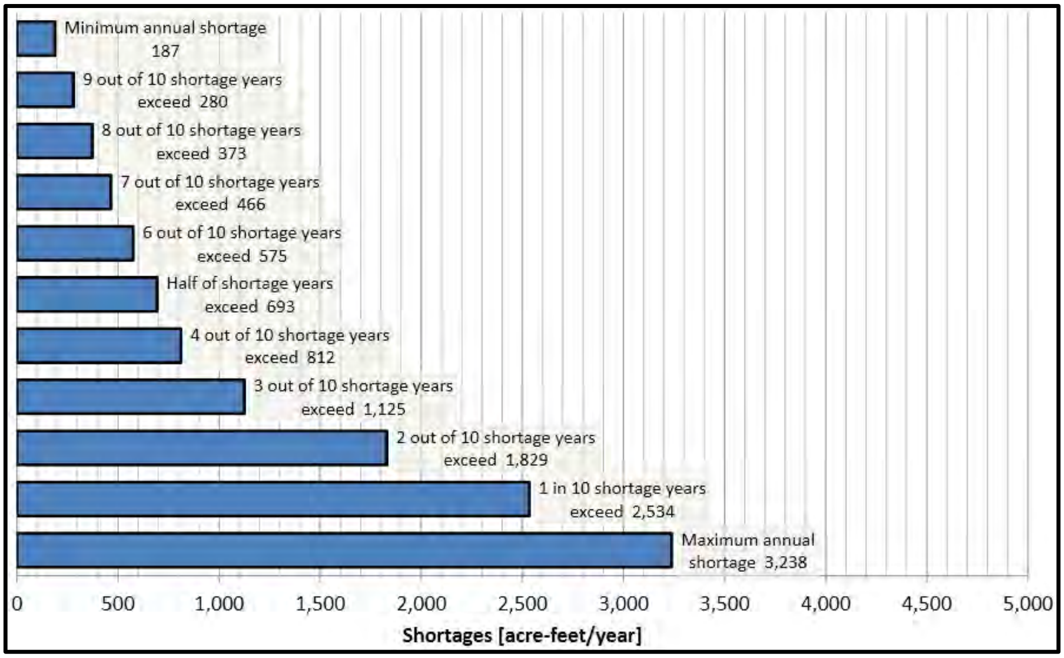

Figure 3-8
Range of Potable Water Demand Shortage, Existing Conditions, Year 2033 Demands

The long-term 2063 planning period has most years with Level 4 and Level 5 drought (Table 3-7). The annual shortage for Level 5 years is shown in Figure 3-9 with a statistical distribution in Figure 3-10. The median annual shortage is 6,201 acre-feet per year. The single worst year shortage is 22,578 acre-feet per year.

Table 3-7
2063 Drought Frequency Based on Model Results and Existing Conditions

Drought Level	Number of Months	Frequency (percent)
Level 1: No Drought	0	0%
Level 2: Mild Drought	0	0%
Level 3: Moderate Drought	29	3%
Level 4: Severe Drought	474	50%
Level 5: Emergency	445	47%

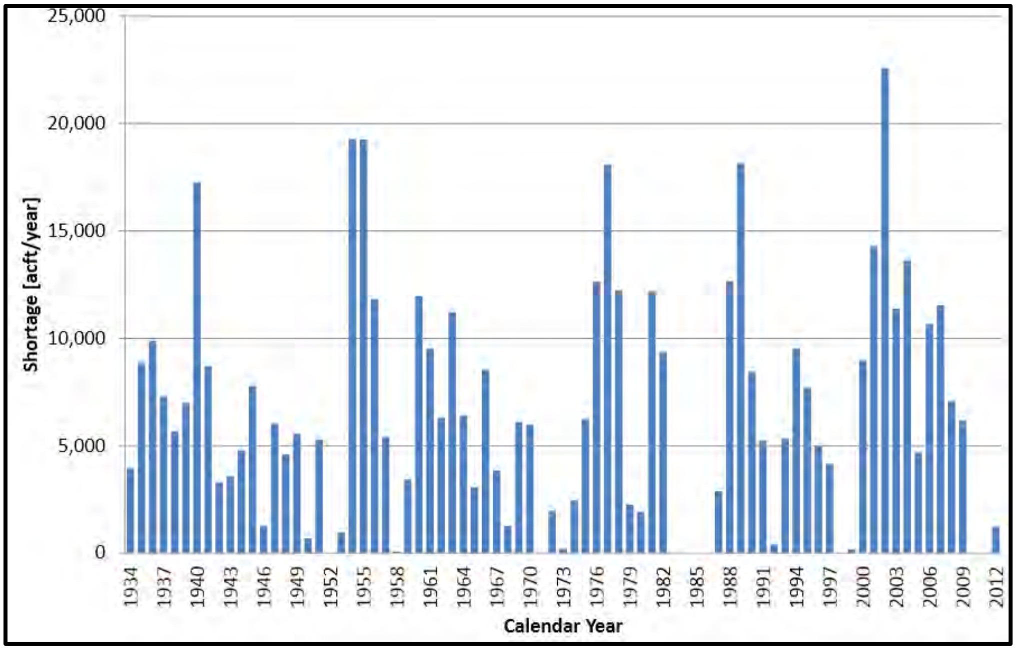

Figure 3-9
Portable Water Demand Shortage, Existing Conditions, Year 2063 Demands

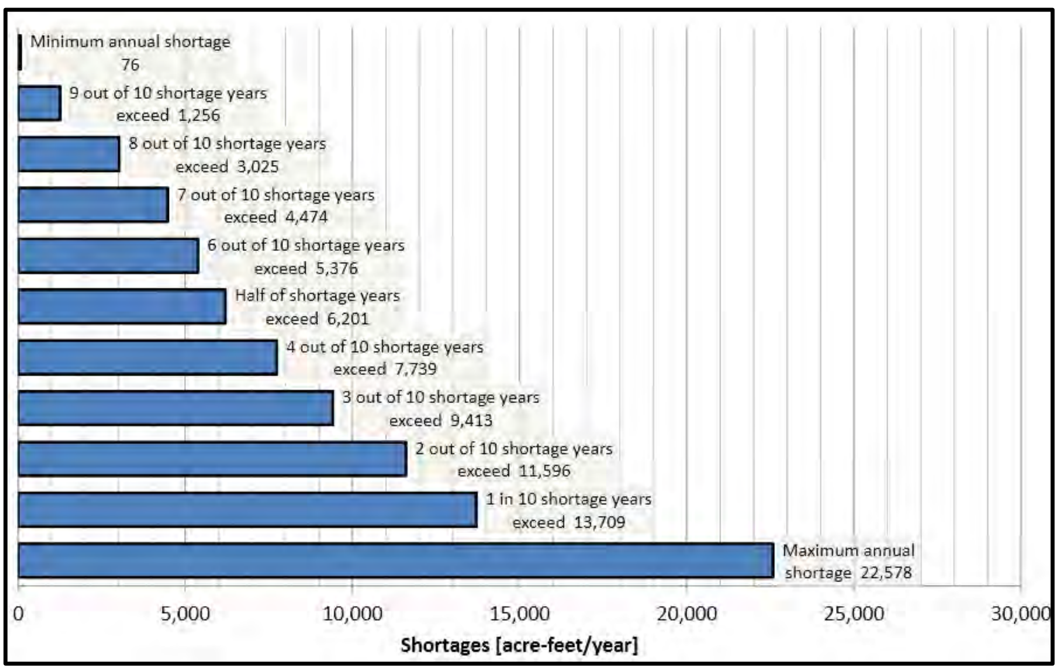

Figure 3-10
Distribution of Potable Water Demand Shortage, Existing Conditions, Year 2063 Demands

The shortage analysis described in Table 3-4 to Table 3-7 compares existing and projected water demands against historic runoff. As runoff is highly dependent on snowpack, a sensitivity analysis was performed where runoff is based on 75% of the historic snowpack. Appendix 3A describes the approach used to calculate the reduced runoff. Table 3-8 to Table 3-11 show the existing system performance for various planning periods using the reduced snowpack. Figure 3-11 to Figure 3-13 show the range of potable shortages for years of Level 5 drought level. The existing planning period is mostly Level 1, although 58 of the simulated months fall into Level 4. By the 2023 planning period, shortages with the reduced snowpack resemble the 2033 planning period with historic runoff. The 2033 planning period with reduced snowpack begins to resemble the 2063 planning period with historic runoff.

Table 3-8
2013 Drought Frequency Based on Model Results, Existing Conditions, and 75% of Historic Snowpack

Drought Level	Historic Hydrology Frequency [%]	Reduced Snowpack Frequency [%]
Level 1: No Drought	97%	66%
Level 2: Mild Drought	3%	19%
Level 3: Moderate Drought	0%	9%
Level 4: Severe Drought	0%	7%
Level 5: Emergency	0%	0%

The inflows of the raw water collection system were reconstructed for the period 1933 to 2012. There were several droughts during this time. The most recent, from 2000 to 2008, is likely the drought of record in terms of overall severity and duration. The raw water collection system has sufficient yield for the existing (2013) and near term (2023) planning periods. In the latter planning period, there are additional stresses (1% of months rated as severe drought) and increased use of imported water.

Starting with the mid term planning period (2033), the existing raw water system does not fully provide for potable demands during certain drought events. Roughly 1% of months have shortages and slightly over a quarter of months are rated as severe drought. By the long term (2063) planning period, potable demand shortages are fairly consistent for each year. Only a handful of months are not rated as severe drought or emergency conditions. The sustainability of the raw water collection is sensitive to climate, in particular the water supply from snowpack. If a long-term reduction of 25% in snowpack occurs, more extreme shortages might occur in earlier planning periods. For example, the shortages simulated in the long term planning period could occur in the mid term planning period with reduced snowpack.

Table 3-9
2023 Drought Frequency Based on Model Results, Existing Conditions, and 75% of Historic Snowpack

Drought Level	Historic Hydrology Frequency [%]	Reduced Snowpack Frequency [%]
Level 1: No Drought	65%	8%
Level 2: Mild Drought	27%	18%
Level 3: Moderate Drought	8%	23%
Level 4: Severe Drought	1%	46%
Level 5: Emergency	0%	4%

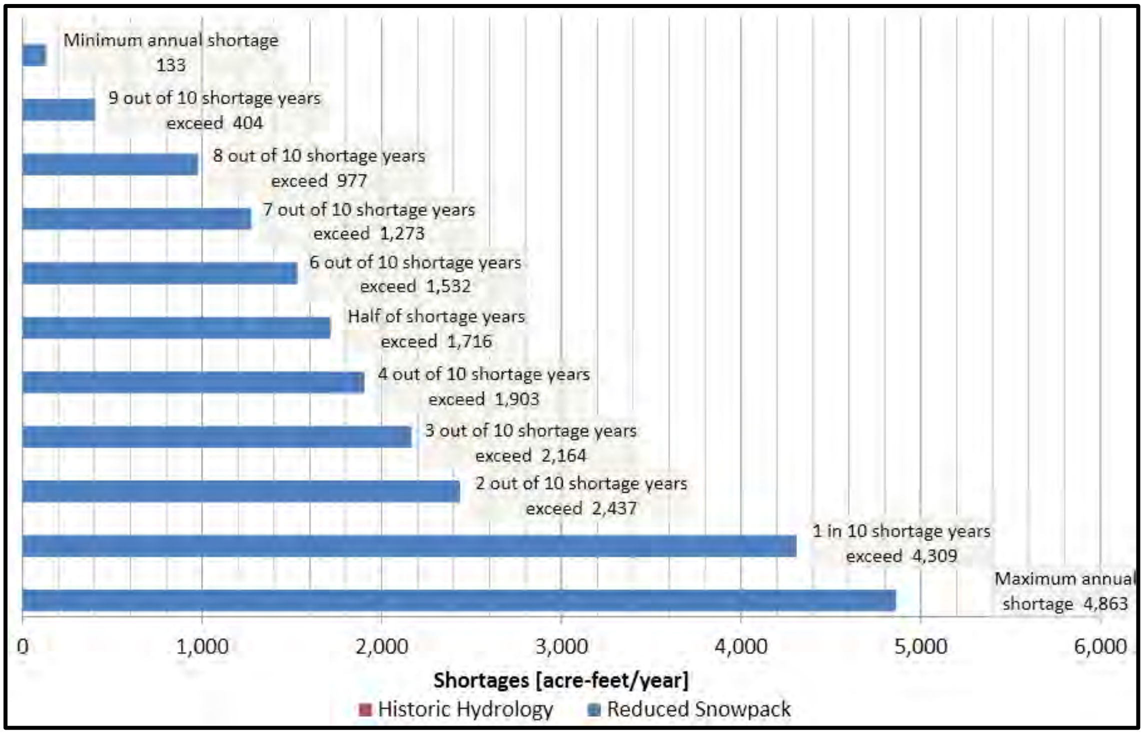

Figure 3-11
Distribution of Potable Water Demand Shortage, Existing Conditions, Reduced Snowpack, Year 2023 Demands

Table 3-10
2033 Drought Frequency Based on Model Results, Existing Conditions, and 75% of Historic Snowpack

Drought Level	Historic Hydrology Frequency [%]	Reduced Snowpack Frequency [%]
Level 1: No Drought	6%	0%
Level 2: Mild Drought	38%	0%
Level 3: Moderate Drought	29%	6%
Level 4: Severe Drought	26%	59%
Level 5: Emergency	1%	35%

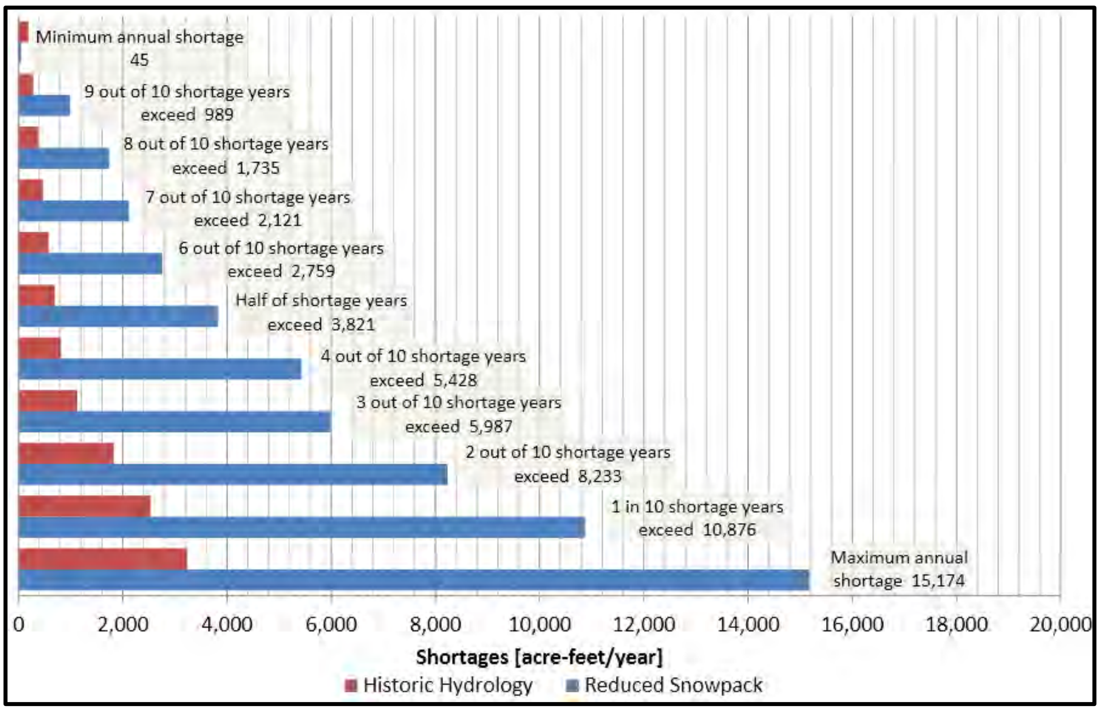

Figure 3-12
Distribution of Potable Water Demand Shortage, Existing Conditions, Reduced Snowpack, Year 2033 Demands

Table 3-11

2063 Drought Frequency Based on Model Results, Existing Conditions, and 75% of Historic Snowpack

Drought Level	Historic Hydrology Frequency [%]	Reduced Snowpack Frequency [%]
Level 1: No Drought	0%	0%
Level 2: Mild Drought	0%	0%
Level 3: Moderate Drought	3%	0%
Level 4: Severe Drought	50%	21%
Level 5: Emergency	47%	79%

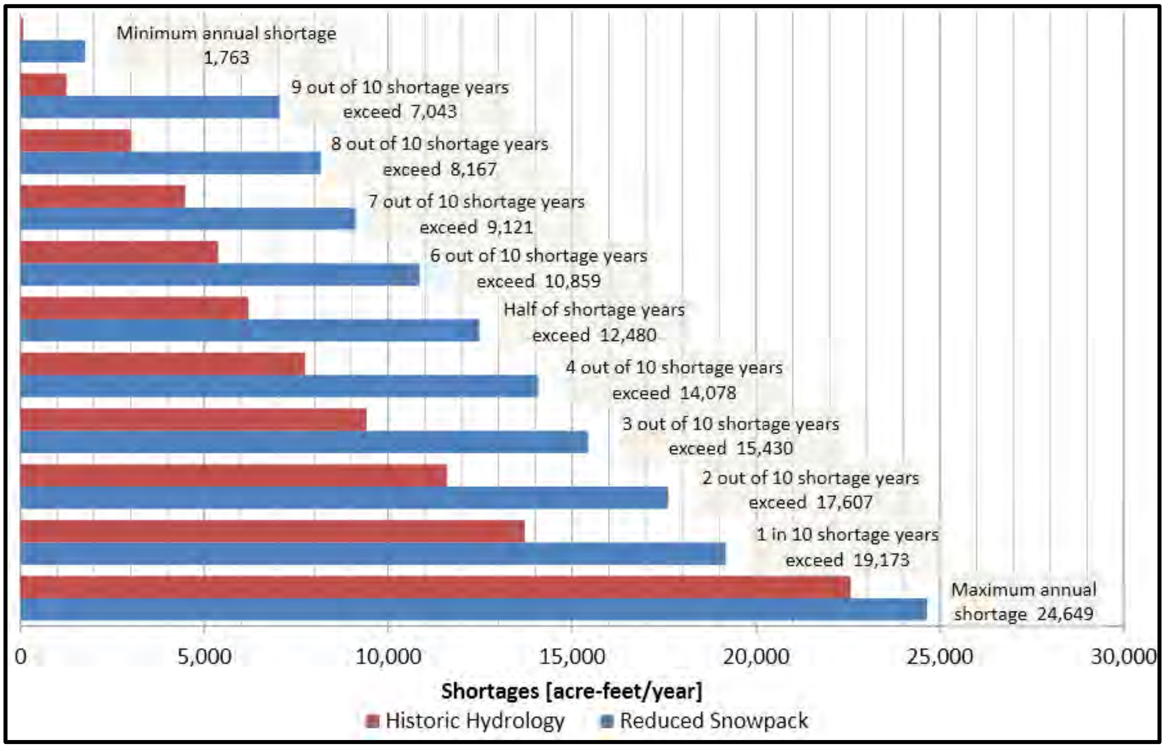

Figure 3-13

Distribution of Potable Water Demand Shortage, Existing Conditions, Reduced Snowpack, Year 2063 Demands

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3.3 North Crow Creek Raw Water Collection System Evaluation

The North Crow Creek Watershed comprises an area of approximately 30 square miles above Upper North Crow Reservoir. Upper North Crow Reservoir has a storage capacity of approximately 1,850 acre-feet. Below Upper North Crow Reservoir, lies an additional 12 square mile area draining into the North Crow Diversion Reservoir, which has a negligible storage capacity on its own. The existing pipeline configurations are shown on Figure 3-14.

Currently, a 20-inch pipeline from the North Crow Diversion Reservoir flows by gravity to the Gilchrist School area (Wye), having a length of 37,329 feet (7.07 miles). At the Wye, the flow from the North Crow Diversion pipeline can go into either or both of two parallel 20-inch pipelines that transmit to Round Top, though the system was designed to use the north line. From Round Top, there is a raw water transmission pipeline crossing the Warren Air Force Base to the Cheyenne Parks Lakes, including Lake Terry, Lake Absarraca, Kiwanis Lake, and Sloan Lake. From there, water is pumped to irrigate the surrounding parks and recreation areas. Existing raw water users in this area include: the Cheyenne Parks system and municipal (Airport) golf course, as well as the Cheyenne Country Club for a reported total of approximately 83 acres of lakes and 333 acres of green space. The raw water irrigation delivery system is shown in Figure 3-15.

The pipeline pressures at the Wye area are too low to compete with the water pressures in the pipelines coming from Crystal Lake Reservoir (the Crystal-Sherard transmission pipelines) and thus the resulting flows can only be transmitted to the Happy Jack Country Church area. In review of the source water records, there has historically been very little use of the North Crow Creek water resource since the retirement of the Round Top Water Treatment Plant in 2002. In a discussion with Mr. Herman Noe, BOPU Engineering and Water Resource Manager, a reasonable target for use of the North Crow Creek water resource could be on the order of two million gallons per day for about 100 days a year (600 ac-ft/year). The historic record for inflows to the Upper North Crow Reservoir indicates that for 85% of the years the total annual reservoir inflows exceed this target diversion amount. Potential users for expansion of raw water coverage include the FE Warren airbase, which operates its own golf course and can already be served from Lake Pearson, and the WYDOT and Game and Fish headquarters, each of which have typical office campus green areas (approximately 13 acres). Another possible user is the Laramie County Public Schools Central High School and McCormick Junior High Campus (13.5 acres), and the Governor's mansion (8.9 acres). Finally, it would be possible to reach the Mylar and Smalley park (8.7 acres green space and 2.8 Lake acres).

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Figure 3-14
Existing and Potential North Crow Pipelines

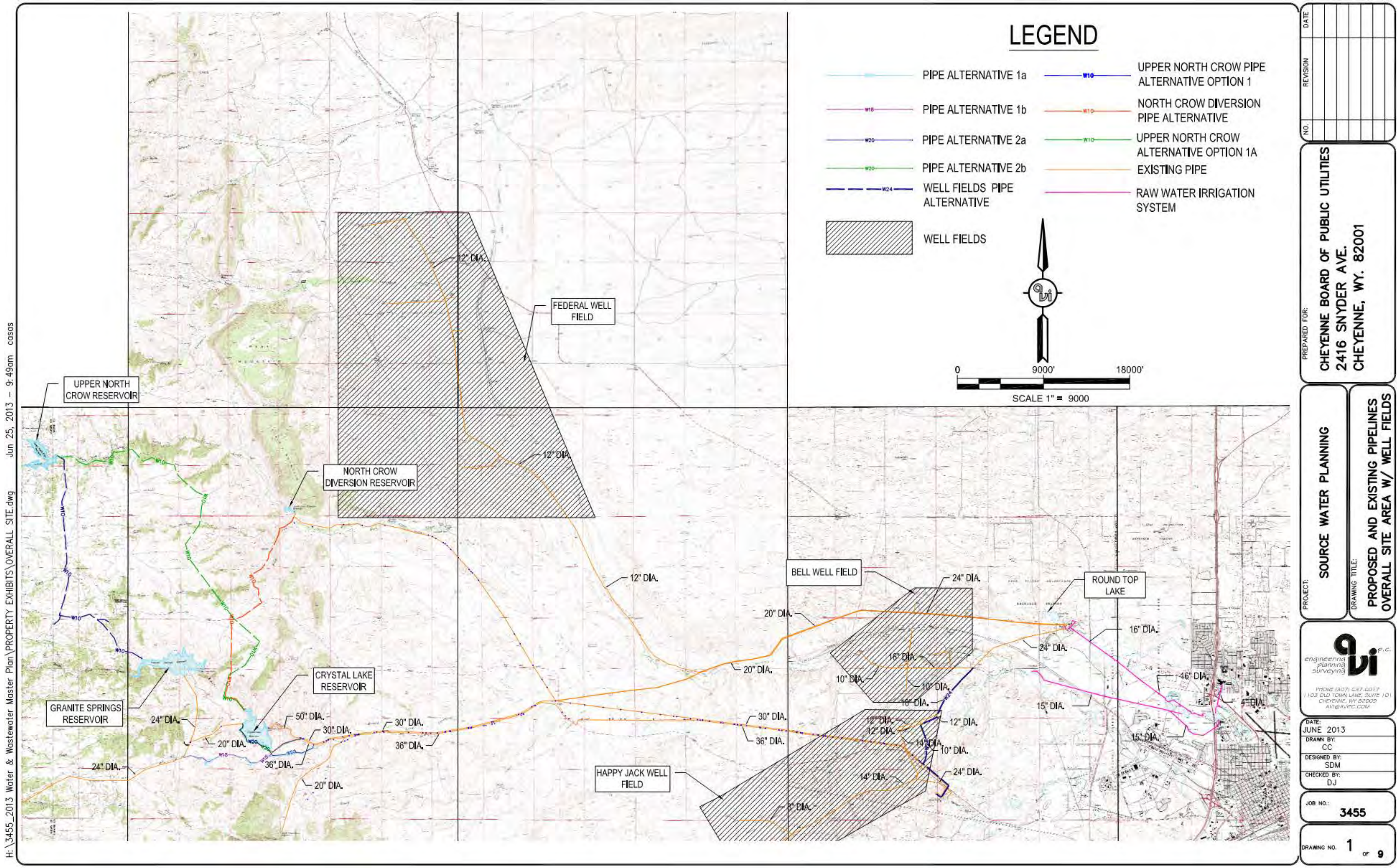
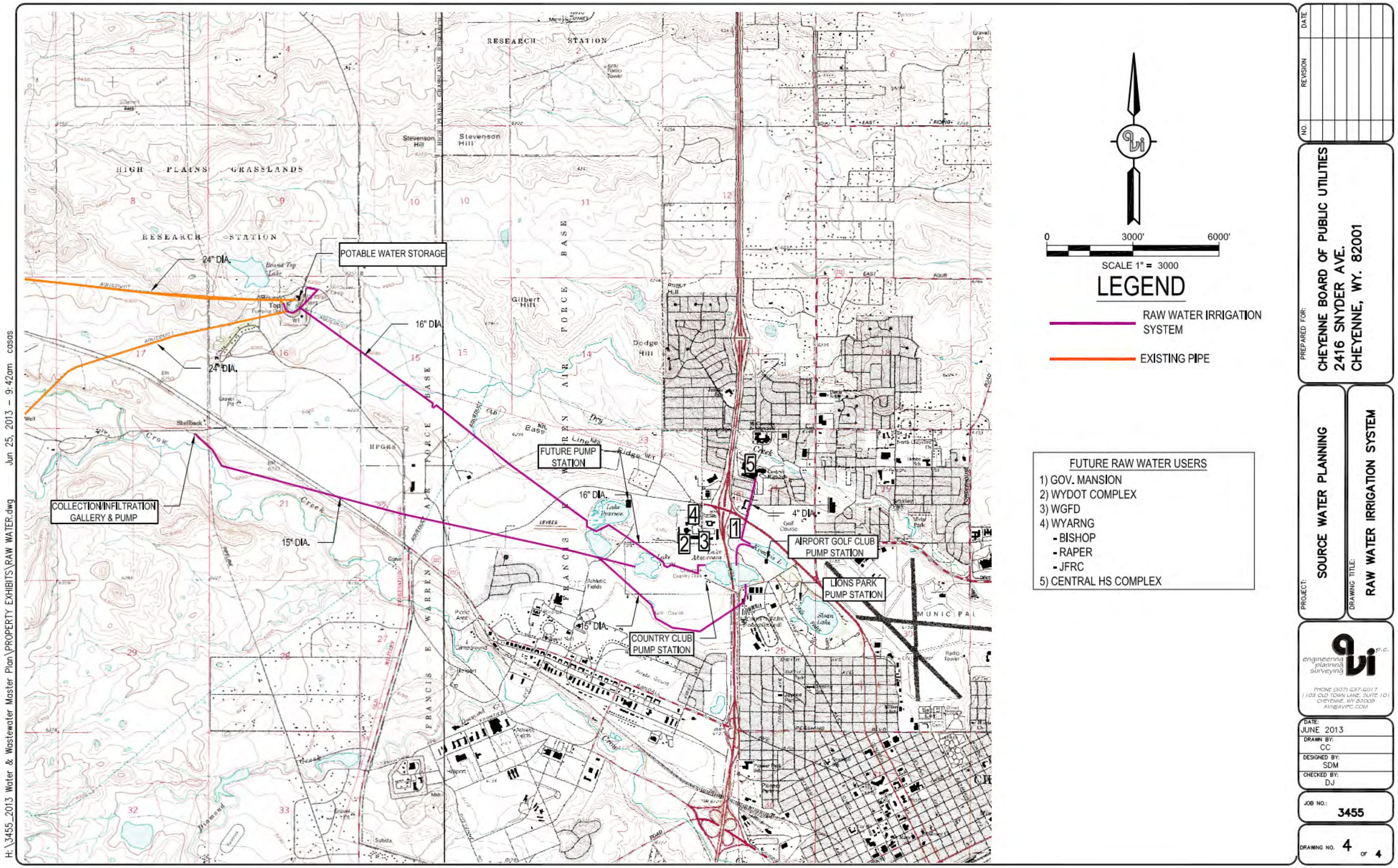


Figure 3-15
Raw Water Irrigation Pipelines



3.3.1 Alternatives for Better Utilization of North Crow Creek Water Resources

Four alternatives were evaluated for enhancing the use of the North Crow Creek surface water resource.

Alternative 1: Pump directly to Crystal Lake Reservoir from North Crow Diversion Reservoir

Crystal Lake Reservoir is about 80 feet of elevation above the outlet at North Crow Diversion Reservoir. A pump station would be required to divert flow into the Crystal Lake Reservoir, where it would be combined with the other source water and pass through the transmission pipelines to the Sherard WTP. The required boost in pressure would be about 33 psi plus the transmission losses from the future 26,184 foot (4.96 mi.) pipeline. A 10-inch pipeline running at 5.7 feet per second (3.1 cfs or 2 mgd) would lose an additional 260 feet of head (114 psi) due to pipe losses. The pipe could operate at approximately 150 psi, within the range of normal PVC (C-905) pipe material tolerances. In addition, the State Engineer's Office (SEO) would likely require modification of the water rights for North Crow Creek, reflecting a change of storage and diversion locations. The profile for this potential alignment is shown on Figure 3-16.

Alternative 2: Convey water through a new transmission pipeline from Upper North Crow Reservoir to Granite Springs Reservoir

A third alternative is to convey water from Upper North Crow Reservoir to Granite Springs Reservoir directly, traversing approximately 28,450 feet (5.39 mi.) utilizing public or quasi-public rights-of-way as well as public and private ranch lands. The most direct alignment would go over two hilltops which would require a pump station to start a siphon flow, as well as installation of air release valves. Since the water elevation at Granite Springs Reservoir is approximately 560 feet below the Upper North Crow reservoir, the flow could function under siphon conditions once established. If an 8-inch transmission pipeline were used instead of 10-inch, flows could still be on the order of 1.6 mgd. A revision of North Crow Creek water rights would have to be filed with the SEO to reflect the relocation of the water storage and point of diversion. The profile for this potential alignment is shown on Figure 3-17. Additional exploration of some natural intermittent flow channels could reduce the required length of the alignment, if discharge to the natural stream channel could accommodate the flow.

Alternative 3: Pump existing pipeline at the Wye

A second alternative for utilizing the North Crow surface water is to boost pressure at the Wye to force water from the existing 20-inch gravity flow pipeline into the 30-inch transmission pipeline from Crystal Lake Reservoir. A capacity analysis indicates this existing 20-inch pipeline should be able to convey around 11 mgd to the Wye by gravity from the North Crow Diversion Reservoir, but taking only 2 mgd results in a residual pressure head of 296 feet (133 psi). The water could be sent directly to the Sherard WTP. Alternatively, flow could be forced back up the 30-inch transmission pipeline toward Crystal Lake Reservoir, but only to the point where the 36-

inch transmission main comes off of the 50-inch combined pipeline approximately 4,790 feet from Crystal Lake Reservoir. The elevation head of Crystal Lake Reservoir with regard to Wye is about 360 feet or 156 psi of static head, so a pump station at Wye would need to add at least 63 feet or 23 psi of head. For this alternative, there would likely not be a change of storage or diversion for the water rights.

Alternative 4: Gravity Conveyance from Upper North Crow Reservoir to Crystal Lake Reservoir

A fourth alternative takes a much longer alignment from Upper North Crow Reservoir, following the contours to the east and avoiding the two hills mentioned above. This alternative would function by gravity and has a theoretical throughput of 2.9 mgd using a 10-inch PVC pipeline. The length of this pipeline concept is 47,508 feet (9.0 mi.) and terminates at Crystal Lake Reservoir rather than Granite Springs Reservoir. If the natural channel upstream of Crystal Lake Reservoir could accommodate the flow, the potential alignment could be shortened somewhat. Further study would be required for this alternative. A revision of North Crow Creek water rights would have to be filed with the SEO to reflect the relocation of the water storage and diversion point. The profile for this potential alignment is shown in Figure 3-18.

Figure 3-16
Potential Alignment North Crow Diversion Reservoir to Crystal Lake Reservoir (Alternative 1)

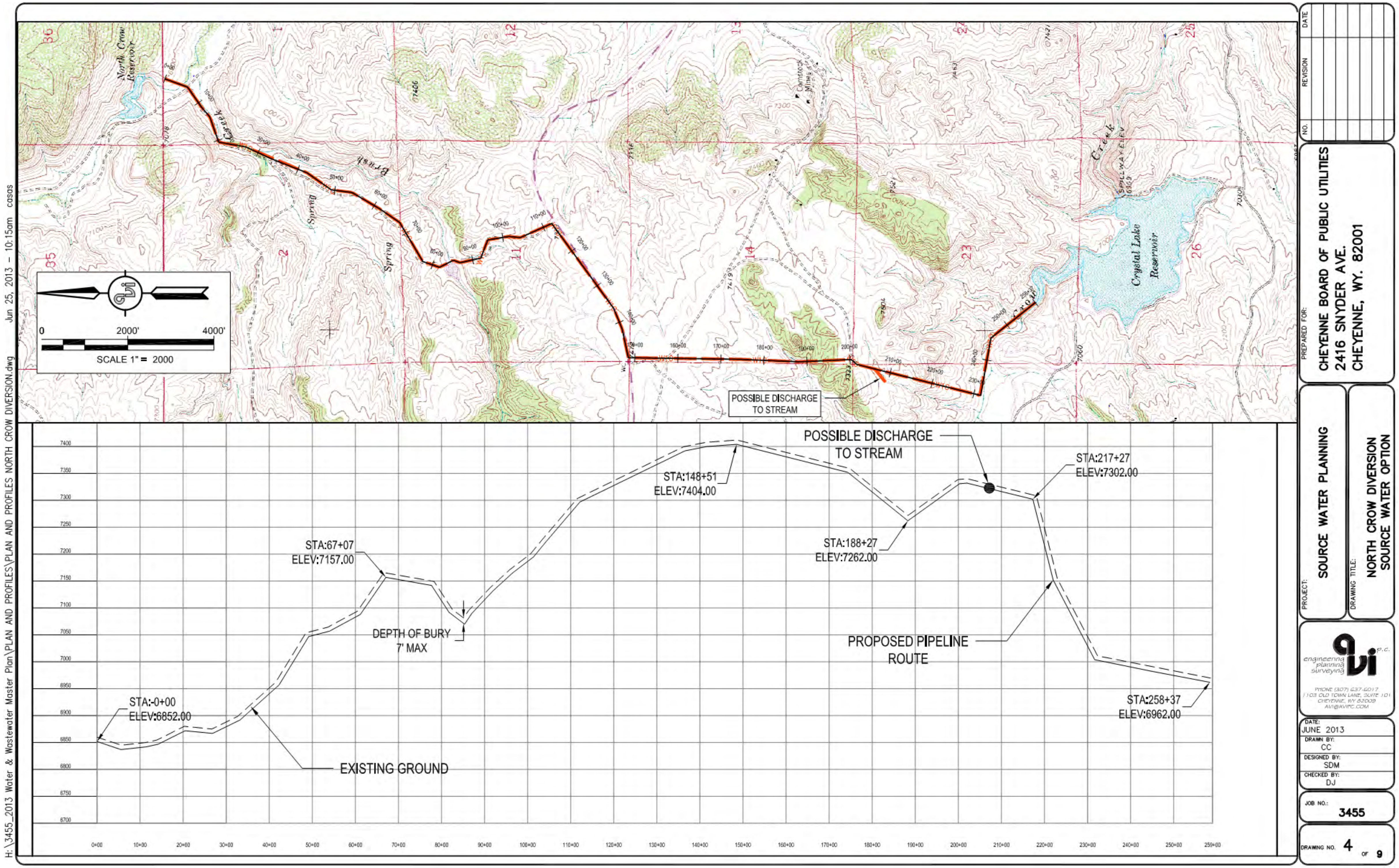


Figure 3-17
Potential Alignment Upper North Crow Reservoir to Granite Springs Reservoir (Alternative 3)

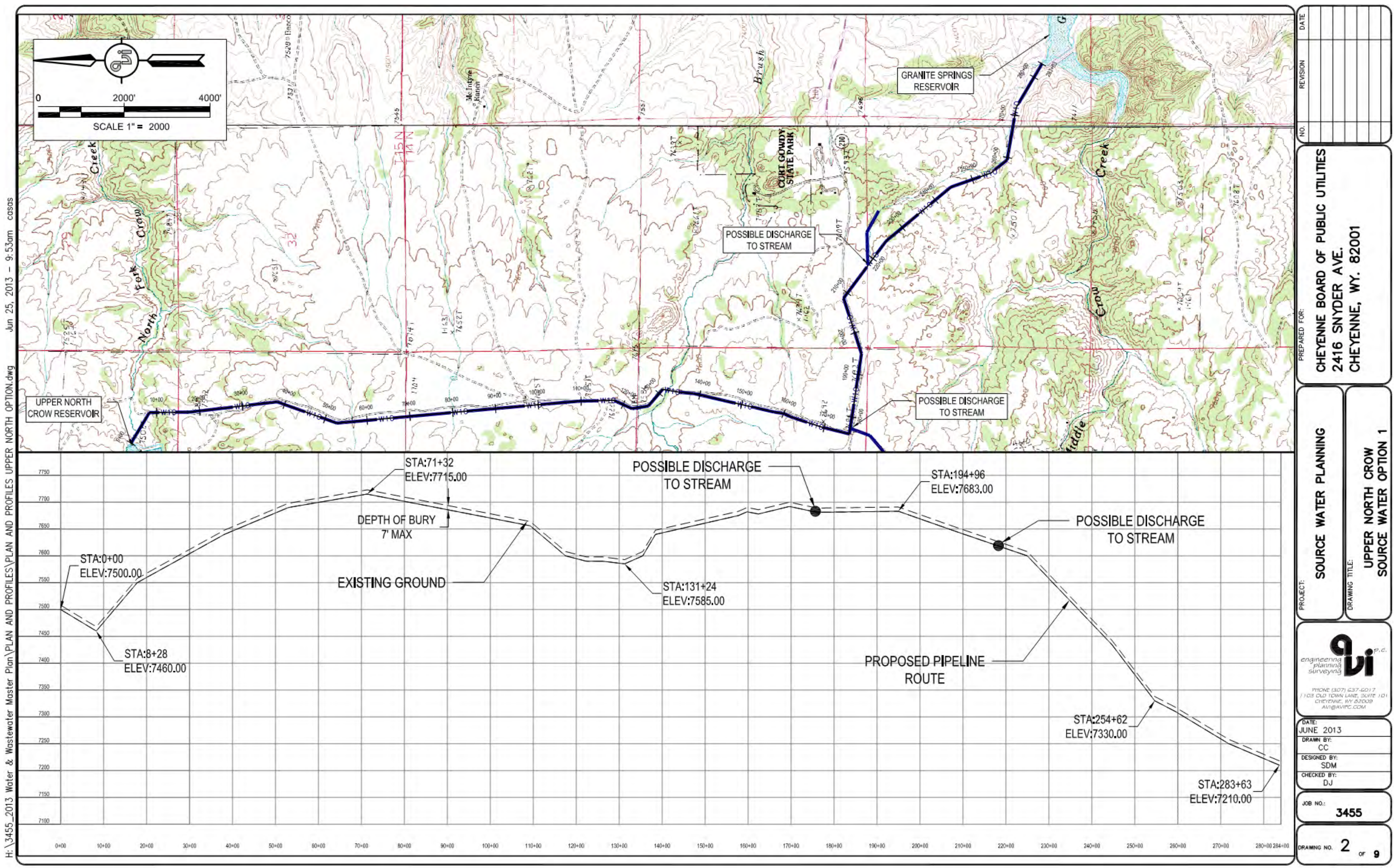
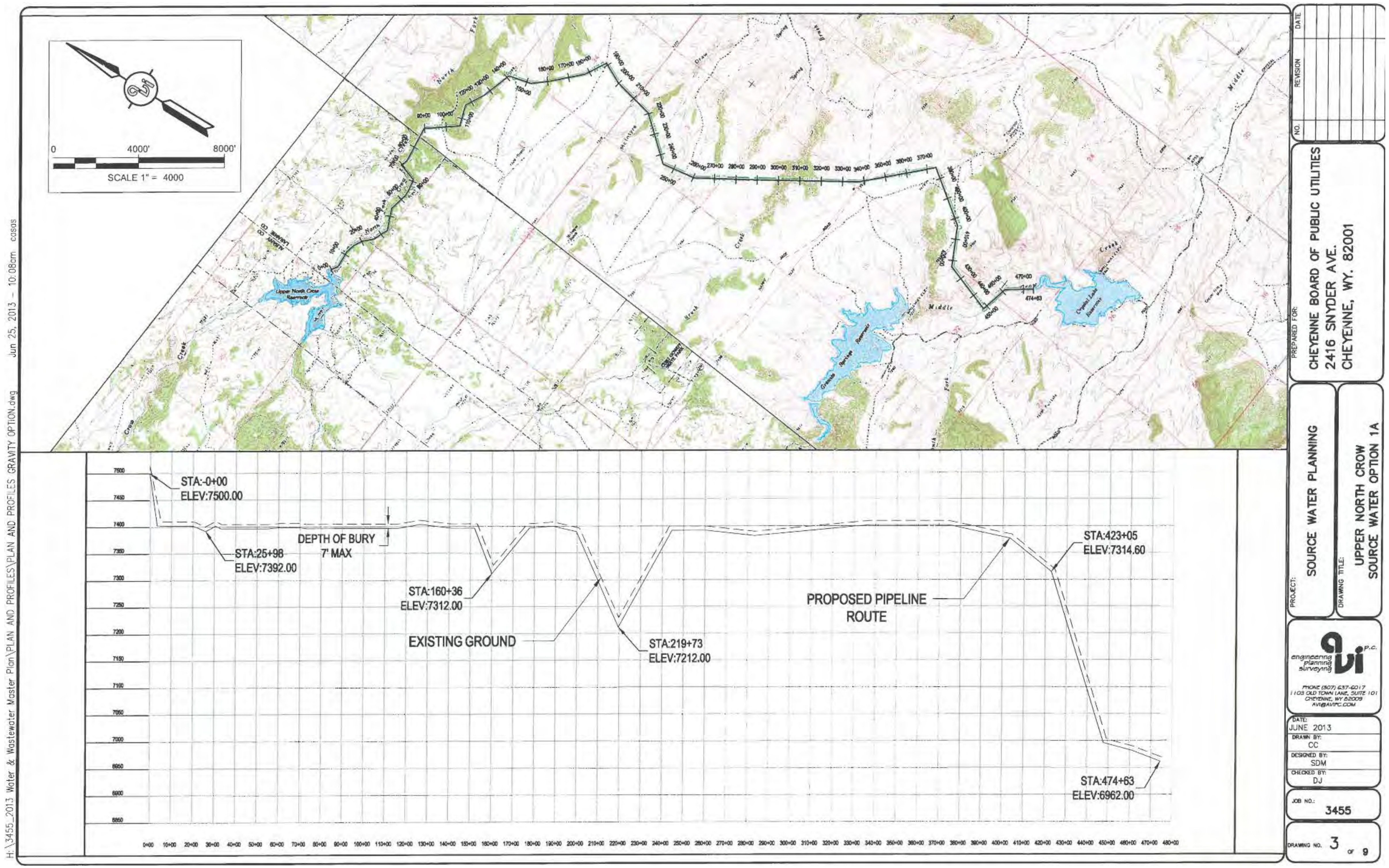


Figure 3-18
Potential Alignment Gravity Conveyance from Upper North Crow Reservoir to Crystal Lake Reservoir (Alternative 4)



3.3.2 North Crow Creek Raw Water Collection System Cost Estimates

The estimated cost of each potential alignment is shown in Table 3-12. These cost estimates are based on Means cost estimating documentation and do not include property and permit acquisition for the selected alignment.

Table 3-12
Estimated Cost for North Crow Raw Water Collection Alternatives

Option	Description	Diameter (inches)	Material	Distance (feet)	Unit Cost ($/ft)	Concept Cost
1	North Crow Div. to Crystal Lake	10	PVC	26184	110	$ 2,880,000
	Pump station required					$ 1,000,000
	Total Concept Cost for Alternative 1					$ 3,880,000
2	Upper North Crow to Granite Springs Reservoir	10	PVC	28450	110	$ 3,130,500
		8	PVC	28450	90	$ 2,560,500
	Pump station required					$ 800,000
	Total Concept Cost for Alternative 2					$ 5,691,000
3	North Crow Div. to Wye (existing)	20		37329		
	Pump station only at WYE w/connections, valves					$ 1,200,000
	Total Concept Cost for Alternative 3					$ 1,200,000
4	Upper North Crow to Crystal (gravity)	10	PVC	47508	110	$ 5,225,900
	Total Concept Cost for Alternative 4					$ 5,225,900

3.3.3 North Crow Creek Raw Water Collection System Impacts on Projected Potable Supply Deficits

The potential North Crow Creek Raw Water Collection System was evaluated for improvements to eliminate the projected potable supply deficits. A uniform flow of 1.45 cfs from January to August (614 acre-feet per year) was used applied to Sherard WTP demands prior to other in-basin and Stage I/II waters. This uniform flow distributes the proposed import water without generating shortages in North and South Crow Creek reservoirs in the existing planning period during the worse drought event. Table 3-13 shows the frequencies of drought levels under this proposed condition for year 2033 projected demands. There is an improvement of 3 months for Level 5 droughts and 11 months for Level 4. The distribution of the Level 5 years is shown in Figure 3-19. The change in drought frequencies and annual shortage distributions for year 2063 is an improvement of 7 months for Level 5. Table 3-14 and Figure 3-20 shows these results.

Table 3-13
Proposed North Crow Creek Pipeline Impacts on Drought Level Frequency, Year 2033 Projected Demands

Drought Level	Existing Condition Frequency [%]	Proposed Condition Frequency [%]
Level 1: No Drought	6%	7%
Level 2: Mild Drought	38%	37%
Level 3: Moderate Drought	29%	31%
Level 4: Severe Drought	26%	25%
Level 5: Emergency	1%	1%

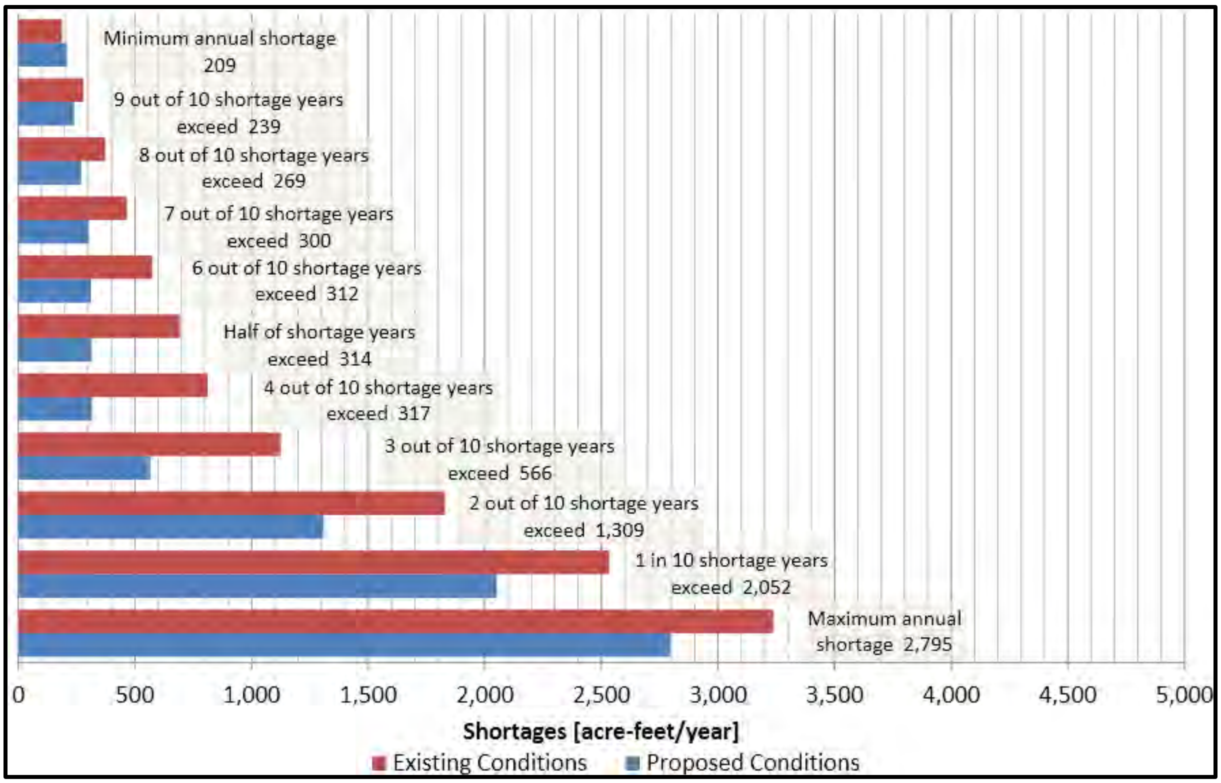


Figure 3-19
Distribution of Annual Potable Shortages using Proposed North Crow Creek Pipeline and Year 2033 Projected Demands

Table 3-14
Proposed North Crow Creek Pipeline Impacts on Drought Level Frequency, Year 2063 Projected Demands

Drought Level	Existing Condition Frequency [%]	Proposed Condition Frequency [%]
Level 1: No Drought	0%	0%
Level 2: Mild Drought	0%	0%
Level 3: Moderate Drought	3%	3%
Level 4: Severe Drought	50%	51%
Level 5: Emergency	47%	46%

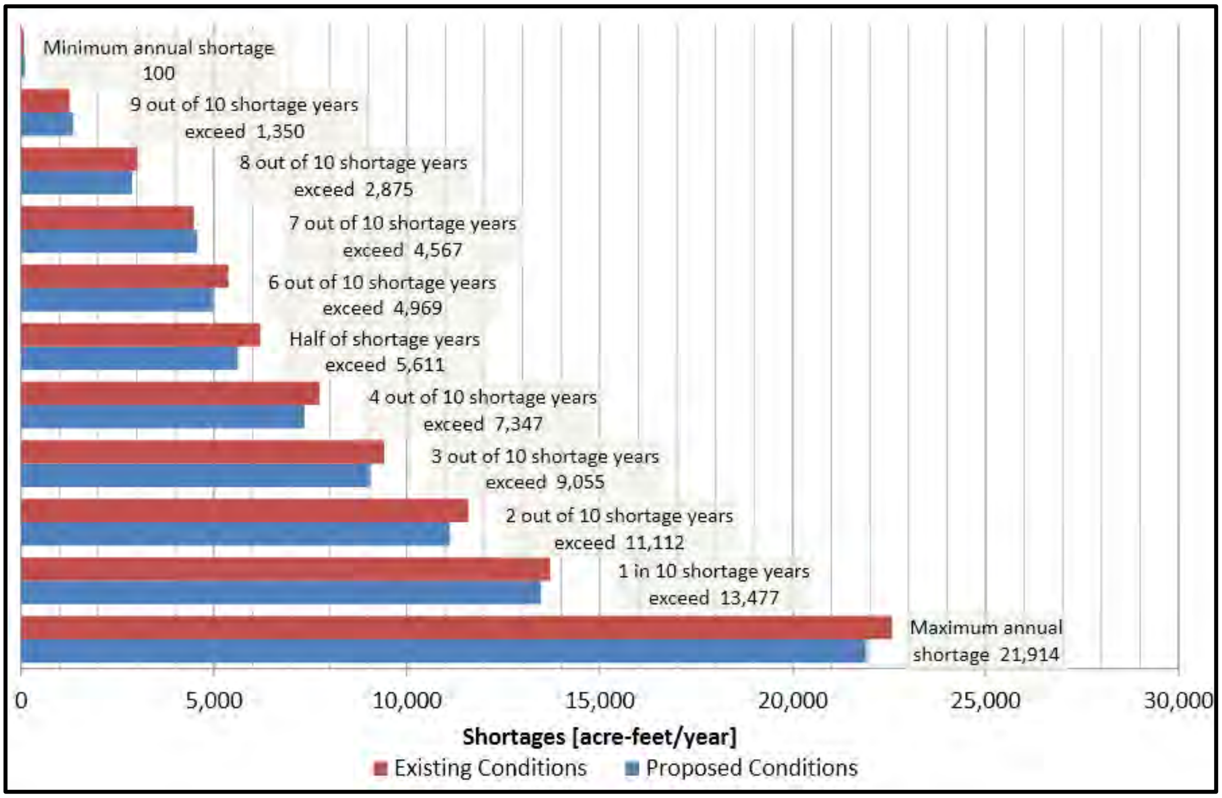


Figure 3-20
Distribution of Annual Potable Shortages using Proposed North Crow Creek Pipeline and Year 2063 Projected Demands

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3.4 Sustainability of Existing Groundwater and Well Fields

3.4.1 Existing Well Field System Description

As shown on Figure 3-21: BOPU Wells and Well Fields, BOPU municipal well fields are located west of Cheyenne and are grouped into four well fields: Bell, Happy Jack, Borie, and Federal. As of 2012, there are 35 active wells and 5 wells that are out of service due to structural problems or low yield. Some wells that were once part of the well field system have been plugged and abandoned as listed in Table 3-15. Structural components of each well, such as pump and motor specifications, are provided in the most recent version of the 2000 Wellhead Protection Plan.

The Bell, Happy Jack, and Borie well fields obtain groundwater primarily from the Ogallala Formation. The wells are distributed across a 5-mile (west-east) by 9-mile (north-south) area approximately 5 miles west of the City. The Bell wells are situated along Crow Creek within the boundaries of the Polo Ranch. The Happy Jack wells are situated along two parallel pipelines oriented northeast-southwest along Spring Creek. The Borie wells are situated south of I-80 and north of Lone Tree Creek and the Belvoir Ranch.

The Federal Well field obtains groundwater primarily from the White River Group and is located 15 miles northwest of the City. The Federal Well field is oriented north-south within the Federal Valley and is in the drainages of South Lodgepole Creek and the North Fork of Crow Creek.

Previous Studies

The groundwater supply for BOPU has been extensively studied since the mid 1930's. Ogle and Jordan (1997) provide a bibliography of documents dated from 1910 to 1996 related to BOPU's municipal well fields. Since 1996, BOPU has obtained additional groundwater resource data from the annual well replacement and rehabilitation program.

Summaries of groundwater resources in the Cheyenne area are provided in Lowry and Crist (1967), the 1994 Water Master Plan, the 2003 Water Master Plan, Weston (1996), and the 2000 Wellhead Protection Plan. Foley (1942) and Morgan (1946) provide a detailed description of the area's hydrogeology and data collection during the early stages of well field development. The Wyoming Water Development Commission (WWDC) has funded several studies of water resources on the recently acquired Belvoir Ranch (2005, 2007), as well as an Aquifer Storage and Recovery (ASR) project for the City's existing well fields (Lytle, 2011).

Much of the information from the 2003 Water Master Plan pertaining to groundwater is still current, and has been carried forward to this 2013 update.

History of Well Field Development

In response to increased demands and supply shortages from Crow Creek during the 1930's drought, BOPU installed four wells between 1934 and 1936. The installation of these wells initiated the development of the Happy Jack Well field in 1937. As the City continued to grow in the early and mid 1940s, the Happy Jack and Borie well fields were expanded, and the first few wells of the Federal Well field were installed. Between 1953 and 1958, the Bell and Federal well fields were developed. The chronology of well installation can be ascertained by the "Year Original Well Drilled" column in the data table of Table 3-15.

The configuration of the four well fields and the development of groundwater supplies have remained mostly unchanged since 1958, in large part due to the expansion of surface water supplies by the Stage I/II projects. A severe drought from 2000 to 2008, along with projected future water demands, has precipitated the need to evaluate and expand groundwater supplies.

Well Replacement and Rehabilitation Program

By the mid-1980s, after 50 years of operation, continued operation of the wells was compromised by the deterioration of the thin-walled steel well casings. Casing failure and frequent pump damage from sand prompted BOPU to initiate a program to replace and/or rehabilitate the original production wells. The program began in 1985 and has resulted in the replacement of 34 wells, with 26 of the wells having been replaced since 1993 as listed in Table 3-15. The well replacement program has generated reports that provide detailed information on lithology, aquifer characteristics, and water quality at each replacement well.

The well replacement program is ongoing and may result, if BOPU chooses, in the replacement of additional wells.

The objectives of the well replacement program are as follows:

- Bring groundwater production back to original production levels (if possible).
- Construct wells to current municipal standards.
- Automate and improve well control/monitoring using SCADA.

The program has improved the reliability of the well infrastructure and the automatic monitoring of water levels and well production. Twenty-three of the original wells were converted into monitoring wells that provide temporal continuity to historic water level data. Pump tests and geophysical logs obtained at each new well have provided additional information on aquifer characteristics. Table 3-16 summarizes selected lithologic and aquifer data obtained during the well replacement program.

The efficiency of the original wells had decreased over time. Initially, an expectation of the well replacement program was that the new wells would match the production of the original wells installed in the 1930s, 1940s, and 1950s. Alternatively, the goal was that the new well would match the adjudicated rate as determined by the SEO in 1992 or match well production prior to well replacement. Table 3-17 lists the documented well production rates at various times compared to the actual production in 2002 and 2012 of wells that have been replaced. Although some replacement wells have maintained noticeably higher yields than the original well (i.e., Borie #1, Elkar #7, Finnerty #2, and others), the 2002 production at replacement wells is typically lower than the original well yield. A comparison of replacement well yield in 1992 to yield in 2002 indicates that total instantaneous yield in 2002 is almost 1,400 gpm less than in 1992. Additional declines of 730 gpm were observed from 2002 to 2012.

As stated by Weston (2000): "well replacement has proven to be an unreliable process… rehabilitating a marginally productive old well may not result in a higher production capacity." A variety of factors, as listed below, may explain the differences in pre- and post-replacement well production:

- Lateral changes in aquifer properties such that the new offset well is located in more or less permeable aquifer material (i.e., Holman #1 and Conrey #1).
- Well design (i.e., well depth, aquifer penetration, screen intervals, pump capacity), filter pack, differences in development techniques and time.
- Over-estimation of the long-term well yield based on short-term pump tests.
- Changes in aquifer production caused by regional static water level declines.

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Figure 3-21: BOPU Wells and Well Fields

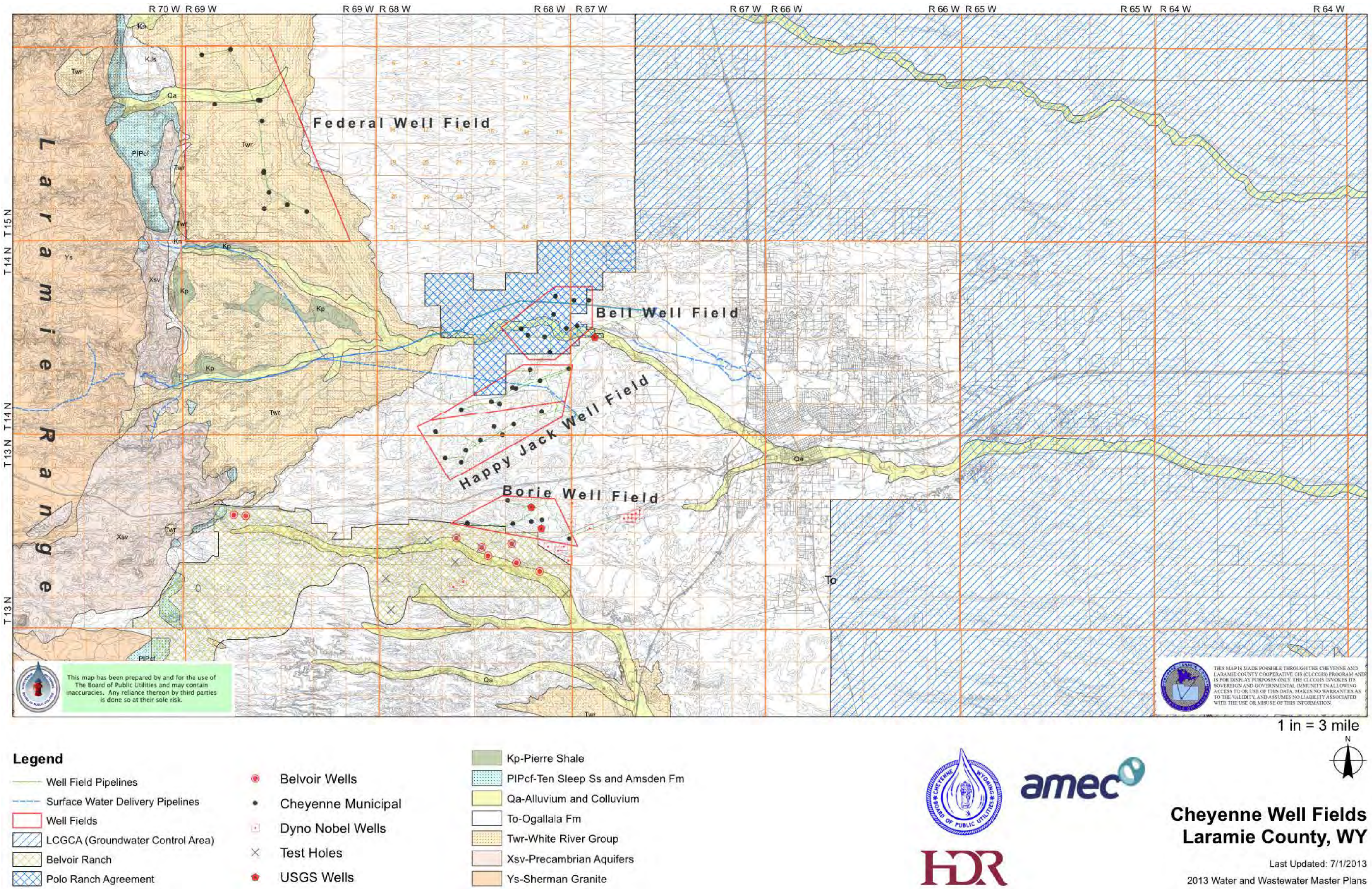

Table 3-15
Chronology of Well Installation, Replacement, and Retirement

Well Name	Location (TRS)	Original Permit	Year of Priority	Adjudicated Amount (gpm)	Year Original Well Drilled	Year Well Replaced	Date Last Rehab	Last Rehab Equipment	2012 Status	Well Total Depth (ft)	Screen Interval (ft)	Pump Set (ft)	SCADA Y/N)	Adjacent Monitoring Well (Y/N)	Water Quality Issues
Borie Well Field															
Borie #1	13, 68,16NE SE	S.C 291	1946	1075	1946	1994	2010	VFD	Active	362	170-342	220	Y	Y	TCE
Elkar #7	13, 68, 14 NW NW	S.C. 285	1945	700	1945	1994	2010	VFD	Active	291	160-273	200	Y	Y	Radon, TCE
Finnerty #2	13, 68	S.C 286	1945	350	1944	N.R.	2010	VFD	Active	210	195	171	Y	Y	TCE
Weber #1	13, 68 14 NW SW	W.R 13	1947	550	1946	1993	2011	VFD	Active	395	163-369	225	Y	Y	TCE
USGS: Borie	13, 68,13 SW SW	?	?	?	1942	n/a	n/a	n/a	Monitoring	?	?	?	?	?	---
Federal Well Field															
Merritt #5	15, 69, 6 SW NE	S.C 278	1942	350	1942	1990	2012	VFD	Active	184	80-184	131	Y	Y	Radon
Merritt #6	15, 69, 5 NW NE	S.C 279	1942	150	1943	2000	2012	VFD	Active	178	93-168	158	Y	Y	---
Tax #1	15, 69, 8 SW SW	S.C 277	1942	0	1942	1956	n/a	VFD	Abandoned	375	?	?	N	N	---
Merritt #1	15, 69, 9, NE SW	S.C 276	1942	200	1942	n/a	n/a	VFD	Out of Service	308	167-308	230	N	N	---
State #2	15, 69, 16 SW NE	W.R 340	1955	425	1954	2000	2012	VFD	Active	354	203-344	265	Y	Y	Radon
Merritt #14	15, 69, 33 NW NE	W.R. 342	1955	250	1954	2001	2010	VFD	Active	245	152-232	195	Y	Y	Radon
State #1	15, 69, 28 NW SE	S.C. 258	1954	225	1954	N.R.	2009	New Well and Equip.	Active	394	75-135	115	Y	Y	---
Merritt #15	15, 69, 33 NW NE	W.R. 257	1954	350	1954	1989	2010	VFD	Active	140	105-212	136	Y	N	Radon
Merritt #8	15, 69, 27 SE SW	W.R. 256	1954	325	1953	1999	2012	VFD	Active	178	111-168	153	Y	Y	Radon
Merritt #9	15, 69, 34 NE NE	W.R. 341	1955	320	1953	1999	2010	VFD	Active	250	59-235	140	Y	Y	Radon
Bell Well Field															
Bell #5	14, 68, 13 NE SE	W.R. 474	1956	300	1956	2000	2010	VFD	Active	293	78-283	150	Y	N	---
Bell #6	14, 67, 18 NW SW	W.R. 475	1956	350	1956	N.R.	2009	New Well and Equip.	Active	225	165-225	190	Y	Y	---
Bell #8	14, 68, 14 NW SE	W.R. 476	1956	400	1956	1991	2008	VFD	Active	170	140-160	138	Y	Y	---
Bell #10	14, 69, 24 SE SW	W.R. 477	1956	150	1956	N.R.	2010	VFD	Active	250	N.A.	220	Y	N	---
Bell #11	14, 68, 13 SE NW	W.R. 478	1956	800	1956	1999	2010	VFD	Active	166	49-156	120	Y	N	Radon
Bell #12	14, 70 14 SW SE	W.R. 479	1956	300	1956	N.R.	2008	VFD	Active	208	N.A.	180	Y	Y	---
Bell/Fed #16	14, 68, 7 SW SW	U.W. 43	1958	500	1958	1982	2010	VFD	Active	270	170-270	240	Y	N	Radon
Bell #17	14, 71, 13 SW SW	W.R. 480	1956	300	1956	N.R.	2012	VFD	Active	225	N.A.	200	Y	Y	---
Bell/Fed #25	14, 68, 12 NW SE	U.W. 45	1958	500	1957	1999	2010	VFD	Active	272	136-264	240	Y	Y	Radon

Well Name	Location (TRS)	Original Permit	Year of Priority	Adjudicated Amount (gpm)	Year Original Well Drilled	Year Well Replaced	Date Last Rehab	Last Rehab Equipment	2012 Status	Well Total Depth (ft)	Screen Interval (ft)	Pump Set (ft)	SCADA Y/N)	Adjacent Monitoring Well (Y/N)	Water Quality Issues
Bell/Fed #24	14, 67, 7 SW SE	U.W. 44	1958	425	1957	n/a	n/a	VFD	Out of Service	--	?	?	N	N	---
USGS: Bell #14	14, 67, 18 SE SE	?	?	300	1956	n/a	n/a	n/a	?	?	?	?	?	?	---
Happy Jack Well Field - North															
Holman #1	14, 68, 24 SE SE	S.C. 265	1934	475	1934	1994	2010	VFD	Active	370	100-335	300	Y	Y	---
Eddy #2	14, 68, 23 SE SE	S.C 269	1937	350	1934	1994	2010	VFD	Active	335	95-321	260	Y	Y	Radon
Elkar #1	14, 68, 25, SW SW	S.C. 266	1936	300	1936	1994	2010	VFD	Active	459	105-437	200	Y	Y	---
Bailey #5	14, 68, 26 SE NE	S.C 268	1940	125	1940	1994	2013	?	Active	317	88-302	200	Y	Y	---
Bailey #1	14, 68, 26 SE NW	S.C. 267	1941	0	1934	n/a	n/a	VFD	Abandoned	215	?	?	?	N	---
Koppes #1	14, 68, 34 NE NE	S.C. 270	1940	500	1940	1994	2010	VFD	Active	304	100-288	220 to 210	Y	Y	---
Koppes #2	14, 68, 27 SW SE	S.C. 271	1940	775	1940	1997	2009	VFD	Active	331	122-322	210	Y	Y	---
Koppes #6	14, 68, 33 NW NE	S.C. 290	1946	200	1945	1989	2010	VFD	Active	270	200-270	238	?	N	Radon
Happy Jack Well Field - South															
Happy Jack #1	14, 68, 36 SE NE	S.C. 270	1941	0	1941	n/a	n/a	n/a	Abandoned	152	--	?	?	N	---
Happy Jack #2	14, 68, 36 SW NE	S.C. 274	1941	0	1941	n/a	n/a	n/a	Abandoned	184	--	?	?	N	---
Happy Jack #3	14, 68, 36 SW NW	S.C. 275	1941	250	1941	1985	2010	VFD	Active	285	145-285	252	Y	N	Radon
King #4	14, 68, 35 NE SW	S.C.287	1945	500	1945	1994	2010	VFD	Active	350	170-330	240	Y	Y	Radon
Koppes #4	14, 68, 34 SE SE	S.C. 281	1944	250	1944	199	2010	VFD	Active	362	177-253	250	Y	Y	Radon
Koppes #3	14, 68, 34 SE SE	U.W.108831	1943	425	1943	1998	2010	VFD	Active	375	130-365	211	Y	Y	---
King #2	13, 68, 3 NW NW	S.C. 284	1947	0	1945	1990	1992	?	Out of Service	150	120-150	131	N	N	---
Conrey #1	14, 68, 32 NW SE	S.C. 288	1947	225	1947	1994	1998	VFD	Out of Service	423	183-273	?	N	Y	---
Koppes #5	13, 68, 33 SW SE	W.R. 14	1945	0	1945	n/a	n/a	n/a	Abandoned	n/a	?	?	?	?	---
Elkar #5	13, 68, 4 SW NE	S.C. 282	1944	750	1944	1986	2010	VFD	Active	410	180-360	300 (?)	Y	Y	---
King #1	13,68, 4 SW SE	U.W. 95498	1994	275	1944	1994	2012	VFD	Active	360	185-353	270	Y	N	Radon
King #5	13, 68, 4 NE NE	S.C.289	1945	275	1945	1997	2012	VFD	Active	395	230-385	295	Y	N	---
USGS: King #3	14, 68, 35 SE SW	?	?	?	1945	n/a	n/a	n/a	Monitoring	230	?	?	?	?	---

N.R = Not Replaced.

Table 3-16
Lithologic and Aquifer Data

Well Name	Ground Elev. (ft)	Well Total Depth (ft)	Unit of Well Completion	Thickness of To and/or Twr w/ Contact Elev. (ft amsl)	Earliest DTW (ft)	Saturated Thickness of To (ft)	2012 Yield (gpm)	Transmisivity (gpd/ft)	Storage Coefficient (unitless)
Borie Well Field									
Borie #1	6,640	362	To	To: 345/6295	98 (1947)	206	515	43,000	8 x 10-2
Elkar #7	6,601	291	To	To: 291 (TD)	85 (1947)	172	420	37,000	?
Finnerty #2	6,665	210	To	To: 280/6385	45 (1947)	?	425	23,400	?
Weber #1	6,415	395	To, Twr	To: 262/6153	43+ (1947)	244	190	5,800	1.2 x 10-4
USGS: Borie	6,528	?	To	?	37 (1945)	?	--	?	?
Federal Well Field									
Merritt #5	6,841	184	Twr	Twr: 184 (TD)	30 (1942)	?	150	13,000	3.2 x 10-3
Merritt #6	6,680	178	Twr	Twr: 168/6512	86 (1944)	?	135	4,200	5.0 x 10-3
Tax #1	6,833	375	?	?	147 (1944)	?	?	?	?
Merritt #1	6,857	308	Twr	Twr: 320/6537	73 (1942)	?	--	7,500	?
State #2	6,780	354	Twr	Twr: 346/6434	23 (1954)	?	345	3,220	1.7 x 10-3
Merritt #14	6,665	245	Twr	Twr: 245 (TD)	Flowing	?	50	2,600	?
State #1	6,642	394	Twr	Twr: 148/6494	16 (1954)	?	65	N.A.	?
Merritt #15	6,680	140	Twr	Twr: 185(?)/6495	30 (1954)	?	300	8,600	3.5 x 10-4
Merritt #8	6,620	178	Twr	Twr: 175/6450	Flowing	?	340	25,000	1.5 x 10-4
Merritt #9	6,595	250	Twr, Kl, Kfh	Twr: 130/6465	16 (1954)	?	190	7,800	4.0 x 10-2
Bell Well Field									
Bell #5	6,263	293	Twr (?)	Twr: 283 (TD)	0	?	167	8,800	1.2 x 10-4
Bell #6	6,270	225	To, Twr	?	Flowing	?	240	13,600	5.6 x 10-5
Bell #8	6,348	170	To	To: 163/6185	35	93	185	29,000	4.4 x 10-5
Bell #10	6,328	250	To, Twr	?	40	?	276	6,200	--
Bell #11	6,280	166	To	To: 154/6126	Flowing	133	470	36,300	4.5 x 10-5
Bell #12	6,344	208	To, Twr	?	82	?	241	19,200	5.9 x 10-5
Bell/Fed #16	6,328	270	To, Twr	?	26	?	320	8,900	?
Bell #17	6,330	225	To, Twr	?	42	?	190	27,000	?
Bell/Fed #25	6,380	272	To, Twr	To: 260/6120	88	156	?	13,900	1.1 x 10-4
Bell/Fed #24	6,333	--	To, Twr	To: 290/6043	53	?	190	?	?
USGS: Bell #14	6,248	?	To	?	13 (1957)	?	--	?	?

Well Name	Ground Elev. (ft)	Well Total Depth (ft)	Unit of Well Completion	Thickness of To and/or Twr w/ Contact Elev. (ft amsl)	Earliest DTW (ft)	Saturated Thickness of To (ft)	2012 Yield (gpm)	Transmisivity (gpd/ft)	Storage Coefficient (unitless)
Happy Jack Well Field - North									
Holman #1	6,322	370	To, Twr	To: 280/6042	17 (1942)	252	200	17,500	1 x 10-3
Eddy #2	6,390	335	To, Twr	To: 235/6155	27 (1941)	162	240	6,000	5 x 10-4
Elkar #1	6,410	459	To, Twr	To: 235/6175	29 (1947)	184	145	8,800	?
Bailey #5	6,400	317	To, Twr	To: 215/ 6185	12 (1940)	137	--	16,000	?
Bailey #1	6,393	215	?	?	6 (1947)	?	?	?	?
Koppes #1	6,465	304	To, Twr	To: 260/6205	19 (1940)	154	405	29,300	?
Koppes #2	6,554	331	To, Twr	To: 245/6309	30 (1940)	123	380	20,000	1.1 x 10-5
Koppes #6	6,585	270	To	To: 265/6320	123 (1947)	56	80	4,100	9.6 x 10-3
Happy Jack Well Field - South									
Happy Jack #1	6,365	152	To	?	21 (1941)	?	?	?	?
Happy Jack #2	6,409	184	To	?	22 (1941)	?	?	?	?
Happy Jack #3	6,428	285	To, Twr	To: 220/6208	14 (1941)	166	200	40,000	?
King #4	6,513	350	To, Twr	To: 305/6207	77 (1947)	183	200	16,500	2 x 10-4
Koppes #4	6,550	362	To, Twr	To: 228/6332	85 (1947)	84	315	2,480	3.1 x 10-4
Koppes #3	6,527	375	To, Twr	To: 184/6343	70 (1947)	56	360	62,700	4.5 x 10-4
King #2	6,554	150	To	To: 332(?)/6222	76 (1947)	?	--	36,500	4.4 x 10-4
Conrey #1	6,648	423	To, Twr	To: 225/6423	145 (1947)	11	--	250	?
Koppes #5	6,635	?	?	?	140 (1947)	?	?	34,316	?
Elkar #5	6,585	410	To, Twr	?	101 (1947)	?	400	9,100	?
King #1	6,627	360	To, Twr	To: 290/6337	118 (1947)	100	151	13,200	?
King #5	6,684	395	To, Twr	To: 285/6399	173 (1947)	43	60	9,100	5.4 x 10-4
USGS: King #3	6,520	230	To	To: 230 (TD)	67 (1946)	?	?	?	?

N.A. = Not Applicable

Table 3-17
Well Production Rates

Well Name	Adjudicated Amount (gpm)	Original Well Yield[1] (gpm)	Measured Yield in 1992[1] (gpm)	2002 Yield[2] (gpd)	2012 Yield (gpm)	1992 to 2012 Change in Yield (gpm)	1992 to 2002 Change in Yield (gpm)	2003 to 2012 Change in Yield (gpm)
Borie Well Field								
Borie #1	1,075	410	328	500	515	187	172	15
Elkar #7	700	400	334	550	420	86	216	-130
Finnerty #2	350	495	288	230	425	137	-58	195
Weber #1	550	--	392	200	190	-202	-192	-10
USGS: Borie	n/a	--	--	--	--	n/a	n/a	--
Federal Well Field								
Merritt #5	350	--	350	250	150	-200	-100	-100
Merritt #6	150	150	233	120	135	-98	-113	15
Tax #1	-	--	n/a	n/a	n/a	N.D.	N.D.	n/a
Merritt #1	200	--	N.A	--	--	-38	-103	--
State #2	425	425	383	280	345	N.D.	N.D.	65
Merritt #14	250	--	N.A	150	50	-46	N.D.	-100
State #1	225	--	111	--	65	-54	-104	--
Merritt #15	350	--	354	250	300	-106	-166	50
Merritt #8	325	325	446	280	340	-118	-58	60
Merritt #9	320	320	308	250	190	-200	-100	-60
Bell Well Field								
Bell #5	300	--	111	170	167	56	59	-3
Bell #6	350	--	350	285	240	-110	-65	-45
Bell #8	400	445	N.A	250	185	N.D.	N.D.	-65
Bell #10	150	--	142	--	276	134	N.D.	--
Bell #11	800	800	685	550	470	-215	-135	-80
Bell #12	300	--	338	325	241	-97	-13	-84
Bell/Fed #16	500	--	438	300	320	-118	-138	20
Bell #17	300	--	278	285	190	-88	7	-95
Bell/Fed #25	500	?	?	?	?	-148	-58	?
Bell/Fed #24	425	425	338	280	190	N.D.	N.D.	-90
USGS: Bell #14	300	--	223	--	--	56	59	--

Well Name	Adjudicated Amount (gpm)	Original Well Yield[1] (gpm)	Measured Yield in 1992[1] (gpm)	2002 Yield[2] (gpd)	2012 Yield (gpm)	1992 to 2012 Change in Yield (gpm)	1992 to 2002 Change in Yield (gpm)	2003 to 2012 Change in Yield (gpm)
Happy Jack Well Field - North								
Holman #1	475	450	312	140	200	-112	-172	60
Eddy #2	350	450	183	230	240	57	47	10
Elkar #1	300	500	188	140	145	-43	-48	5
Bailey #5	125	445	94	50	--	N.D.	-44	--
Bailey #1	0	?	n/a	n/a	n/a	93	38	n/a
Koppes #1	500	400	312	350	405	24	194	55
Koppes #2	725	365	356	550	380	-95	-15	-170
Koppes #6	200	--	175	160	80	-112	-172	-80
Happy Jack Well Field - South								
Happy Jack #1	0	n/a	n/a	n/a	n/a	-75	-62	n/a
Happy Jack #2	0	n/a	n/a	n/a	n/a	-63	37	n/a
Happy Jack #3	250	200	275	213	200	90	55	-13
King #4	500	301	263	300	200	83	3	-100
Koppes #4	250	250	225	280	315	N.D.	N.D.	35
Koppes #3	425	340	277	280	360	N.D.	N.D.	80
King #2	0	530	105	--	--	-354	-254	--
Conrey #1	225	365	222	--	--	N.D.	N.D.	--
Koppes #5	0	n/a	n/a	n/a	n/a	-206	-126	n/a
Elkar #5	750	230	754	500	400	-75	-62	-100
King #1	275	395	N.A	140	151	-63	37	11
King #5	275	230	266	140	60	90	55	-80
USGS: King #3	n/a	n/a	n/a	n/a	n/a	83	3	n/a
					Total Change in Yield	-1,639	-1,394	-729

[1] Original Well Yield and Measured Yield Data from 1994 Water Master Plan.
[2] 2002 Well Yield Data from 2003 Water Master Plan
N.A = Not Applicable
N.R. =Not Replaced

3.4.2 Geology and Hydrogeology of the Well Fields

The geology and hydrogeology in the vicinity of BOPU well fields has been described in numerous studies, most notably Morgan (1946), Lowry and Crist (1967), Ertec (1984), the 1994 and 2003 Water Master Plans, Weston (1996), and Weston (2000). Considerable detail is provided in these studies and only aspects relevant to BOPU well fields and future groundwater development will be discussed.

Figure 3-22 contains a geologic map, stratigraphic column, schematic cross-section, and data table that summarize the basic geology and hydrogeology beneath BOPU well fields. The gently east-dipping Tertiary-age sediments of the Ogallala Formation and the White River Group are exposed over an extensive area west of the City and unconformably overlie the more steeply eastward dipping and older Pennsylvanian to upper Cretaceous sedimentary units. Along the foothills of the Laramie Range, the Tertiary units have been eroded away and the underlying pre-Tertiary units are exposed along a thin north-south trending band. At the foot of the Laramie Range, the pre-Tertiary units have been deformed by westward dipping thrust faults resulting in steeply dipping, overturned, and folded strata.

Tertiary Aquifer

The Tertiary-age Ogallala Formation and White River Group are the source of groundwater to the Bell, Happy Jack, Borie, and Federal well fields. Although these two units are well-defined stratigraphically, and can be distinguished in outcrop, lithologic similarities make unit identification difficult from drill cuttings (Weston, 1996). Geophysical logs generated during the well replacement program have provided useful data to more objectively identify the subsurface contact between the Ogallala Formation and White River Group and potential high yield sand/gravel layers. The resistivity log provides the best diagnostic tool; alternating high (sand/gravel) and low (silts/clays) resistivity values characterize the Ogallala Formation, and consistent low values with occasional high values characterize the White River Group. These general observations may not strictly apply from one location to another, but do provide an improved basis for unit differentiation.

Based on deep test holes drilled at the Holman #1, Koppes #3, and Conrey #1 wells at the Happy Jack Well field, the aggregate thickness of the two units varies from 390 to 560 ft. Variation in aggregate and individual unit thickness is due to topographic location and the shape of the predeposition erosional surface (i.e., the contacts between the Lance/White River and White River/Ogallala are erosional unconformities).

Hydraulically, the SEO considers the Tertiary High Plains aquifer units in southeast Wyoming – the Ogallala, Arikaree, and White River – as one hydraulic unit and, consequently, a well can be completed in more than one unit. The WDEQ treats wells across more than one water bearing zone differently. Chapter 12, Section 9. (b) (iii) (b) (IX) of the WQD Rules of Regulations addresses wells that penetrate more than one aquifer. This section requires that wells be constructed with impervious seals to prevent migration of water from one aquifer or water bearing strata to another. In the case of the High Plains Aquifer, WQD currently interprets this rule to allow wells to be screened across one water bearing strata only. A definition of "water bearing strata" is not provided in the regulations, so this interpretation is somewhat subjective.

Crist (1980) developed a finite difference groundwater model for the County that considered the Ogallala Formation, Arikaree Formation, and White River Group as one aquifer. At BOPU well fields, the Arikaree Formation is absent due to erosion.

Ogallala Formation

The Ogallala Formation is exposed at the Bell, Happy Jack, and Borie well fields. The Ogallala is composed of well to poorly-sorted fine to coarse-grained sandstone, granule to pebble conglomerate, siltstone, and minor beds of claystone, volcanic ash, and limestone (Weston, 1996). In general, the sand and gravel were derived from the Sherman Granite in the Laramie Range, and the silts and clays from erosion of the underlying White River Group. The poor to moderately cemented sediments of the Ogallala Formation obtain a thickness of up to 345 ft in the vicinity of the Borie Well field, and at the Happy Jack Well field the average thickness is 250 ft. Along the reach of Crow Creek in the Bell Well field, the Ogallala Formation thins to approximately 155 ft (Bell #8 and Bell #11) and may be absent at Bell #5 (Weston, 2000b). However, the absence of Ogallala Formation at Bell #5 is not shown in the cross-sections through the Bell Well field (Weston, 2000c), which attests to the difficulties differentiating between the two units. Recent explorations on the Belvoir Ranch property have shown sufficient Ogallala thickness for municipal well development.

Morgan (1946) provides an excellent description of the general spatial distribution of fine and coarse-grained sediments in the Ogallala Formation near Cheyenne:

> *"It [Ogallala Formation] is composed largely of silt and clay with interbedded lenses and beds of sand and gravel. The proportion of sand and gravel to silt and clay is highest near the mountains and decreases eastward toward Cheyenne. East of a north-south pipeline that corresponds approximately to the pipeline between Ranges 67 and 68 West, gravel and sand beds form only a small part of the formation and near Cheyenne gravel lenses have been encountered by only a few wells. West of the pipeline, gravel beds and lenses form a large part of the formation and appear to be interconnected. Eastward from the main mass of sand and gravel near the mountains, individual lenses and stringers of sand and gravel extend finger-like into the silt and clay beds of the eastern part of the formation. North of Crow Creek the textural gradation from west to east is not as pronounced as in the area south of Crow Creek and the entire formation [north of Crow Creek] is made up predominately of silt, clay, and fine sand."*

Morgan's observations indicate that the Ogallala Formation is noticeably finer grained north of Crow Creek and east of BOPU well fields. A detailed drilling project conducted by the USGS (Ogle and Hallberg, 2000) northwest of the Veterans Administration Hospital in the City also indicated that the Ogallala Formation east of the well fields was predominantly fine-grained, consisting primarily of sands, silts, and clays.

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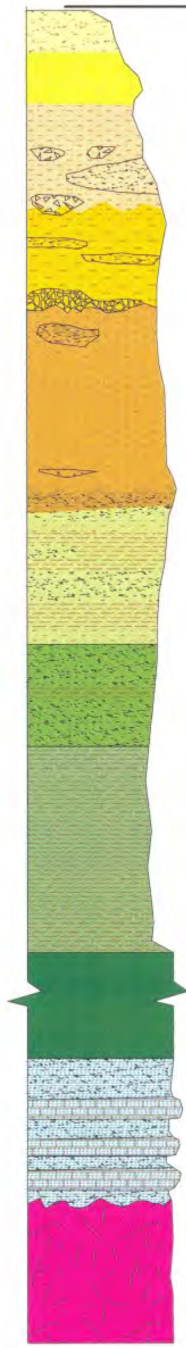
Figure 3-22
Geology beneath BOPU Well Fields

LITHOLOGY



THICKNESS FT.	AGE	UNIT	LITHOLOGIC DESCRIPTION	AQUIFER DESCRIPTION	AQUIFER PROPERTIES**
0-85	QUATERNARY	ALLUVIUM	STREAM CHANNEL SILTS, SAND, AND GRAVEL WITH SOME COBBLES AND BOULDERS	WHERE DEPOSITED, MAY PRODUCE SMALL TO LARGE YIELDS, GOOD WATER QUALITY.	YIELDS: 20 - 1,700 GPM TRANMISSIVITIES: 10,000 - 230,000 GPD / FT STORAGE COEFFICIENTS: 10E-3 - 10E-2 HYDRAULIC CONDUCTIVITIES: 690 - 3200 GPD / FT²
0-200	QUATERNARY	TERRACE			
0-500	TERTIARY – PLIOCENE	HIGH PLAINS AQUIFER – OGALLALA	HETEROGENEOUS MIX OF INTERBEDDED AND LENTICULAR SILT, CLAY, SAND, GRAVEL, AND CEMENTED CONGLOMERATE DEPOSITED BY ALLUVIAL FAN COMPLEX.	REGIONAL AQUIFER, SMALL TO LARGE YIELDS, GOOD WATER QUALITY.	YIELDS: 5 - 1,800 GPM TRANMISSIVITIES: 45 - 343,000 GPD / FT STORAGE COEFFICIENTS: 10E-5 - 10E-2 HYDRAULIC CONDUCTIVITIES: 164 - 4000 GPD / FT²
0-400	TERTIARY – MIOCENE	HIGH PLAINS AQUIFER – ARIKAREE	LOOSE TO WELL CEMENTED VERY FINE GRAINED TO FINE-GRAINED GRAY TO WHITE SANDSTONE AND SILT, CONTAINING HARD CONCRETIONARY ZONES. MAY BE A COARSE CHANNEL CONGLOMERATE AT BASE IN SOME AREAS.	REGIONAL AQUIFER, SMALL TO LARGE YIELDS, GOOD WATER QUALITY.	YIELDS: 5 - 1,600 GPM TRANMISSIVITIES: 110 - 77,000 GPD / FT STORAGE COEFFICIENTS: 10E-4 - 10E-3 HYDRAULIC CONDUCTIVITIES: 10 - 100 GPD / FT²
0-480	TERTIARY – OLIGOCENE	HIGH PLAINS AQUIFER – WHITE RIVER GROUP – BRULE	BUFF TO PINK MASSIVE SILTSTONE WITH OCCASIONAL BEDS OF VOLCANIC ASH AND IRREGULAR DEPOSITS OF SANDSTONE AND GRAVEL.	REGIONAL AQUIFER, SMALL TO LARGE YIELDS, GOOD WATER QUALITY.	YIELDS: 1 - 2,000 GPM TRANMISSIVITIES: 2,600 - 780,000 GPD / FT STORAGE COEFFICIENTS: 10E-5 - 10E-1 HYDRAULIC CONDUCTIVITIES: 0.1 - 6000 GPD / FT²
0-100	TERTIARY – OLIGOCENE	HIGH PLAINS AQUIFER – WHITE RIVER GROUP – CHADRON	SILICA CEMENTED ARKOSIC SANDSTONE AND CONGLOMERATE WITH SILTSTONE/CLAYSTONE. NEAR LARAMIE RANGE.		
0-1500	UPPER CRETACEOUS	LANCE FORMATION	INTERBEDDED SANDSTONE AND SHALE WITH OCCASIONAL COAL.	SMALL TO MODERATE YIELDS, GOOD WATER QUALITY IN SOME AREAS.	YIELDS: 14 - 239 GPM TRANMISSIVITIES: 65 - 5,400 GPD / FT STORAGE COEFFICIENTS: 10E-5 - 10E-3
+/- 250	UPPER CRETACEOUS	FOX HILLS SANDSTONE	MED-GRAINED GRAY TO YELLOW-BROWN SILTY SANDSTONE WITH OCCASIONAL DARK SHALE.	SMALL YIELDS, GOOD WATER QUALITY IN SOME AREAS.	
+/- 5700	UPPER CRETACEOUS	PIERRE SHALE	DARK GRAY SHALE WITH MINOR SANDSTONE INTERBEDS.	REGIONAL AQUITARD, POOR WATER QUALITY.	
			~2,400 FEET OF UPPER CRETACEOUS (NIOBRARA) TO TRIASSIC ROCK (CHUGWATER/GOOSE EGG).	AQUIFER CHARACTERISTICS POORLY UNDERSTOOD, YIELDS AND WATER QUALITY GENERALLY BELIEVED TO BE POOR.	
800-1,300	PENNSYLVANIAN	CASPER FORMATION	INTERBEDDED RED SANDSTONE AND GRAY LIMESTONE/DOLOMITE, OCCASIONAL RED SHALE. ARKOSIC SANDSTONE AT BASE.	REGIONAL AQUIFER, POTENTIAL FOR LARGE TO MODERATE YIELDS AND GOOD WATER QUALITY IN RECHARGE AREA.	YIELDS: 600 GPM TRANMISSIVITIES: 43,000 - 59,000 GPD / FT HYDRAULIC CONDUCTIVITIES: 45 - 62 GPD / FT²
	PRECAMBRIAN	SHERMAN GRANITE	PREDOMINANTLY GRANITE, GNEISS, AND SCHIST	LOW YIELDS, GOOD WATER QUALITY	

NOT TO SCALE

* GENERALIZED STRATIGRAPHIC COLUMN COMPILED FROM: LOWRY AND CRIST, 1967 AND BLACK AND VEATCH, 2003.
** AQUIFER PROPERTY VALUES REPRESENT RANGES OF KNOWN VALUES FOR LARAMIE COUNTY FROM NUMEROUS CONSULTANT AND USGS REPORTS.

Morgan (1946) postulated that the depositional environment for the Ogallala Formation was an alluvial fan complex centered on Lone Tree Creek. Weston (1996) provides a detailed discussion of depositional environments and the interaction of alluvial fans (proximal and distal portions), debris flows, braided streams, and the anticipated sedimentary textures from each depositional process.

Two slightly different viewpoints have been developed regarding the subsurface distribution of sediments in the Ogallala Formation. Ertec (1984) simplified the Ogallala Formation into three layers at the Bell, Happy Jack, and Borie well fields. From top to bottom: a near surface gravel, an intervening fine-grained confining layer, and a basal gravel. Weston (2000c) provides an alternative viewpoint with a detailed geometry of discontinuous sand/gravel channels surrounded by finer grained material. Detailed lithologic cross-sections through all of the well fields are provided in the 2000 Wellhead Protection Plan. Cross sections of the Belvoir Ranch were completed as part of recent WWDC studies (2005, 2007, 2008, and 2012).

The primary distinguishing feature of the Ogallala Formation is the heterogeneous nature and lateral variability of fine and coarse grained sediments. For example, in the 1930s to 1950s approximately 61 test wells were drilled in the area of the Bell, Happy Jack, and Borie well fields to obtain 34 production wells. The localized nature of sand and gravel deposits was also demonstrated during the well replacement program when the Bailey #5 and Conrey #1 replacement wells did not encounter the productive sand and gravel deposits present in the nearby original well (Weston, 1996). Coarse-grained deposits are not always present and wells drilled in close proximity can encounter noticeably different lithology. Unique interference patterns between wells during pumping (Morgan, 1946; Weston, 1996) are also a direct reflection of the directional (anisotropic) nature of the sand and gravel deposits. As stated by Theis (1941), "the gravels that furnish the water to the wells are very erratic in their occurrence." Previous studies also indicate that the more transmissive channels have a northeasterly trend (Morgan, 1946; Weston, 1996).

Although many of the wells in the Bell, Happy Jack, and Borie well fields have some portion of the screened interval in the underlying White River Group, it is generally believed that the vast majority of the groundwater produced from these well fields is derived from sand and gravel channels in the Ogallala (Weston, 2000c; 1994 Water Master Plan). Consequently, the transmissivity values listed in Table 3-16 (except the Federal Well field) calculated from pump tests are representative of the permeable portions of the Ogallala Formation. Transmissivity values range from 2,480 to 62,700 gpd/ft with an average value of 20,500 gpd/ft. Storage coefficients calculated from the pump tests vary from 0.08 to 0.000059 and indicate confined conditions (Weston, 2000c).

Aquifer characteristics based on production well pump tests are skewed toward the more productive part of the Ogallala Aquifer (Lowry and Crist, 1967). Even so, Theis (1941) commented that, "the [Ogallala] Aquifer is only a mediocre aquifer." Based on whole-aquifer yields and hydraulic gradients, Lowry and Crist (1967) estimated an effective transmissivity of approximately 3,800 gpd/ft. During the calibration of a groundwater flow model, Ertec (1984) estimated even lower transmissivity values at the Bell and Happy Jack well fields of approximately 300 gpd/ft.

In the vicinity of the Bell, Happy Jack, and Borie well fields, the depth to groundwater in the Ogallala Aquifer varies from 0 ft along Crow Creek (e.g., Bell #6 and Bell #11 flow occasionally) to over 200 ft at the most western wells of the Happy Jack Well field (e.g., Koppes #6, Conrey #1, King #5, and King #1). The thickness of the Ogallala Aquifer increases from west to east, being less than 50 ft thick at the western wells and over 200 ft thick at the eastern wells (i.e., Holman #1 and Weber #1); this geology is illustrated in the 2003 Master Plan Figure 3-22.

Piezometric surface maps have been generated for most of Laramie County for the years 1977, 1994, and 2004, as well as predevelopment (WWDC, 2008). The latest surface was produced by the USGS (2011) for the year 2009. Groundwater generally flows from southwest to northeast through the Borie, Happy Jack, and Bell well fields.

Recharge to the Ogallala Aquifer occurs primarily from two sources: (1) direct infiltration of precipitation on the outcrop and (2) seepage from streams that flow eastward across the outcrop (Morgan, 1946), with precipitation infiltration being the dominant mechanism for recharge (1994 Water Master Plan). Morgan (1946) estimated that approximately 0.83 inches/yr (i.e., 5.5 percent of average annual precipitation) recharges the Ogallala Aquifer from the infiltration of precipitation. This estimate was used later by Lowry and Crist (1967), and the groundwater model developed by Crist (1980) was calibrated satisfactorily using the 0.83 inches/yr value for the post-Mesozoic formations which include the White River Group (1994 Water Master Plan).

The loss of stream flow to the Ogallala occurs at Lone Tree Creek, Goose Creek, Duck Creek, and Crow Creek. Foley (1942) estimated recharge to the Ogallala Aquifer from Duck, Goose, and Lone Tree Creeks to be about 2.5 mgd. In the 1940s, groundwater in the Tertiary Aquifer discharged into Crow Creek (Theis, 1941) as indicated by the water table surface in 1943 (Morgan, 1946).

White River Group

The White River Group is exposed in the Federal Valley, between the Islay Escarpment and the Laramie Range). The Federal Well field is situated on the White River Group and all of the production wells obtain water from this unit. At the Federal Well field, where the upper part of the unit has been eroded away, the White River Group has a thickness ranging from 170 to 350 ft (Weston, 2000c). Beneath the Bell, Happy Jack, and Borie well fields, where the White River Group is overlain by the Ogallala Formation, the White River Group attains a thickness ranging from 110 to 400 ft.

The White River Group consists "primarily of massive brittle argillaceous siltstone containing a few beds of sandstone, conglomerate, and volcanic ash" (Lowry and Crist, 1967). The ash beds were used by Ogle and Hallberg (2000) to distinguish the White River Group from the Ogallala Formation. The siltstone is eolian and fluvial re-worked eolian deposits while the sand and gravel layers are fluvial (Weston, 2000c). As a whole, the predominance of massive siltstone with occasional sand and gravel is the primary characteristic that distinguishes the White River Group from the Ogallala Formation. In outcrop, near vertical clastic dikes are also diagnostic of the White River Group. The siltstone has poor intrinsic permeability and wells must penetrate local sand/gravel channels or fractures to obtain adequate yields.

The Federal Well field is one of the few areas in Wyoming where the White River Group is productive. Resistivity logs at Merritt #6, State #2, and Merritt #8 indicate sand and gravel layers in the lower part of the unit, however, recent cross-sections through the Federal Well field (Weston, 2000c) show localized sand/gravel channels. Well drilling efforts in the Federal Well field area have experienced many dry/low yield wells interspersed with productive wells, which supports the presence of localized channels. When sand/gravel layers are encountered, well yields and transmissivity values at the Federal Well field are similar to, but slightly less than, the Ogallala Aquifer. The average yield at the Federal wells is 230 gpm and the average transmissivity value calculated from pump tests is 9,000 gpd/ft. Storage coefficients indicate confined conditions in the White River Group.

During groundwater flow model development, Ertec (1984) used a transmissivity value of 692 gpd/ft for the White River Group applied to the Federal Well field area. Like the Ogallala Formation, the large disparity between measured and modeled transmissivity values for the White River Group reflect the heterogeneity of the aquifer such that the aquifer as a whole is much less permeable than the permeable sand/gravel layers penetrated by the wells (1994 Water Master Plan).

According to Morgan (1946), the productive sand and gravel layers encountered in the White River Group at Federal do not appear to extend to the east beneath the other BOPU well fields. Most of the Happy Jack wells penetrate only the upper 100 ft of the White River which tends to be fine-grained. However, a few wells have penetrated the entire White River Group based on the resistivity logs from deep test holes at Koppes #3 and Conrey #1, which indicate a few 5 to 10-ft thick sand/gravel layers at the base of the unit. The lower portion of the Conrey well was screened in the White River Group, and pump tests indicated very poor yield. The Koppes #3 well is screened across sand/gravel of both the Ogallala Formation and White River Group and initially had excellent production characteristics. Unfortunately the relative contribution of the White River Group sediments to the Koppes #3 well cannot be determined with available data. The Elkar #1 is screened across an 80 ft thick sequence of sand/gravel (as indicated by the resistivity log) in the lower part of the White River Group and a portion of the overlying Ogallala Formation, but the well yield of 140 gpm is marginal. In general, it appears that the White River Group beneath the Happy Jack Well field is not very productive.

In Pine Bluffs and western Nebraska, productive irrigation wells extract water from fractured White River Group. The role of fractures in the White River siltstone at the Federal Well field is not known (Weston, 2000c). Groundwater in the White River Group generally flows from west to east. Recharge mechanisms to the White River Aquifer are probably similar to those of the Ogallala Aquifer, but are less understood. Recharge occurs from the infiltration of precipitation over the outcrop area and the loss of stream flow along Crow Creek, North Fork of Crow Creek, and South Lodgepole Creek.

3.4.3 Groundwater Rights and Agreements

Table 3-15 also summarizes the status of groundwater rights at the municipal production wells as of 2012. During the well replacement and rehabilitation program, the original permit number, adjudicated water right, and priority date at a well were relocated to the new offset well. If production from the new well exceeded the original water right, enlargements were filed with new priority dates (i.e., Merritt #5, Merritt #14, Bell #5, Bell #8, Eddy #2, Koppes #1, Koppes #2, Koppes #3, King #4, Elkar #5, Borie #1, and Elkar #7). Enlargements have also been filed at Merritt #6 (U.W. 103571) and Merritt #8 (U.W. 103572); however, these enlargements are for stock use only. The original wells that were converted into monitoring wells were given new permit numbers that reflect their current use for monitoring purposes.

Groundwater Control Areas

Groundwater development in the County had reached such extent that in 1981 the Laramie County Groundwater Control Area (LCGCA) was established in the eastern three-fourths of the County. As shown on Figure 3-23, the western boundary of the LCGCA is 2.3 miles northeast of the Bell Well field and, therefore the Federal, Bell, Happy Jack, and Borie well fields are not within the jurisdiction of the LCGCA. Within the LCGCA, however, any groundwater development involving more than small-yield stock and domestic wells must be reviewed by the LCGCA Advisory Board. Since the establishment of the LCGCA, only a few new large-yield water wells have been permitted in the control area and, in general, have not resulted in an increase in historic withdrawals. It would be difficult for BOPU to obtain permits for new municipal wells within the LCGCA.

The SEO is currently developing a County-wide groundwater model to determine if modifications to the LCGCA are needed. The modeling effort is currently underway as of this writing. Results may include recommendations for changes to the LCGCA, which will likely affect future municipal well field development for the City and much of Laramie County.

Another area of local groundwater regulation is the North Cheyenne Study Area (NCSA). The southwest corner of the NCSA is shown on Figure 3-23 and encompasses the eastern most portion of the Bell Well field including Bell #6, Bell #16, and Bell #24. Due to numerous domestic well installations in the area north of the City, increased drawdown conflicts between users and a groundwater model that predicted significant additional groundwater level declines, groundwater development in the NCSA is subject to detailed review. In the NCSA, all well applications are subject to case-by-case review by the SEO and must meet certain construction standards designed to ensure long-term groundwater availability. Groundwater development is also constrained by the requirement that any permitted well be put to beneficial use within one year.

It is unlikely that additional groundwater development by BOPU in the area east and northeast of the Bell Well field would be compatible with the groundwater management objectives of the LCGCA and NCSA. Development of groundwater to the south, west, and north of the existing well fields is not constrained by the LCGCA and NCSA. However, BOPU needs to recognize the trend toward increasing control and limitation of groundwater development near the City.

Polo Ranch Agreement

In 1955, BOPU entered into a "Drilling and Water Use Agreement" with John Bell that gave BOPU exclusive right to drill for and use groundwater from the Bell property situated along the Crow Creek Basin. The agreement does not limit the development of water rights by BOPU within the agreement area. The 1955 agreement applies to the Polo Ranch Company via a 1977 agreement. Figure 3-24 shows the Polo Ranch agreement area that includes all of the Bell Well field.

Permits for the Bell wells have BOPU and the Polo Ranch as co- applicant/assignee. As part of the 1977 agreement, the following wells were permitted for municipal, stock, domestic, and irrigation use: Bell #5, Bell #6, Bell #8, Bell #10, Bell #11, Bell #12, and Bell #17. The following wells were permitted for municipal, stock, and domestic use: Bell #16, Bell #24, and Bell #25.

In exchange for exclusive access and use of groundwater, BOPU must provide the following to the Polo Ranch:

- One-sixth of the first 155.5 million gallons produced by BOPU during the 12-month period beginning October 1st of each year.
- One-eighth of all water produced in excess of 155.5 million gallons during the 12-month period beginning October 1st of each year.
- If total production during the 12-month period beginning October 1st exceeds 1 billion gallons, then the one-sixth rule does not apply, and the Polo Ranch is entitled to one-eighth of the total production by BOPU in the agreement area.

Over the 20-year period from 1983 to 2002, the average annual production from the Bell Well field has been 460.1 million gallons (1,410 ac-ft) and the maximum annual production was 813.3 million gallons (2,500 ac-ft) (2003 Master Plan). Per the terms of the agreement, on an average annual basis, BOPU provides the Polo Ranch approximately 200 ac-ft of water from the Bell Well field.

As of 2012, the water rights from production wells at the Federal, Bell, Happy Jack, and Borie well fields total 15,220 gpm (24,550 ac-ft/yr). As shown on Figure 3-23 all of the water rights are adjudicated except the enlargements at Bell #8, and Koppes #3. BOPU should proceed with the adjudication of enlargements at these wells. Based on actual pumping rates in 2012 at individual wells, production at the Koppes #4 well exceeds the adjudicated water right and BOPU should file an enlargement for this well.

With respect to the protection of adjudicated groundwater rights, BOPU must, at a minimum, record that a well was pumped at the adjudicated rate once every 5 years. This protects BOPU from potential claims of water right abandonment from other users. The party claiming abandonment must demonstrate that they can, in fact, beneficially use (and extract) the amount of water claimed for abandonment. This is similar to surface water right abandonment procedures.

BOPU has recorded annually that wells were pumped at the adjudicated rate, and should continue to do so. In essence, this has ensured that no well "slips through the cracks" by not being pumped once every 5 years. The well does not have to be pumped for any specified time period.

Once a well has been adjudicated, the SEO does not go back on a defined schedule to verify the continued ability to pump the adjudicated rate. The SEO will only go back to test a well if there is a claim of abandonment on the water right. If BOPU installs a new pump with a greater discharge capacity compared to the original pump upon which the adjudication was based, then an enlargement needs to be filed. If a smaller capacity pump is installed, then BOPU runs the risk of an abandonment claim if the well cannot be pumped at the adjudicated amount once over a 5-year period.

Figure 3-23
LCGCA Boundary

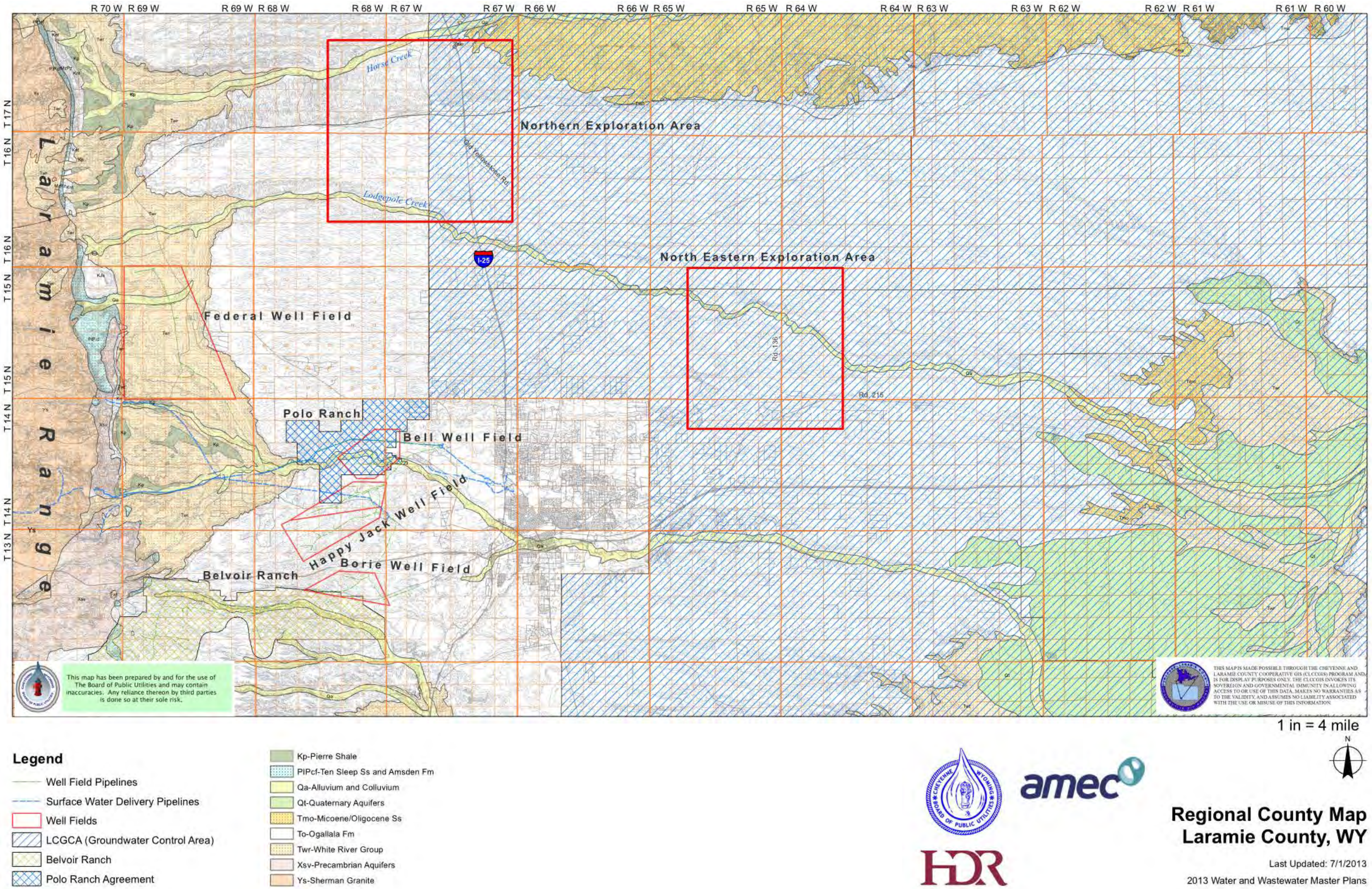
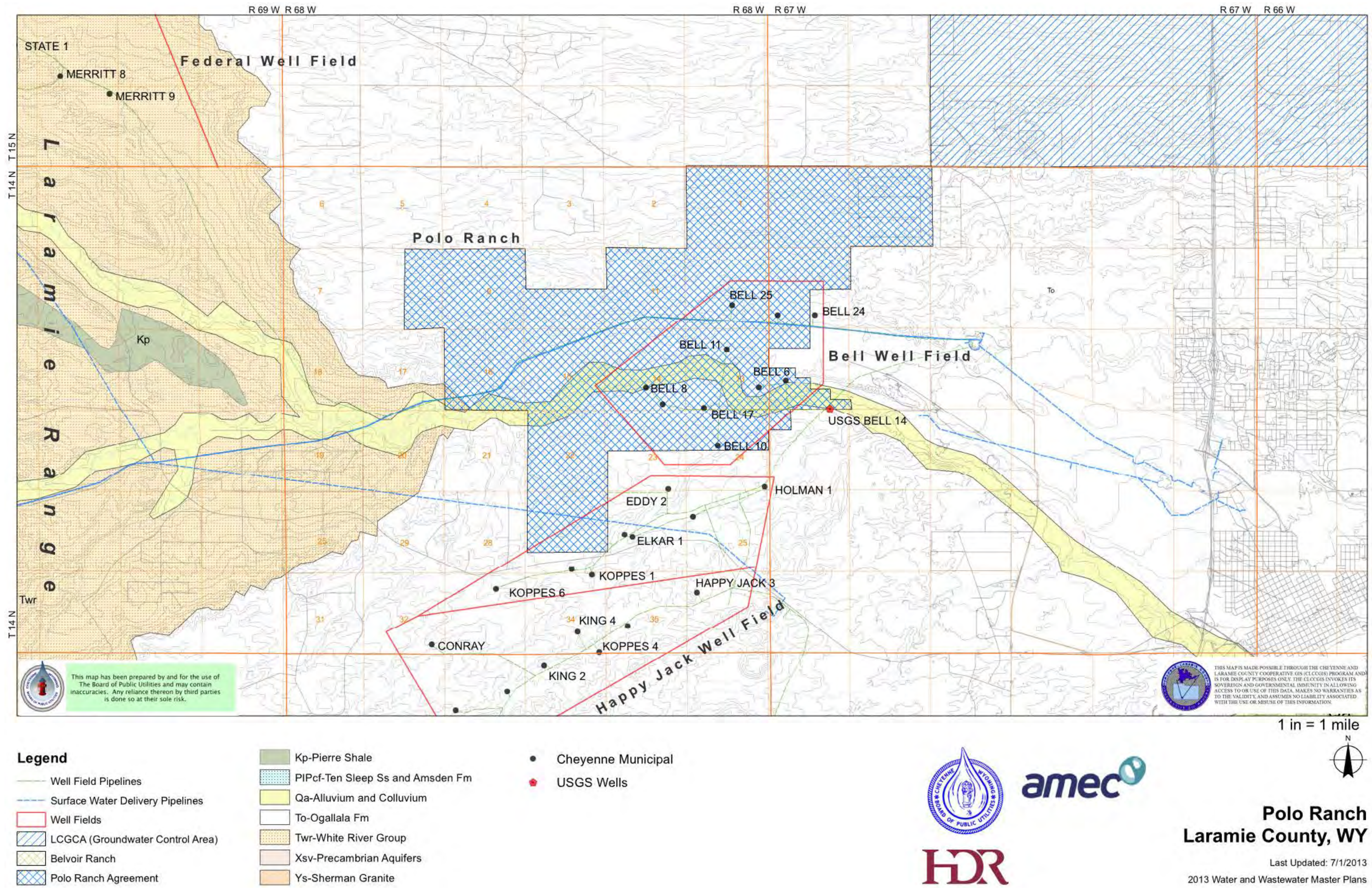


Figure 3-24
Polo Ranch Agreement Location and Boundary



3.4.4 Well Field Performance

In 1992, BOPU municipal wells were adjudicated by the SEO based on the demonstrated production of each well. Enlargements were filed when the demonstrated production at a well exceeded the original water right. In addition to setting production limits at each well, the 1992 adjudication established limits on the aggregate production from the entire well field system such that BOPU is allowed to pump a total combined quantity of 5,500 ac-ft of groundwater on an average annual basis and a combined total quantity of no more than 10,000 ac-ft of groundwater in any one calendar year from its Bell, Happy Jack, Borie, and Federal well fields. Such average annual pumping is computed on the basis of 10 consecutive years commencing with the calendar year 1991. BOPU also is prohibited from pumping more than 55,000 ac-ft of groundwater in any 10-year period from its Bell, Happy Jack, Borie, and Federal well fields under current rights.

These aggregate values were based on the historic production and beneficial use of BOPU wells, with allowances for higher demand drought-years, rather than on the sum of individual well appropriations which would total 23,990 ac-ft if pumped continuously for a year at the full appropriation. According to the SEO personnel supervising the adjudication process, the annual limits are not intended to reflect a judgment as to the sustainable production potential or "safe yield" of the aquifers supplying the well fields. The average and maximum production limits may or may not be physically available in a given year, and production within these limits does not preclude conflicts with other groundwater appropriators (1994, 2003 Water Master Plans).

Over the past 12 years, since the well field production limits have been in effect, production from the Bell, Happy Jack, Borie, and Federal well fields has been below the limits set for 10-year, 1-year, and average annual production. From 2000 to 2012, the 10-year total production was 47,718 ac-ft, the average annual production was 4,322 ac-ft, and the one-year maximum production was 5,121 ac-ft. Running 10-year averages have remained below the established production limits.

In the event that BOPU develops additional groundwater supplies, such as the Belvoir Ranch, the administration of the well field production limits by the SEO is not known precisely. Because the Federal, Bell, Happy Jack, and Borie well fields derive groundwater from the Tertiary Aquifer, the SEO may consider that any future additional development in the Tertiary Aquifer would be included within the well field production limits. However, if BOPU were to develop groundwater from a different aquifer system, a technical argument can be made to the SEO that the new well field should not be included in the production limits set for the Federal, Bell, Happy Jack, and Borie well fields. Regardless, the expansion of existing well fields and/or the development of new groundwater resources capable of exceeding the established well field production limits will require that BOPU negotiate and apply to the SEO for an enlargement of the aggregate well field production limits.

Groundwater Portion of Water Supply

Over the period from 1991 to 2002, the relative contribution of groundwater to BOPU's total water supply has varied from 24 percent to 37 percent, with an average annual contribution of 30 percent (the 2003 Water Master Plan). In general, groundwater has been used for three purposes, with a fourth purpose recently recognized during the 2002 drought:

- To blend with surface water for pipeline corrosion control and compliance with drinking water standards.
- To "make up the difference" between the surface water supply and demand.
- As a peaking supply during the high demand summer months.
- As a more drought resistant supply compared to surface water.

Table 3-18 provides an overview of groundwater production from 2003 to 2012 at the Bell, Happy Jack, Borie, and Federal well fields. Well field production graphs also show the relative contribution of each well field (Figure 3-25). An historic overview of groundwater production is presented in the 2003 Water Master Plan.

Table 3-18
Average Annual Well field Production (2003 to 2012)

Well Field	Average Production (ac-ft/yr)	Relative Contribution (percent)
Bell	1,112	24
Borie	1,177	25
Federal	765	17
Happy Jack	1,576	34
Total	**4,629**	**100**

Historic Well Field Production

Records from the 2003 Water Master Plan indicate historic total well field production ranges from 500 to 8,300 ac-ft/yr. Groundwater production from the period 1991 to 2002 has ranged from 3,200 to 6,400 ac-ft, and has averaged approximately 4,400 ac-ft/yr. From 1941 to 2002, the annual average well field production was 3,400 ac-ft. During this time period, the annual well field production ranged from a minimum of 520 ac-ft in 1949 to a maximum of 8,380 ac-ft in

1977. For the period 2003 to 2012 the average production was 4,600 ac-ft. Some of the annual variations in well field production can be explained by precipitation patterns. Figure 3-26 shows annual precipitation at the Hecla Station (T14N, R69W, Sec. 30) from 1943 to 2002. In general, annual well field production corresponds inversely with annual precipitation (1994, 2003 Water Master Plans). During years of low precipitation, surface water supplies are stressed and water demand is high (especially during summer-time lawn irrigation), and consequently the groundwater supply is called upon to make up the difference.

There are three time periods with distinctive well field production patterns as described below:

- **1941 to 1951.** Annual average well field production of 1,700 ac-ft during the initial phase of well field development (Happy Jack, Borie, and Federal [partial]).
- **1952 to 1993.** Annual average well field production of 3,500 ac-ft. Full well field development by 1957, expansion of surface water supplies in 1965 and 1988, and prior to well replacement/rehabilitation program.
- **1994 to 2002.** Annual average well field production of 5,100 ac-ft. Period of active well replacement and rehabilitation.
- **2003 to 2012.** Annual average well field production of 4,322 ac-ft. Making up approximately 29 percent of total water extracted from both surface and groundwater supplies.

From 2003-2012, the average annual production from the four well fields has been distributed as shown in Table 3-18.

Monthly Pattern of Well Field Production

Figure 3-27 illustrates the average monthly groundwater production during a calendar year for the period 2003 to 2012. Average monthly production during the 6-month period from November to April is 207 ac-ft, increases to a maximum of 968 ac-ft in July, and decreases gradually from August to November. Approximately 45 percent of the total annual production occurs during the summer months of June, July, and August, and 72 percent of the annual production occurs during the 6-month period from May to October. The monthly pattern of groundwater production demonstrates the use of groundwater as a peaking supply.

Figure 3-28 illustrates average well field production during June, July, and August from 2003 to 2012. Over this 10-year period, well field production capability during the peak demand period has averaged 38 ac-ft/day and has remained fairly consistent. Similar to annual trends, combined production from the Bell and Happy Jack well fields is approximately 58 percent of the total production during the summer months.

The monthly production figures are based on the most recent historic production data from BOPU. Future production from the Bell, Happy Jack, Borie, and Federal well fields may change when the remaining original and out-of-service wells have been replaced or repaired and depending on the response of the High Plains aquifer to pumping.

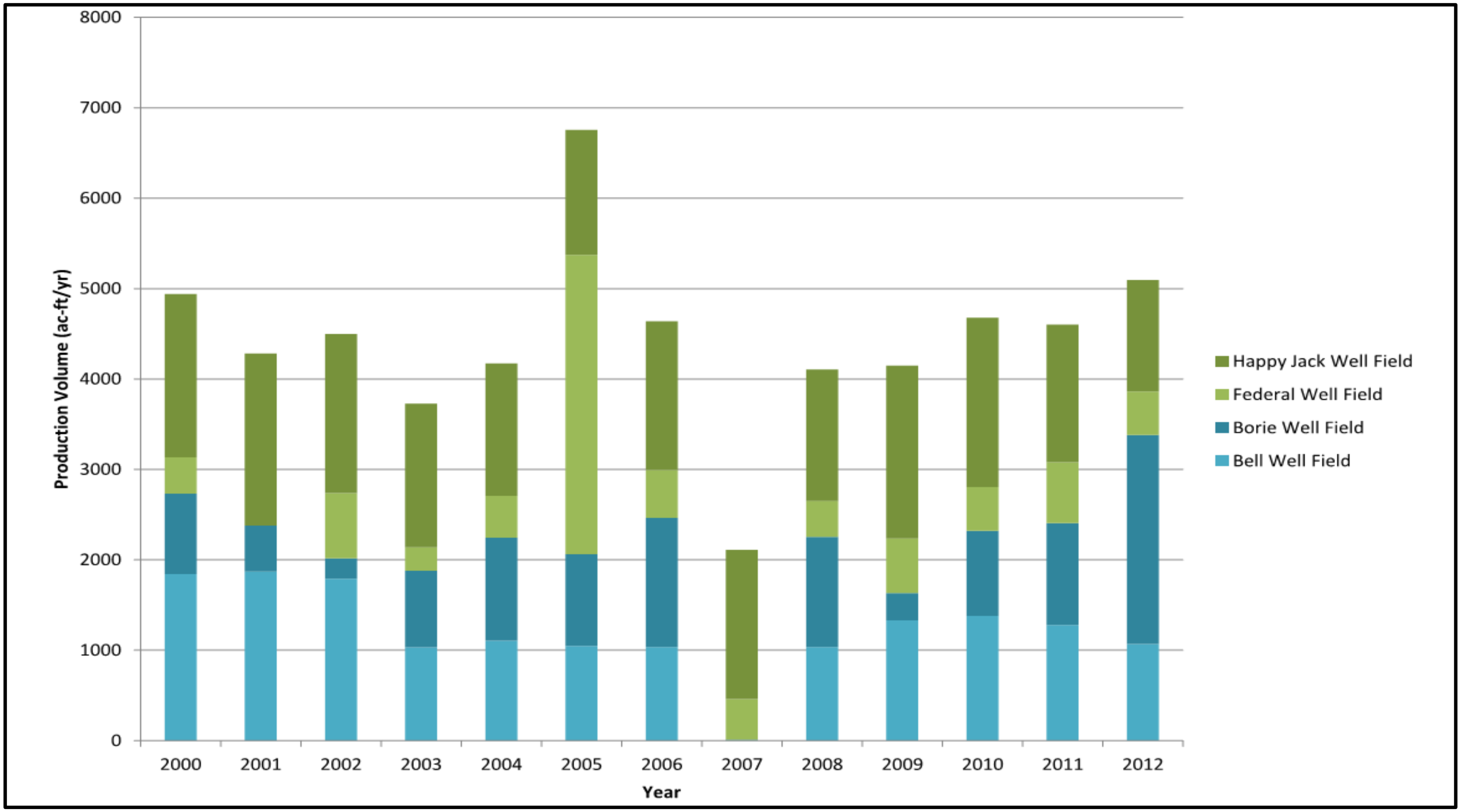


Figure 3-25
Well Field Production 2000 to 2012

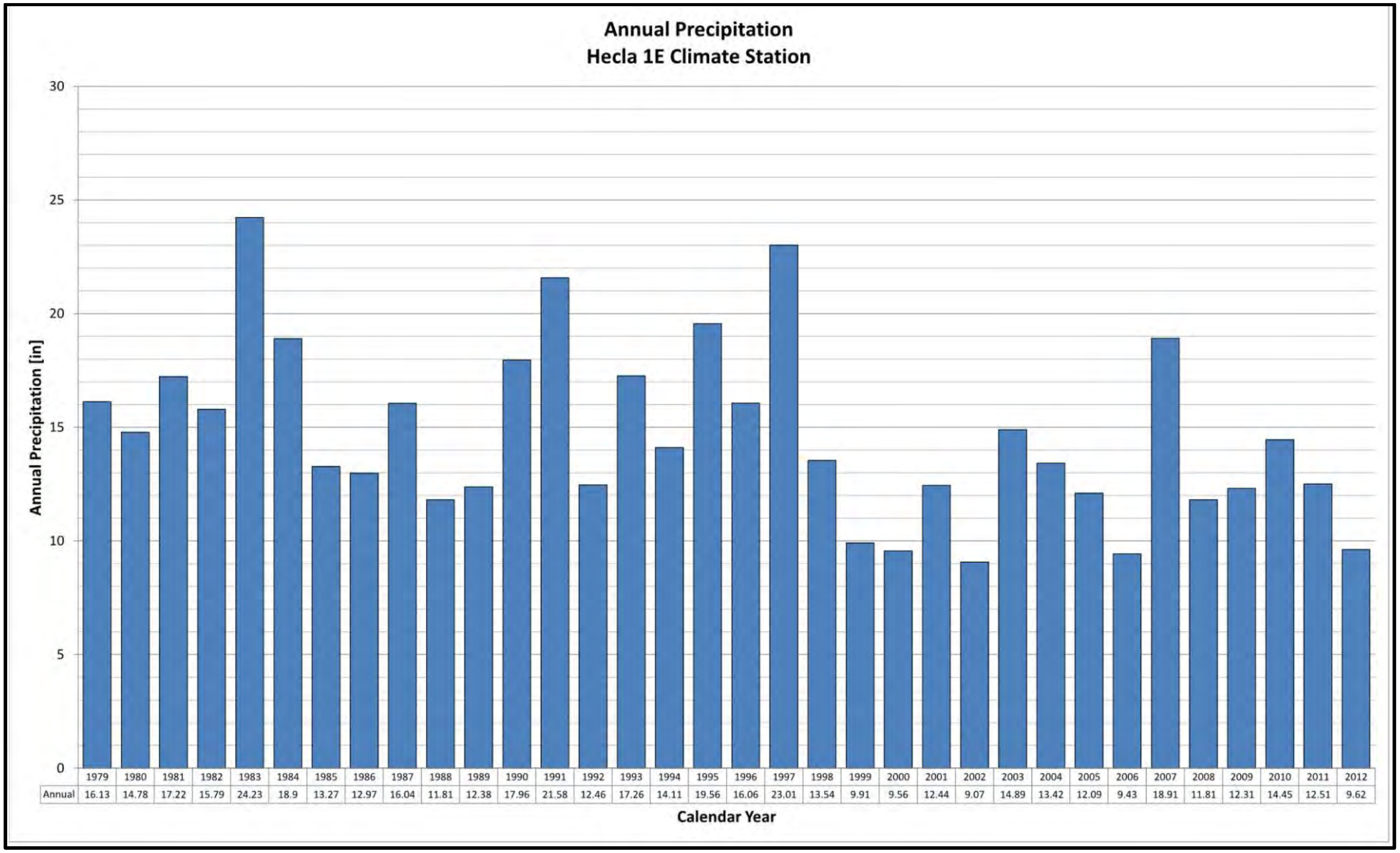


Figure 3-26
Annual precipitation at the Hecla Station

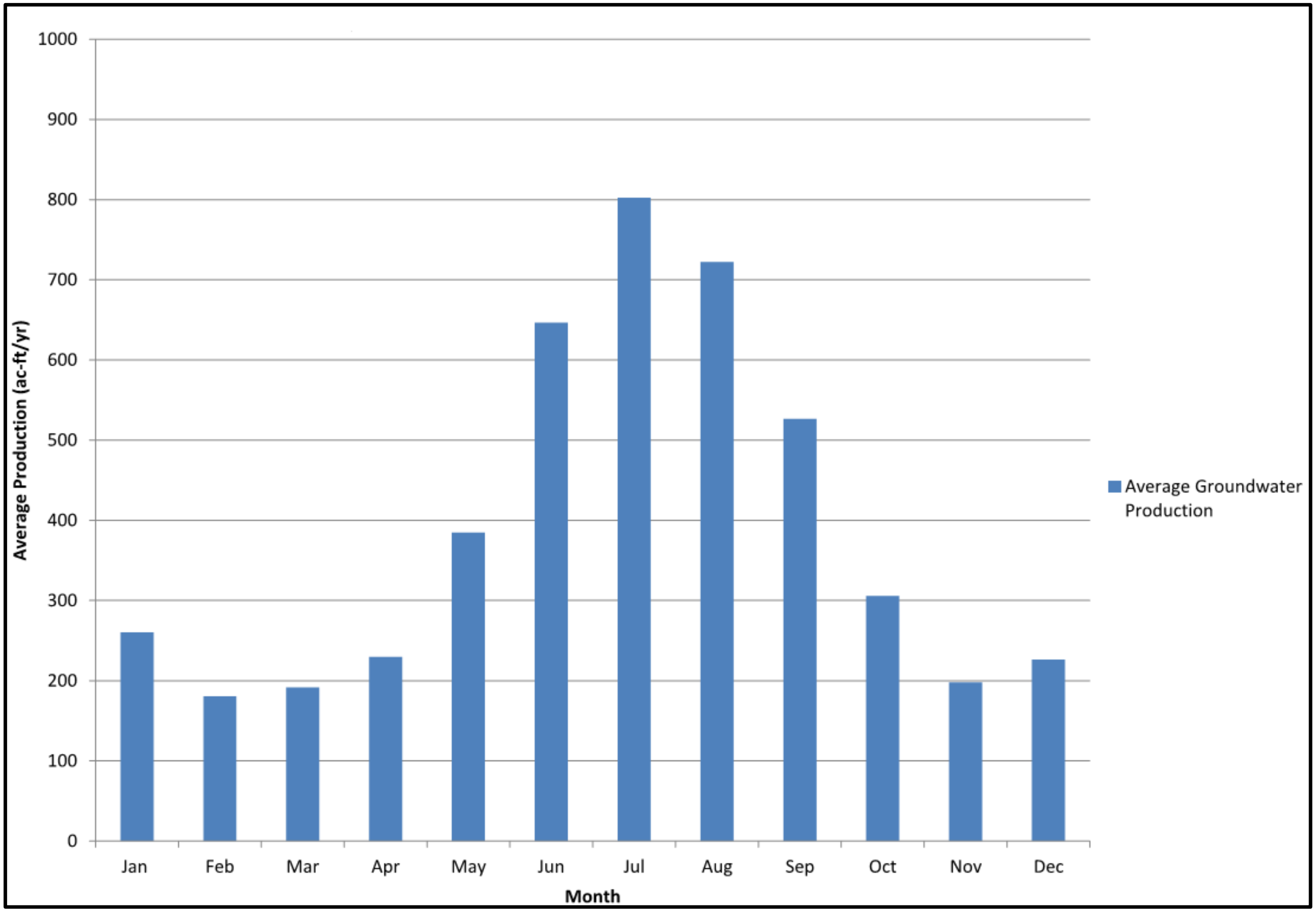


Figure 3-27
Average Monthly Groundwater Production (2003 to 2012)

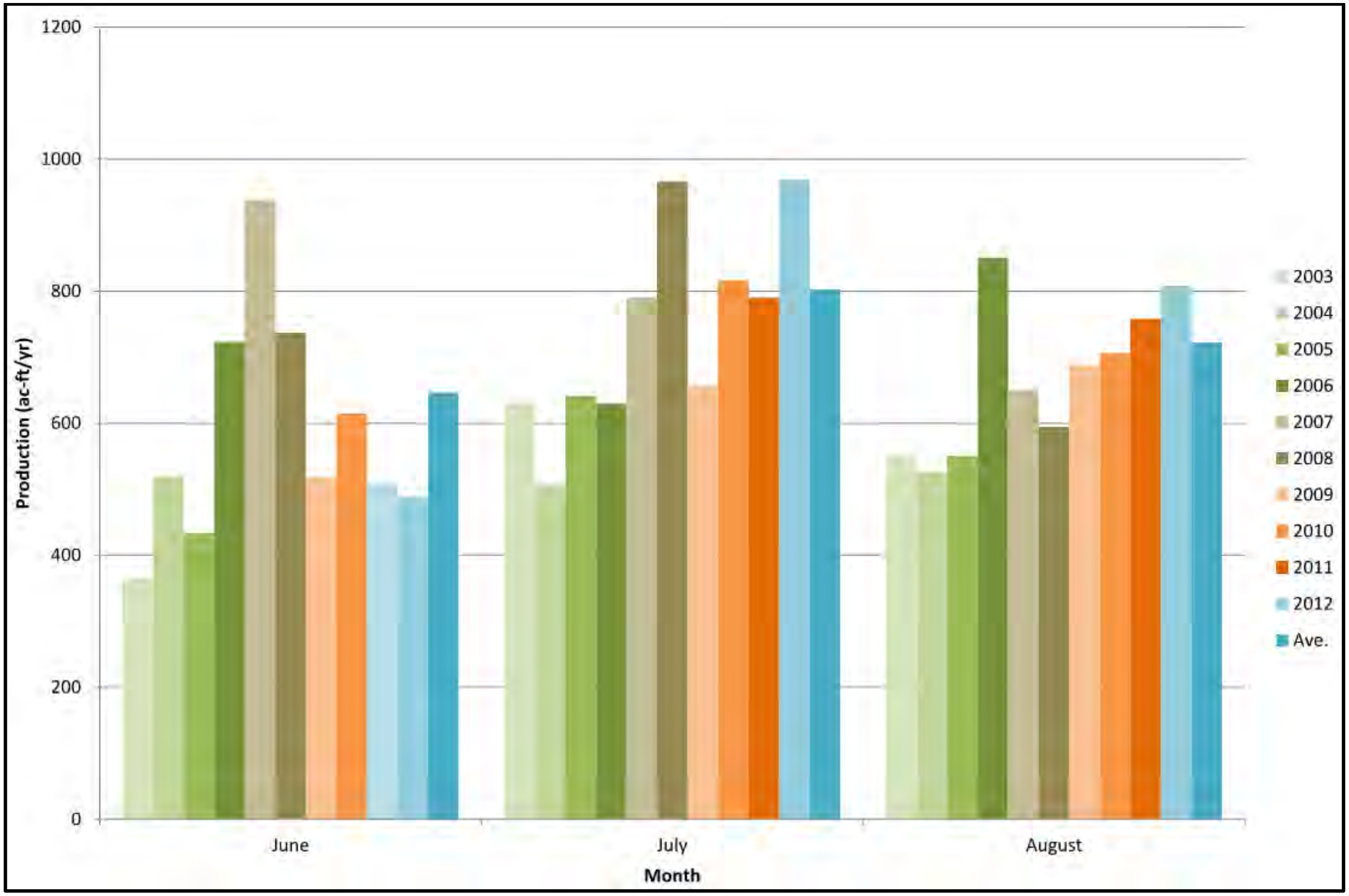


Figure 3-28
Summer Month Production (2003 to 2012)

Drought Operations during 2002

During the spring and summer of 2002, BOPU experienced noticeable difficulty in extracting additional quantities of groundwater from the well fields. Some wells, notably King #5, Elkar #7, Elkar #1, Bell #8, and Merritt #9, the production had to be controlled in order to prevent the water level from dropping too low. The sustainable production from many of the wells was lower than in past years, as shown in Table 3-17. BOPU has noticed that the anticipated "safe yield" at recently replaced wells (i.e., design pump rate) has not been sustainable. These observations have well field operators very concerned about the long-term sustainable yield from the Tertiary Aquifer. In response to the production difficulties in 2002, well field operators lowered 2003 annual production estimates to approximately 3,900 ac-ft.

A comparison of annual and seasonal well field production in 2002 with previous years suggests a slight decline, but does not provide conclusive evidence of a long-term trend. Annual production in 2002 was 4,400 ac-ft, which is noticeably less than the 1994 to 2002 average annual production of 5,100 ac-ft, but is still within the range of annual production values during that period. Annual production in 2002 at the Bell, Happy Jack, and Federal well fields was fairly typical of the previous 3 or 4 years. In 2002, well field production during the summer months, as shown on Figure 3-25 was less than the previous 4 years (1998 to 2001), but is still within the range of production values observed since 1994.

The question is whether the decrease in annual and summer production from 1998 to 2002 indicates a long-term trend, and if so, what might be the primary cause? Since 1998, the annual precipitation at the Hecla Station has been below normal and the cumulative departure from the mean has been increasingly negative since 2000, as shown on Figure 3-26. Poor recharge over the period 1999 to 2002 may have been partially responsible for the Tertiary Aquifer being more sensitive to well field pumping. The dynamic nature of well field operation, well condition/repair/replacement, demands, and aquifer recharge prevents a conclusive analysis, but historic head declines and trends, as discussed in the next section, may indicate future difficulties in maintaining present levels of production.

Historic Impacts to the Tertiary High Plains Aquifer

BOPU well fields have been extracting groundwater from the Tertiary High Plains aquifer for more than 70 years, and there have been noticeable impacts to the aquifer. However, BOPU is not the only user of groundwater in the area surrounding the well fields. Domestic, irrigation, and industrial users are competing for the same resource and a greater awareness of further impact to the resource and associated conflicts is warranted. The following sections will discuss the historic impacts to the Tertiary High Plains aquifer and the response of the aquifer to well field production.

3.4.5 Recent Well Field Production

Within the last five years, groundwater has made up approximately 30 percent of the total water supplied to the treatment plant. This percentage rises drastically during summer peaking months at times supplying beyond 40 percent of total water (Figure 3-29). The percentage of groundwater vs. total water used has been steadily increasing throughout the past 10 years, likely due to the 2002-2010 drought. However, due to 2033 to 2063 water demand projections, it is expected that groundwater will become an increasingly valuable resource.

Currently, on a ten year average, the Happy Jack well field is responsible for 36 percent of the groundwater used, Bell and Borie well fields supply 26 and 27 percent respectively, and the Federal well field supplies approximately 15 percent of the annual average of 4,322 ac-ft/yr (Table 3-18). Since 2002, the total loss in well field production is approximately 700 gpm or 1129 ac-ft/yr (Table 3-17). This is effectively the same as losing three average production wells.

Borie Well Field

The Borie well field is responsible for producing 27 percent of all groundwater for the year. On average, the Borie wells extract approximately 1,200 ac-ft/yr, with the average well capable of pumping at a maximum rate of 387 gpm. This well field has gained 196 gpm production capabilities in the past 20 years, of which 55 percent of that gain occurred in the past 10 years. Specifically, the Weber well has lost over 200 gpm in production capacity in the 1992 through 2012 period. A very small percentage of this was lost in the last 10 years. The Elkar #7 well experienced a 200 gpm gain in production capacity post-rehabilitation, but has been sharply declining in production since 2002, losing 130 gpm in the past ten years. The Borie well field is capable of supplying 1550 gpm during peaking demands. Refer to Table 3-17 for additional production values.

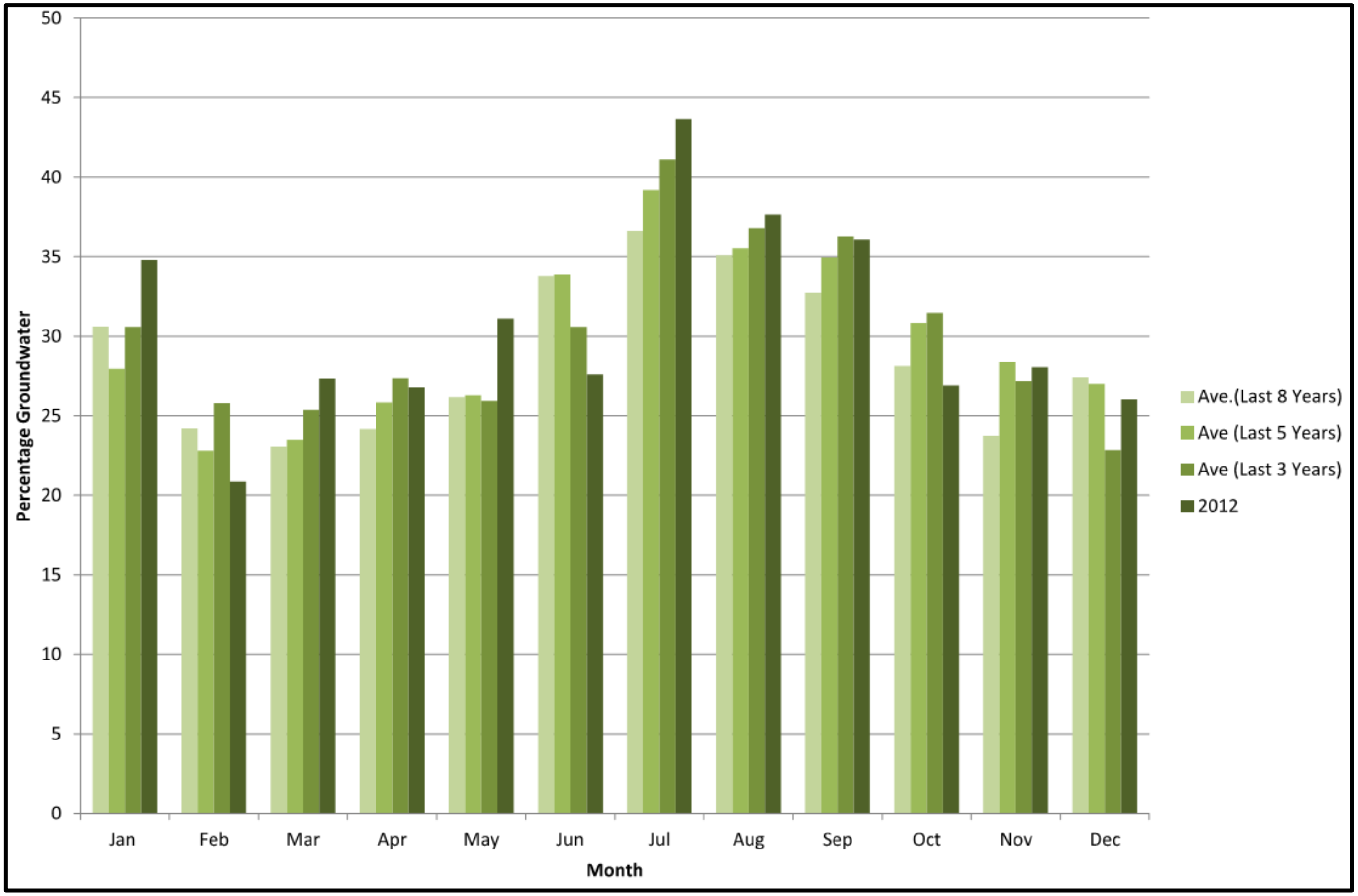


Figure 3-29
Monthly Average Blend Percentage

Federal Well Field

The Federal well field has the majority of its wells drawing from the White River Group of the High Plains aquifer. This well field supplied approximately 11 percent of the total groundwater demand in 2012. Due to the tight nature of the White River Group, wells in this field are only capable of producing an average of 197 gpm. Declining aquifer water levels have been seen throughout the well field. Recovery times post-pumping are on the order of several months. Overall, since 1992, wells in the Federal field have lost a total of 660 gpm in production, either due to well condition deterioration, porosity collapse within the aquifer, or decrease in saturated thickness of the aquifer. The Merritt #5 well lost 200 gpm of production capacity since 2002. Fifty percent of the capacity was lost in the past ten year span. The Merritt #8 well gained 60 gpm of yield in the past ten years. However, it is producing below its maximum original yield, even after replacement. The current yield of the Federal well field during summer peaking season is 1,575 gpm, but the wells will only produce for short durations (Table 3-17).

Bell Well Field

The Bell well field produces water from the Ogallala formation with several wells extending into the underlying White River group. This well field supplies approximately 26 percent of the total groundwater demand. Wells in the Bell well field produce from approximately 150 gpm to nearly 500 gpm with an average well producing 253 gpm. Since 1992, this well field has lost approximately 600 gpm in total production capacity, with 442 gpm of the loss, or 70 percent, occurring in the past ten years. The Bell #5 well gained 56 gpm from 1992 to 2012. During this same period, however, the Bell #11, Bell #12, and Federal/Bell #25 wells lost 215, 97, and 150 gpm of production capacity, respectively. During peaking demands, this well field is capable of producing water at 2,445 gpm. Production values are presented in Table 3-17.

Happy Jack Well Field

The Happy Jack well fields supply 36 percent of the city's total groundwater supply. The total groundwater production has declined by 878 gpm since 1992, with 32 percent of the total loss occurring within the last ten years. The majority of the wells within the Happy Jack well field are screened in the Ogallala and White River formations. Peak production capability of this well field is approximately 3,100 gpm. Average current production rate in this field is 240 gpm. Koppes #1 has seen a small increase in production capabilities since 1992, and the Elkar #5 has lost 254 gpm within the last 20 years, 100 of which were lost within the last ten years. Both wells currently have production rates of up to 400 gpm. (Table 3-17).

Recent Production Trends

The Federal, Bell, and Happy Jack well fields have all experienced production rate declines. However, production rates in the Borie well field have slightly increased over the past 10 year period. Despite significant static water level declines, the Borie well field production rates have increased by 18 gpm. The Federal, Bell, and Happy Jack well fields have seen declines ranging between 70 gpm to 442 gpm. Although this may be due to several factors, noticeable and measureable declines in static water levels have been observed in the four well fields.

3.4.6 Static Water Levels

The tracking and managing of static water levels as they relate to the saturated thickness of the aquifer is critical in maintaining the overall health of an aquifer. Rapid and drastic decreases in static water levels often result in production loss, increased potential for contamination, and decreases in the lateral conductivity of an aquifer.

Declining conditions such as these may occur due to several factors. Because water is generally incompressible, water within the pore spaces of sediments acts as a form of support for the aquifer system. Removal of this water may result in an inability for the aquifer to maintain its porous structure causing subsidence due to porosity collapse. Over-pumping from an aquifer extends the radius of the cone of depression. This extended span of the cone of depression may increase the capture zone, mobilizing any contaminants present within the new flow regime drawing them towards the well. The High Plains aquifer has experienced deterioration of conditions such as these in other areas. For example, Kansas has dried up much of the western half of the state's drainages, devastating riparian and aquatic ecosystems (McGuire, 1997). Parts of New Mexico have also experienced the effects of over-exploitation of groundwater as they measured up to 50 feet of ground surface subsidence (USGS, 2002).

All BOPU wells generally demonstrate continuously changing water levels, declining during pumping and recovering during non-pumping periods. True static conditions are rarely, if ever, observed as recovery generally continues throughout the winter to the start of the next pumping season. Because of this, the highest winter-spring non-pumping water levels have been used to represent static conditions, even though these may not truly be static conditions. The times between these measurement points and the last pumping periods are not consistent, and may account for some variability in the static water levels as discussed in this Master Plan. This is consistent with methodology used in the previous 2003 Water Master Plan.

Head declines have been noted across all four well fields. Observations within each well field show the average total head decline ranges from 29.8 to 81.8 feet (Table 3-19). Data shows that from the period of well field construction in the 1940's up to 1992, the average total head decline ranged from 12 to 37feet in the Bell and Borie well fields. Since 1992, the rate of head decline has increased drastically. Within the ten year span of 1992-2002, the Borie well field lost an additional average of 7.5 feet, the Federal well field lost an average of 11 feet, the Bell well field lost an average of 13.5 feet, and the Happy Jack field declined an average of 27. The Merritt #5, Bell #16, Koppes #2, Koppes #6, King #2, Elkar #5, and King #1 wells saw the most significant head declines of 30, 28, 42, 34, 44, 48, and 51 feet, respectively, within the 1992 to 2002 period (Table 3-19). Between 1992 and 2002, the Weber #1, Bell #25, and Eddy #2 were the only wells that did not experience an overall water level decline. Precipitation during this ten year period was relatively average in comparison to the past 50 years.

Area-wide head declines in the Ogallala aquifer are primarily responsible for the reduction or cessation of flowing wells. Examples of past flowing wells include Merritt #8, Weber #1, Bell #5, Bell #6, and Bell #11. Currently, none of these wells are flowing; most wells have noticed a decrease in service water level of 20+ feet (2003 Master Plan).

Current Static Water Levels

Static water level conditions have continued to decline through 2012. Within the past 10 year period, the well fields have experienced a loss of 14 to 25 feet. Although some of this is a result of the recent dry period and reduced recharge, the over pumping of groundwater has become evident. Average well field declines are presented in Figure 3-30. The static water level within the Happy Jack well field declined by an average of 25 ft in head from 2002 to 2012, with a cumulative decline since the 1940's of 82 ft. Within the last 10 years, the Borie well field has experienced 30.6 percent of its total head decline. Hydrographs for the Eddy #2, Bailey #5, Koppes #2, and King #5 are presented in Appendix 3-B. These wells have all experienced substantial losses of saturated thickness. During this period, the Happy Jack well field was pumped at approximately 1576 ac-ft/yr, at rates averaging 240 gpm.

The Federal well field experienced an average static water level decline of 17 ft during the 2002-2012 period. The average total decline within the wells of the Federal field since inception is 38 feet. Of this decline, 45.5 percent occurred during the 2002 to 2010 period. Head decline varied greatly within the field, ranging from a loss of 3 feet in the Merritt #9 well to a loss of 86 feet in the Merritt #14 well. The large overall decline observed in the Federal well field is likely due to the lower overall hydraulic conductivity of the White River Group, as demonstrated by the lower than average pumping rates, as well as the lengthy recovery times.

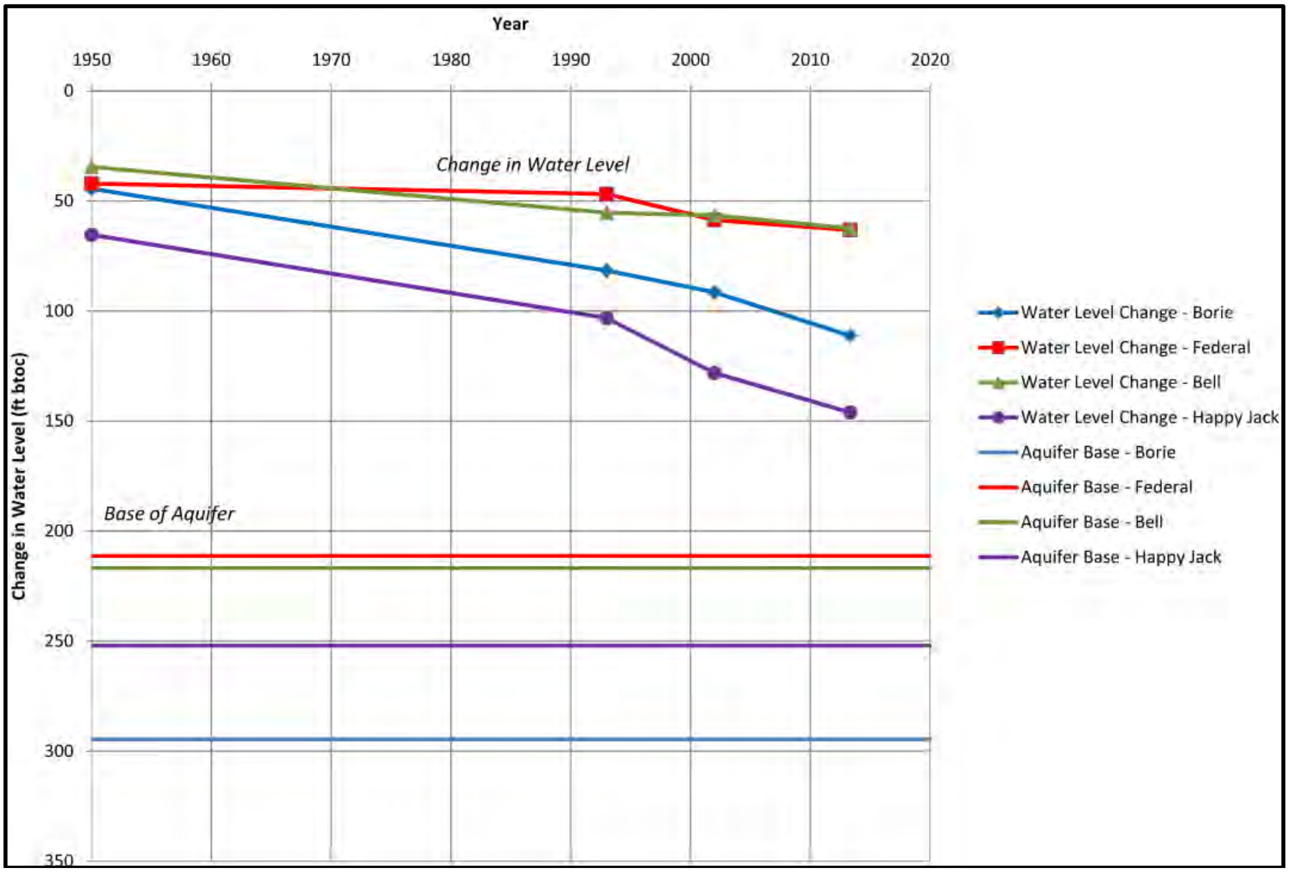


Figure 3-30
Water Level Changes by Well Field

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Table 3-19. Distance to Groundwater

Well Name	Earliest DTW [1] (ft)	1993 DTW [1] (ft)	2002 DTW [2] (ft)	2012 DTW [3] (ft)	1993 to 2002 Head Decline (ft)	2002 to 2012 Head Decline (ft)	Total Historic Head Decline (to 2012) (ft)
Borie Well Field							
Borie #1	98 (1947)	129	139	206	-10	-67	-108
Elkar #7	85 (1947)	110	119	181	-9	-62	-96
Finnerty #2	45 (1947)	72	N.D.	135	N.D.	N.D.	-90
Weber #1	43+ (1947)	34	18	30	16	-12	13
USGS: Borie	37 (1945)	63	90	N.D.	-27	90	N.D.
Federal Well Field							
Merritt #5	30 (1942)	48	78	89	-30	-11	-59
Merritt #6	86 (1944)	N.D.	82	99	N.D.	-17	-13
Tax #1	147 (1944)	Plugged	Plugged	Plugged	Plugged	Plugged	Plugged
Merritt #1	73 (1942)	104	112 (2001)	N.D.	-8	N.D.	N.D.
State #2	23 (1954)	N.D.	60	75	N.D.	-15	-52
Merritt #14	Flowing	22	26	101	-4	-75	>-101
State #1	16 (1954)	72	N.D.	67	N.D.	N.D.	-51
Merritt #15	30 (1954)	65	74	41	-9	33	-11
Merritt #8	Flowing	Flowing	15	29	>-15	-14	>-29
Merritt #9	16 (1954)	17	22	32	-5	-10	-16
Bell Well Field							
Bell #5	0	N.D.	10	7 (2011)	N.D.	3	-7
Bell #6	Flowing	N.D.	N.D.	13 (2011)	N.D.	N.D.	>-13
Bell #8	35	N.D.	70	179 (2011)	N.D.	-109	-144
Bell #10	40	35	N.D.	46 (2011)	N.D.	N.D.	-6
Bell #11	Flowing	Flowing	21	37 (2011)	>-21	-16	>-37
Bell #12	82	69	N.D.	91 (2011)	N.D.	N.D.	-9
Bell/Fed #16	26	76	104	46 (2011)	-28	58	-20
Bell #17	42	61	N.D.	76 (2011)	N.D.	N.D.	-34
Bell/Fed #25	88	109	104	161	5	-57	-73
Bell/Fed #24	53	73	N.D.	N.D.	N.D.	N.D.	N.D.

Well Name	Earliest DTW (1) (ft)	1993 DTW (1) (ft)	2002 DTW (2) (ft)	2012 DTW (3) (ft)	1993 to 2002 Head Decline (ft)	2002 to 2012 Head Decline (ft)	Total Historic Head Decline (to 2012) (ft)
USGS: Bell #14	13 (1957)	20	30	N.D.	-10	N.D.	N.D.
Happy Jack Well Field - North							
Holman #1	17 (1942)	29	42	120	-13	-78	-103
Eddy #2	27 (1941)	73	73	199	0	-126	-172
Elkar #1	29 (1947)	N.D.	51	127	N.D.	-76	-98
Bailey #5	12 (1940)	55	78	84	-23	-6	-72
Bailey #1	6 (1947)	26	Plugged	N.D.	N.D.	N.D.	N.D.
Koppes #1	19 (1940)	N.D.	106	86	N.D.	20	-67
Koppes #2	30 (1940)	80(3)	122	161	-42	-39	-131
Koppes #6	123 (1947)	175	209	208	-34	1	-85
Happy Jack Well Field - South							
Happy Jack #1	21 (1941)	36	Plugged	N.D.	N.D.	N.D.	N.D.
Happy Jack #2	22 (1941)	N.D.	Plugged	N.D.	N.D.	N.D.	N.D.
Happy Jack #3	14 (1941)	N.D.	54	82	N.D.	-28	-68
King #4	77 (1947)	106	122	135	-16	-13	-58
Koppes #4	85 (1947)	121	134	161	-13	-27	-76
Koppes #3	70 (1947)	N.D.	128	139	N.D.	-11	-69
King #2	76 (1947)	106	>150	N.D.	>-44	N.D.	N.D.
Conrey #1	145 (1947)	188	214	219	-26	-5	-74
Koppes #5	140 (1947)	Plugged	Plugged	Plugged	Plugged	Plugged	Plugged
Elkar #5	101 (1947)	102	150	205	-48	-55	-104
King #1	118 (1947)	145	196	226	-51	-30	-108
King #5	173 (1947)	213	242	280	-29	-38	-107
USGS: King #3	67 (1946)	93	107	N.D.	-14	N.D.	N.D.

(1) Data from 1994 Water Master Plan.

(2) Data from 2003 Water Master Plan

(3) Well yield values from BOPU

N.A. = Not Applicable

N.R. = Not Replaced

Declines within the Bell well field range from a gain of 60 feet in Bell #16 to a loss of 108 feet in Bell #8. The average total decline in the Bell well field is 29.8 ft, with 48.1 percent of the total decline occurring within the last ten years. Average production at the Bell field is approximately 1,100 ac-ft/yr, supplying almost 26 percent of the City's total groundwater volume. Pumping rates in Bell #8 have declined by 65 gpm since 2002, where pumping rates in Bell #16 have increased by 20 gpm to a total of 320 gpm.

Static water levels in the Borie well field have dropped significantly throughout the life of the well field. Static water level decline in the Borie field has averaged 67 feet throughout the history of the Borie well field, approximately 16 ft of the average total static water level decline occurred in the past 10 years. The Borie #1 and Elkar #7 wells have experienced decline in static water level of 31 and 33 feet respectively; where the Weber #1 well experienced an increase of 5 feet. The Borie well field produced at the approximate rate of 1,200 ac-ft/yr during the 2002 to 2012 period, supplying 27 percent of the City's ground water.

3.4.7 Estimates of Sustainable Production from BOPU Well fields

A reliable estimate of long-term sustainable production from BOPU well fields is a critical component to future water supply planning, as well as the maintenance and overall preservation of current groundwater resources. Past efforts by Lowry and Crist (1967), Centrac (1982), and Ertec (1984) used groundwater modeling and recharge estimates from the High Plains Aquifer system to estimate the maximum sustainable production from all well fields as listed in Table 3-20. The 1994 Water Master Plan provides a detailed description of the methodology, analysis, and limitations of the previous estimations.

Table 3-20
Estimated Sustainable Yield for Tertiary High Plains Aquifer

Estimated Sustainable Yield (ac-ft/yr)	Source	Comments
4,900	Lowry and Crist (1967)	Additional 50 ft of head decline
5,290	Centrac (1982)	--
5,500 to 7,000	Ertec (1984)	--

Well field production limits established by the SEO (i.e., an annual average production of 5,500 ac-ft/yr over a 10 year period) appears to reflect the maximum sustainable production estimates derived from these reports, but historic evidence dictates 5,500 ac-ft/yr is not sustainable over several production years. The 1994 and 2003 Water Master Plans provided updated estimates of sustainable production, as highlighted in Table 3-21 and Table 3-22, respectively. Methodology, analyses, and parameters can be found in each respective report.

Table 3-21
1994 Estimated Sustainable Production

Well Field	Estimated Sustainable Production (ac-ft/yr)
Bell	1,700
Borie	400 (?)
Federal	600
Happy Jack	2,100
Total	**4,800**

Table 3-22
2002 Water Master Plan Estimated Sustainable Production

Well Field	Estimated Sustainable Production (ac-ft/yr)
Bell	1,600
Borie	400
Federal	500
Happy Jack	1,600
Total	**4,100**

Sustainable yield approximations have been estimated based on the projections of past investigations, volumetric contributions of each well field within the past 10 years, hydraulic responses seen within each well field, and information from BOPU Well Field Lead Operator. Based on these sustainability estimates and static water level data, revised production volumes for each well field have been generated, and are presented in Table 3-23.

Table 3-23
Revised Estimated Sustainable Production

Well Field	Estimated Sustainable Production (ac-ft/yr)
Bell	1,100
Borie	1,000
Federal	500
Happy Jack	1,400
Total	**4,000**

Annual average production from 1994 through 2002 was 5,100 ac-ft/yr, exceeding the 4,800 ac-ft recommended by the 1994 Water Master Plan (Table 3-21). Observation wells across all four well fields recorded an average of 15 to 20 feet of static water level drop during this period. This drop was not surprising, based on the total sustainable production estimate of 4,100 ac-ft in the 2003 Water Master Plan. Throughout the past ten years, total annual average production was approximately 4,322 ac-ft/yr. Static water levels in the well fields dropped by a range of 14 to 25 feet. Based on comparing static water levels and past sustainable production estimates (Table 3-21, Table 3-22, and Table 3-23), the total sustainable production of the four well fields has been estimated to be 4,000 ac-ft/yr, for a year with average precipitation.

These well field production estimates should be viewed as a general guideline and should be updated periodically in a manner similar to that described in the Drought Monitoring Plan (Appendix 3-B). Monitoring withdrawals and water levels for all nearby major non-municipal users may be necessary to properly assess changes within the Tertiary High Plains aquifer system.

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3.5 Future Well Field Options

There are several available options for new well fields. These include, but are not limited to:

1. The Belvoir Ranch
2. New well fields to the north or northeast of the City
3. Dyno-Nobel Well field
4. The Polo Ranch
5. Deeper Lance/Fox Hills and Casper Aquifers

These options are discussed in the following sections.

The amount of water produced from a single well varies widely. In the four existing City well fields, the average production rate of all High Plains aquifer working wells is 250 gpm, with a minimum of 50 gpm (not counting non-producing wells) and a maximum of 515 gpm. For planning purposes, an average new well can be assumed to produce 250 gpm. Assuming the average well runs continuously for approximately 6 months, the average well will produce 200 ac-ft/yr.

As new well fields are planned, these numbers may be used as a preliminary estimate of the number of wells that will be required. For example, to obtain 2,000 ac-ft/yr from a new well field, the preliminary estimate would be that 10 wells would be required. As better site-specific information is obtained, these estimates should be refined.

3.5.1 Belvoir Ranch

The Belvoir Ranch was purchased by the City of Cheyenne in 2003 for the possible expansion of the City's water supply, along with other uses. The water resources available on the ranch include limited surface diversions from Lone Tree Creek, groundwater from the High Plains Aquifer, and groundwater from deeper formations such as the Casper and Lance/Fox Hills Formations. There are several High Plains aquifer wells on the Belvoir Ranch that were in existence at the time of purchase by the City that are used along with surface water for stock watering and hay irrigation. The limited surface water on the Belvoir Ranch is primarily ephemeral and subject to existing senior water rights, so it is not a good candidate for municipal supply. The groundwater presents an opportunity for development, but it also has some issues associated with senior water rights that must be overcome through the permitting process.

Several studies funded by the City and by the Wyoming Water Development Commission (WWDC) have examined the groundwater resources within this property. The first three reports in this list focused on the High Plains Aquifer. The last report focused on the deeper Casper Formation aquifer.

These studies include:

- Cheyenne Belvoir Ranch Level II Study, 2005
- Cheyenne Belvoir Ranch Level II Study, Phase III-IV, 2007
- Belvoir Ranch High Plains Aquifer – White River Study, 2007
- Cheyenne Belvoir Ranch Groundwater Level II Study, 2012

The High Plains aquifer in Laramie County generally consists of three formations, the Ogallala, the Arikaree, and the White River Group. In the vicinity of the Belvoir Ranch, the Arikaree is not present, and the Ogallala lies directly on top of the White River Group. The Ogallala formation has an average thickness of 280 feet across the property, but it is only saturated on the eastern portion of the ranch property. As part of the above studies, numerous monitor wells were installed in the Ogallala and White River, along with two Ogallala test wells, the Belvoir #5 and #6. The Belvoir #5 well was constructed in 2005 and was shown to be capable of producing 500 to 700 gpm. The Belvoir #6 well was constructed in 2007 and was shown to be capable of producing 300 to 400 gpm. Both of these wells are available to be completed as municipal supply wells. Costs to complete these two wells and piping to the Sherard WTP, with excess capacity for existing Borie field wells were estimated to be about $7.0 millon (2007 dollars).

A portion of the eastern Belvoir Ranch was identified in the 2005 WWDC report as a potential well field. After the 2005 study, the City filed nineteen well permits for potential sites. Some or all of these sites may be appropriate, but a well field optimization study should be conducted to determine the best number of wells that can be constructed and their locations within the identified well field area.

TCE Issues

An abandoned Atlas missile site lies on the northwestern part of the Belvoir Ranch. This was the source of a trichloroethylene (TCE) contamination plume that extends eastward across much of the ranch. Extensive analysis of this TCE plume has been conducted by the Wyoming DEQ and the US Army Corps of Engineers (USACE). Water produced from the High Plains aquifer on the Belvoir ranch either contains TCE or has the potential to contain TCE in the future. The USACE has constructed a TCE removal facility which currently treats water from these wells and has current capacity to treat up to 3,000 gpm. Ultimately, the treatment facility may be expanded to treat 8,000 gpm.

Casper Aquifer

The Paleozoic Casper formation was explored in 2005-2006 by a WWDC funded project. Three test wells were constructed: the Lone Tree #1, the Duck Creek #1, and the Kennedy #2. Results were mixed, as the Lone Tree #1 produced over 700 gpm while the Duck Creek #1 produced only 24 gpm. Four more test wells were installed during an additional study from 2010-2012. These wells yielded water production that ranged from 30 to 200 gpm.

The study recommended that two Casper formation well fields be developed on the western part of the Belvoir Ranch, the Lone Tree Creek well field and the Duck Creek well field.

The Lone Tree Creek well field would consist of seven wells. With four of these wells running at any given time, the field was estimated to produce 2,000 gpm for an annual total of 1,060 acre-feet over a 120 day summer season. The Duck Creek well field would consist of six wells. With three of these wells running simultaneously, the well field was estimated to yield 600 gpm, for a total of 318 acre-feet over a 120 day summer season. The study estimated that the exploration and completion costs for the Lone Tree Creek well field, including transmission pipelines back to the Sherard WTP, would be approximately $22.8 million (2012 dollars). Exploration and completion costs for the Duck Creek well field were estimated to be $17 million.

Water Rights

Cheyenne holds both surface and ground water rights on the Belvoir Ranch. These rights have historically been used for stock and irrigation purposes at the ranch. Currently, the SEO administers surface and ground water in the Lone Tree Creek drainage as “one source of supply” using provisions of Section 41-3-916, Wyoming Statutes, due to the complex nature of the ground water and surface water interaction. According to the 2003 Water Master Plan, Cheyenne has surface and ground water rights of approximately 7.29 cubic feet per second that are tied to the irrigation of 510 acres.

Table 3-24 summarizes groundwater rights and existing data for the 16 water wells on the Belvoir Ranch that were existing when the City purchased the property. The wells are permitted for stock and/or irrigation use and many of the wells have had flow meters installed since the property was purchased. Production records have been submitted annually to the SEO. With the exception of the Enlarged Borie #1[a] that was relocated and recompleted in 1995, all of the wells were installed from 1931 to 1966. The four high yield irrigation wells have a total adjudicated water right of 2,430 gpm.

Water that is diverted from the Cheyenne-adjudicated irrigation wells and then accretes into the Lone Tree Creek by return flow is first used by the Belvoir Ranch and ultimately shared and diverted east of the ranch by other senior water users, more notably the Rex Dolan property, Duck Creek Grazing Association and Terry Grazing (Terry Bison Ranch). Current diversions are on a percentage basis.

[a] This Ranch well has the same name as, but is different from, the Borie #1 municipal well.

During any future contemplated transfer of the water rights from the Belvoir irrigation wells to municipal use, approximately half of the water right will be lost due to historical non-consumptive use. The 10 low yield stock wells have a total unadjudicated water right of 180 gpm. The two high to moderate yield stock wells (Kennedy #1 and Hall #3) have a total adjudicated water right of 365 gpm.

Cheyenne has options with regard to these water rights and future development of the property. Following the exploration and well development performed during the Cheyenne Belvoir Ranch Level II Studies from 2004-2008, Cheyenne filed permit applications for multiple High Plains aquifer well locations. Approved permits were obtained for two High Plains aquifer wells, the Belvoir Ranch No.5 (U.W. 189075) and Belvoir Ranch No. 6 (U.W. 189077). These well permits (and future contemplated permits) have conditions and limitations as follows:

- Production is limited to either the Lance or the High Plains aquifer.
- Written consent is required from the SEO prior to production.
- Flow meters are required to accurately measure total quantities produced from a well.
- Continuously recording stream gaging devices are required above and below any point on Lone tree Creek where water from BOPU flowing wells enters the Creek.
- Install a monitoring well in the alluvium, in the immediate vicinity and below the point where water from any of BOPU flowing wells enters Lone Tree Creek.
- Installation of nested monitoring wells between Lone Tree Creek and the Belvoir 5&6. Nested wells will be completed, one in the alluvium and one in the High Plains Aquifer.
- All continuous stream gaging data will be reported in the form of an annual report.
- Monitor well data will be collected monthly and submitted as an annual report.
- The annual production from the Belvoir 5 & 6 will be limited to that allowed under BOPU existing volumetric cap for water produced from the High Plains aquifer system.

These terms and conditions will likely aid in the resolution of potential interference claims with senior downstream users.

Table 3-24
Belvoir Ranch Groundwater Rights

Well Name	Permit #	Location (TSR)	Install Date	Priority Date	Use	Permit Status	Water Right (gpm)
Belvoir #1	SC71	13, 68, 22, NESW	7/31/2013	7/31/2013	IRR, ST	ADJ	900
1st Enl. Belvoir #1	U.W.86252	13, 68, 22, NESW	n/a	2/1/1987	IRR, ST	ADJ	275
Belvoir No.1	U.W. 4908	13, 68, 22, NESW	n/a	5/1/1944	ST	CAN	15
Belvoir #2	SC 72	13, 68, 22, SENW	6/1/1933	6/1/1933	IRR, ST	ADJ	120
1st Enl. Belvoir #2	U.W.86251	13, 68, 22, SENW	n/a	2/1/1987	IRR, ST	ADJ	405
Belvoir #2	U.W.4909	13, 68, 22, SENW	n/a	5/1/1944	ST	CAN	10
Belvoir #3	U.W.4910	13, 68, 22, SESE	n/a	6/1/1944	ST	UNA	10.5
Borie #1	U.W.41873	13, 68, 23, SESW	8/1/1977	2/1/1978	ST	ADJ	25
---	aka 36351	" "	n/a	n/a	n/a	CAN	n/a
1st Enl. Borie #1	U.W.86253	13, 68, 23, SESW	n/a	2/1/1987	IRR	ADJ	450
2nd Enl. Borie #1	test hole	13, 68, 23, SESW	5/1/1995	n/a	n/a	ABA	n/a
---	U.W.101720	13, 68, 23, SESW	11/1/1995	2/1/1996	IRR	ADJ	75
Johnson #1	U.W.94459	13, 68, 23, NWNW	5/1/1960	1/1/1994	ST	UNA	20
Belvoir #3	SC 73	13, 68, 25, SWNW	5/1/1934	5/1/1934	IRR, ST	ADJ	120
Enl. Belvoir #3	U.W.101719	13, 68, 25, SWNW	n/a	10/1/1995	IRR	UNA	60
Belvoir #4	U.W.4911	13, 68, 25, NWNW	6/1/1944	6/1/1944	ST	UNA	15
Kennedy #1	U.W.4912	13, 69, 16, NWSW	2/1/1959	5/1/1961	ST	ADJ	15
(aka Site D #1)	SC 77	" "	n/a	n/a	IND	CAN	n/a
1st Enl. Kennedy #1	U.W.100917	13, 69, 16, NWSW	n/a	10/1/1995	MIS	ADJ	125

Well Name	Permit #	Location (TSR)	Install Date	Priority Date	Use	Permit Status	Water Right (gpm)
2nd Enl. Kennedy #1	U.W.101774	13, 69, 16, NWSW	n/a	3/1/1996	MIS	ADJ	185
Hall #3	U.W.4914	13, 69, 16, NWSW	Prior '60	5/1/1961	ST	ADJ	25
(aka Site D #2)	SC 322	" "	n/a	n/a	IND	CAN	n/a
Enl. Hall #3	U.W.100916	13, 69, 16, NWSW	n/a	10/1/1995	MIS	ADJ	15
Site D #4	U.W.14196	13, 69, 17, NESE	4/1/1960	4/1/1960	ST	UNA	20
---	SC 323	" "	n/a	n/a	n/a	CAN	n/a
Kerbs #2	U.W.4913	13, 69, 17, NESE	5/1/1961	5/1/1961	ST	UNA	25
Peterson #1	U.W.94460	13, 68, 27, NENE	5/1/1960	1/1/1994	ST	UNA	20
Winter #1	U.W.5810	13, 68, 27, NESW	5/1/1966	5/1/1966	ST	UNA	12
Willadsen #1	WR461	13, 69, 30, NESW	n/a	7/1/1956	ST, IRR	UNA	40
Willadsen #2	U.W.9105	13, 69, 13, SWSE	n/a	12/1890	ST	UNA	5
Cow Camp #1	U.W.94458	13, 69, 24, SESE	5/1/1955	1/1/1994	ST	UNA	15

3.5.2 New Well Field Opportunities North or Northeast of City

New well field development outside of the existing City water rights or within the Laramie County Groundwater Control Area (LCGCA) may pose several issues. Issues the City may want to consider prior to the initiation of a development program mainly relate to the costs involved in land acquisition, current senior water rights holders, potential interference claims with down gradient users, geochemical in-compatibility related to the current blending requirements, cost of developing a new pipeline infrastructure to the proposed locations, and the overall volume of water available.

This section will review resource potential as it pertains to groundwater reservoir storage, saturated thickness, and the expected achievable production rates within each identified location. Regional data gathered from the 2008 Water Resource Atlas of Laramie County Wyoming was used to locate specific areas containing the highest potential for long term sustainability and above average production capacity. Two locations have been identified fitting these criteria to the north and northeast of the City.

Based on the above criteria, it has been determined that new well fields to the south and east of the City would not be economically feasible to pursue. Existing wells in these areas have been historically low producers, exhibiting water level declines over time, and slow water level recovery once pumping has ceased.

These locations will be suitable assuming the following conditions are met. Further data will be necessary to determine if these conditions are true.

- Treatment Plant capacity is sufficient, or will be sufficient to handle any new additional groundwater production.
- Pipelines will be constructed to manage and transport water to new or existing treatment facilities.
- Ground water geochemistry within the identified regions will not affect the current blending procedures.
- Water rights are available for purchase, land acquisition and development can/will occur.
- Groundwater will likely make up a larger percentage of supply water in the future.

North of Cheyenne

A location with good saturated thickness and high transmissivity has been identified from information in the Water Resource Atlas of Laramie County Wyoming (2008). This site is approximately 13 miles north of the Round Top Water Treatment Plant, on State-owned land just west of the intersection at Old Yellowstone Road (WY 219) and I-25. BOPU Lead Well Field Operator independently identified this as an area of interest for a new well field. Notable advantages of this location are: the proximity to infrastructure and major transportation corridors, potential cost savings in pipeline development if connected to the existing Federal well field pipeline infrastructure, and high ground water production capacities recently recorded in wells operating in this area. This site is near the western edge of the LCGCA. Permitting will be faster and easier if a suitable site can be located west of this boundary.

The Water Resource Atlas of Laramie County Wyoming (2008) Figure 6-5 shows wells in this area completed in the Ogallala aquifer, located along Lodgepole Creek approximately three miles west of I-25. The Ogallala Formation in this region is estimated to be 300 to 400 feet in thickness, with hydraulic conductivity values ranging from 50 to 100 gpd/ft^2. Saturated thickness of the High Plains Aquifer at this location ranges from 600 to 800 feet. Specific yield in the area is similar to the City's current well fields ranging from 5 to 10 percent. Water quality analyses will need to occur prior to incorporating water from this area into the current blending procedures.

Estimated Yields

Assuming that a realistic well production rate in this area averages 300 gpm per well, continuous pumping of each well would provide approximately 1.32 ac-ft/day of groundwater. Continuous pumping for six months at this rate would provide about 240 ac-ft.

Northeast of Cheyenne

Two miles northeast of County Road 215 and County Road 136, approximately 17 miles from Round Top Water Treatment Plant, lies a 400-600 foot saturated thick section of the High Plains aquifer system. These aquifer sections generally increase in thickness and yield as one continues further northeast towards Albin. Characteristics of the Ogallala in this area include a saturated thickness up to 300 feet encompassing most of the vertical extent of the aquifer. Potential for yields in this area range from 100 to 500 gpm. However, little exploration has been performed in this area outside of the Cheyenne region. Studies have indicated specific yields of 15 to 25 percent within this area with transmissivities ranging between 50-100 gpd/ft^2.

Expected Yields

Production in this area is likely going to be higher than 300 gpm on average. Transmissivity values are high in this area, so post-pumping recovery of wells will occur very quickly. There is an abundance of water in this area, where an average well can be expected to yield 1.5 to 2.2 ac-ft/day. At this rate, 3 to 4 wells per 1,000 ac-ft/yr will be required.

3.5.3 Dyno-Nobel Well Field Resource

Dyno-Nobel owns and operates about ten High Plains aquifer wells in a well field east of the City's Borie well field. These wells have high production capacities, making them suitable for a municipal supply. The ten wells have a combined production rate of about 2,800 gpm, with individual well rates ranging from 200 to 400 gpm. The City has had brief discussions in the past with Dyno-Nobel about scenarios that would allow the City to utilize this well field for supply. These discussions included the following two options:

1. Exchange City raw surface water for groundwater from the Dyno-Nobel field.
2. Negotiate an agreement that would allow the City to operate the well field and send groundwater to the Sherard WTP in exchange for providing either raw or treated water to the Dyno-Nobel chemical plant.

At first glance, a water exchange as in Option 1 may not seem beneficial. The City currently has to blend groundwater in with surface water to satisfy water quality requirements for treatment plant input. By replacing some surface water with Dyno-Nobel groundwater, the City's well field groundwater demand as a percentage of supply would be reduced. This could result in some energy savings with less well pumping, and this could provide some relief to the City's well fields and the nearby portions of the High Plains Aquifer, possibly extending the life of some wells and well fields.

The second option would give the City control of the Dyno-Nobel well field, which would provide the City with the following advantages:

- Ability to expand well production in the Belvoir Ranch/Borie Field area without Dyno-Nobel interference issues.
- The City would be able to utilize any excess production capacity in the Dyno-Nobel wells beyond what is required for plant operation.
- Greater flexibility with overall groundwater supply; ability to rest portions of the City's existing well fields as the Dyno-Nobel fields contribute water.
- Better control over injected water if ASR is used in the future, particularly if ASR is ever used in the Belvoir Ranch/Borie Field area.
- Increased drought protection with increased groundwater supply.

Dyno-Nobel has likely seen declines in water levels and production from its wells, and would gain the advantage of a more sustainable water supply, as well as the reduced costs from not operating or maintaining the well field. Certainly there are issues that need to be addressed, such as what the City's obligations would be if the Dyno-Nobel plant were to expand, or if water quality requirements for the plant ever changed.

The City should re-engage Dyno-Nobel in discussions that explore these options further. The wells are a proven resource and their proximity to the Sherard WTP would likely make this option economically favorable. If this option will not work for whatever reason, it should be ruled out prior to any additional pipeline design in the Belvoir Ranch/Borie Field area, as it could make a difference in design capacities of any pipelines to the Sherard WTP.

3.5.4 Expansion of Bell Well Field

The Bell well field is located on property known as the Polo Ranch. As discussed in a previous section, the City entered into an agreement with the Polo Ranch property owners in 1955 that allows the City to construct and operate wells on the property. There is additional Polo Ranch property west and north of the current Bell field that is potentially available for additional wells.

This option can be pursued in the near term, since the land subject to exploration is already under an access agreement with the City. The city should initiate a test well program that explores the remainder of the available Polo Ranch property, followed by the installation of municipal supply wells in areas that are found to be capable of good water production. Eight to ten test wells would give good coverage of the remaining Polo Ranch property that is outside of the existing Bell well field. Depending of results of the test well program, the City could likely install up to four new municipal wells.

3.5.5 Deeper Lance/Fox Hills Aquifer

The Lance Formation and the Fox Hills Sandstone are upper Cretaceous-age units that underlie the Tertiary High Plains aquifer. The Fox Hills Formation is gray to white to yellowish brown friable sandstone interbedded with dark sandy shale. The formation ranges from approximately 40 feet to over 250 feet thick in Laramie County. The Fox Hills formation is not present in the Federal well field. In the other three City well fields and the Belvoir Ranch, the Fox Hills has a thickness of 150 to 200 feet, with a depth to the top of the Fox Hills of 1,200 to 1,800 feet.

The Lance formation overlies the Fox Hills formation. The Lance Formation is composed of interbedded tan and gray sandstone, siltstone, gray shale, black carbonaceous shale and thin coal beds. The Lance formation may be as much as 1,500 feet thick in the western part of Laramie County. These deeper sandstone units have been proposed as potential targets for groundwater development, but very little data is available to assess actual production and water quality. Although currently there is not much use of the Lance and Fox Hills aquifers in Laramie County, these formations are extensively used in the Denver area. The town of Pine Bluffs in Eastern Laramie County has a municipal well successfully drawing water from the Fox Hills aquifer system at 205 gpm.

Lance Fox Hills Aquifer Development History

During the Cheyenne Well Replacement and Rehabilitation Program, which focused primarily on well completions in the Tertiary High Plains aquifer, BOPU also installed wells to evaluate the Lance Aquifer. Three replacement wells were designed to evaluate the aquifer characteristics of the Lance Formation: Conrey #1, Holman #1, and Koppes #3 Deep (Weston 1996, 1999). At the south end of the Happy Jack Well field, the top of the Lance Formation occurs at approximately 550 feet and may exceed 700 feet in thickness.

In all three wells, estimated yields from the Lance Formation sandstones were less than 30 gpm despite substantial footages of unit penetration and screen installation. Low yields and hydraulic interference with the overlying Tertiary High Plains aquifer required the plugging and abandonment of the lower portions of the Conrey #1 and Holman #1 wells that were completed in the Lance Formation. The Koppes #3 Deep well was not completed as a production well and is now used only as a monitoring well in the Lance Formation. Based on the results of well testing, the Lance Formation in the vicinity of the Happy Jack Well field is not productive and is not a viable target for future municipal water supply development.

The Merritt #9 is the only City municipal well completed in the Fox Hills Sandstone. Based on the geophysical log, Weston (2000a) identified approximately 90 ft of Fox Hills Sandstone and installed 81 ft of screen adjacent to the unit. The well currently produces approximately 250 gpm. However, the actual production from the Fox Hills Sandstone cannot be determined because 81 ft of screen was installed in the overlying White River Group.

To summarize, the Lance Formation in the vicinity of the City well fields does not have suitable production capacity for municipal development. The Fox Hills Sandstone has not been evaluated in detail, but given its depth of over 1,000 ft in the vicinity of the Happy Jack Well field, and comments by previous investigators, the Fox Hills Sandstone has previously not been considered a viable target for future municipal development other than at the eastern edge of the Federal Well field.

Recommendations

The Fox Hills aquifer has been underutilized in the past due to the depth and cost of drilling. It does present an opportunity, however, and the City should consider exploration projects near the existing well fields and Belvoir Ranch property. Exploration in other areas such as the areas mentioned north and northeast of the City should include deep test hole drilling to explore the Lance/Fox Hills potential along with that of the High Plains aquifer.

3.5.6 Recommendations for New Groundwater Supplies

Over the coming years, all of the discussed options for new well fields may be needed. Based on available water, land ownership and proximity to existing infrastructure (and inferred lower costs), the options for new well fields are ranked as follows:

1. Expand the existing Bell well field on the Polo Ranch. Explore the Lance/Fox Hills aquifer along with the High Plains aquifer.
2. Explore the Dyno-Nobel well field options. Knowing the outcome sooner rather than later will aid in planning other well fields and piping infrastructure.
3. Develop the Belvoir Ranch High Plains aquifer and explore the Lance/Fox Hills aquifer on the eastern portion of the ranch.

4. Develop the western Belvoir Ranch.
5. Develop other new well fields to the north, outside of the LCWCA.
6. Develop other new well fields to the northeast, inside the LCWCA.

3.6 Aquifer Storage and Recovery

3.6.1 Introduction

Aquifer Storage and Recovery (ASR) is a process in which water is stored in the subsurface via injection, infiltration, or other method, and withdrawn for use at a later date. The water used for aquifer storage is often referred to as recharge water. ASR can be used for a number of purposes, including, but not limited to: seasonal, long-term, and emergency storage of excess water, enhancement of ground water levels, improved water quality, and offsetting peak demands. For a successful ASR operation, the following factors, at a minimum, must be addressed:

- *Long-term availability of sufficient recharge water* – ASR will not provide sustainable benefits to BOPU unless recharge water is available in the long term.
- *Compatibility of recharge water quality with aquifer* – This is especially important when using ASR injection wells to deliver the recharge water into a drinking water aquifer, as this water must meet drinking water standards.
- *Ability to demonstrate dominion and control over the recharge water* – To satisfy water rights requirements, it must be shown that the recharged water is accumulating within BOPU's well field and is available for beneficial use by BOPU. ASR injection or infiltration locations should be chosen so that recharge water will maximize benefits to the aquifer within BOPU's well field and not adjacent users.
- *Ability to obtain permits for ASR installation* – Regardless of the method of recharge, ASR operations in Wyoming will require permitting approval from the SEO and Wyoming Department of Environmental Quality (WDEQ).
- *Feasibility of delivering water to the recharge location* – If installations of extensive pipelines, treatment, or retrofitting of equipment for injection are required, the costs may outweigh the benefits of ASR.

3.6.2 Background

BOPU has studied the potential benefits of ASR with the primary purpose of slowing the decline in water levels in their four well fields, and thereby improving or extending existing well yields. The Cheyenne Aquifer Storage Recovery Test Project (CH2MHill, 1998) evaluated potential ASR applications within the Cheyenne urban area and within the Happy Jack well field. Most recently, BOPU commissioned the Managed Aquifer Recharge Storage and Recovery Project, completed by Lytle Water Solutions, LLC (LWS) in 2011. As a part of this study, each of the four BOPU well fields was evaluated for compatibility with various ASR technologies (rapid infiltration basins, vadose zone wells, and injection wells). The study concluded that only the Bell and Happy Jack well fields were amenable to ASR, with rapid infiltration basins or injection wells identified as feasible methods for delivering the water to the aquifer. The Federal well field was eliminated due to unfavorable hydrogeologic conditions, while the Borie well field was eliminated based on hydrogeologic conditions and the presence of TCE in the ground water in some areas. The unfavorable hydrogeologic conditions consisted of groundwater gradients that would move recharged water outside BOPU's area of control and shallow confining layers that would impede rapid infiltration basins or vadose zone injection. Concerns about TCE included the potential for increased mobilization of the contaminant if water is added to contaminated areas.

The LWS Study included a pilot test of a rapid infiltration basin and an ASR injection well in the Happy Jack well field. Results from the rapid infiltration basin pilot test indicated that although high infiltration rates were observed, intervening clay layers appeared to prevent the recharge water from reaching the Ogallala aquifer. Attempts to modify the design of the rapid infiltration basin to include a borehole filled with higher permeability material through the intervening clay layers to the Ogallala failed due to the inability to acquire approval from the WDEQ. The WDEQ rejected the proposal because they felt it would not provide sufficient protection of ground water.

The pilot test of the ASR injection well was conducted at the Koppes 3 well in the Happy Jack well field and included a total of four injection/pumping cycles over a three-month study period. Though the injection rates were limited by the diameter of the existing well, the ASR injection well pilot study was very promising, with significant head build-ups observed in surrounding monitoring wells. Results indicated that the ASR well operated at a high efficiency, with little pumping needed to clean the well screen. Observed water levels in surrounding monitoring wells correlated well with theoretical predictions, showing that BOPU would be able to demonstrate dominion and control over the injected water, a key requirement for ASR.

The LWS study concluded that both rapid infiltration basins and ASR injection wells were promising technologies for an ASR operation in the Ogallala aquifer, specifically within the Happy Jack well field. The use of rapid infiltration basins may be limited by intervening clay layers and the difficulty in permitting potential solutions for this issue. It was concluded that suitable quantities and means of conveying and treating recharge water to BOPU well fields were available. Study limitations did not allow for an evaluation of water quality compatibility, so further analysis of chemical compatibility was recommended. Recommendations were also made for further evaluation of ASR wells at higher injection rates.

3.6.3 Analysis of Key Factors

Availability of Recharge Water

The long-term availability of recharge water is critical to the success and sustainability of an ASR operation. The 2011 LWS Study evaluated the long-term availability of recharge water and determined that there would be sufficient amounts of excess water available through 2052. This analysis was based on water demand and availability projections from the 2003 Master Plan (Black and Veatch, 2003). This analysis has been updated based on the demand projections developed for this 2013 Master Plan (Volume 2) and results from water supply system modeling (using existing water supply conditions).

Projections from Volume 2 of this Master Plan indicate that the average day potable water demands will increase from 17,598 ac-ft/yr in 2013 to 33,179 ac-ft/yr in 2063, as shown in Table 3-25 below. The potential water available for recharge was estimated by using the SWSS model to determine a "safe yield" for each planning period (2013, 2023, 2033, and 2063). Safe yields were determined by artificially increasing the potable water demand until the shortages created in the model are equal to the excess water available. The excess water is calculated as the difference between the artificial increased demand and the starting potable water demand. In this manner, non-potable demands are removed from the safe yield amount and operational losses such as evaporation are accounted for.

Table 3-25
Preliminary Potable Water Demand Projections and Excess Water Available

Year	Average Day Potable Water Demand (mgd)	Annual Potable Water Demand (ac-ft/yr)	Average Water Available or "Safe Yield", Existing Conditions (ac-ft/yr)	Excess Water Available for Recharge (ac-ft/yr)
2013	15.7	17,598	25,074	7,476
2023	18.0	20,176	24,943	4,767
2033	21.2	23,763	22,835	--
2063	29.6	33,179	22,524	--

Table 3-25 indicates that while approximately 7,500 ac-ft/yr may be currently available for recharge, this amount will decrease until no significant excess water will be available. Figure 3-31 below shows the transition between projected years with excess potable water and a shortage of potable water occurring in approximately 2031. This analysis assumes that no additional sources of water are added to BOPU portfolio over the next 50 years. However, since projected average day demands exceed current supplies, additional sources of water will be needed to prevent shortfalls. Unless additional water supplies beyond projected demands are developed, excess water for recharge after 2031 may only be available during years with above-average precipitation.

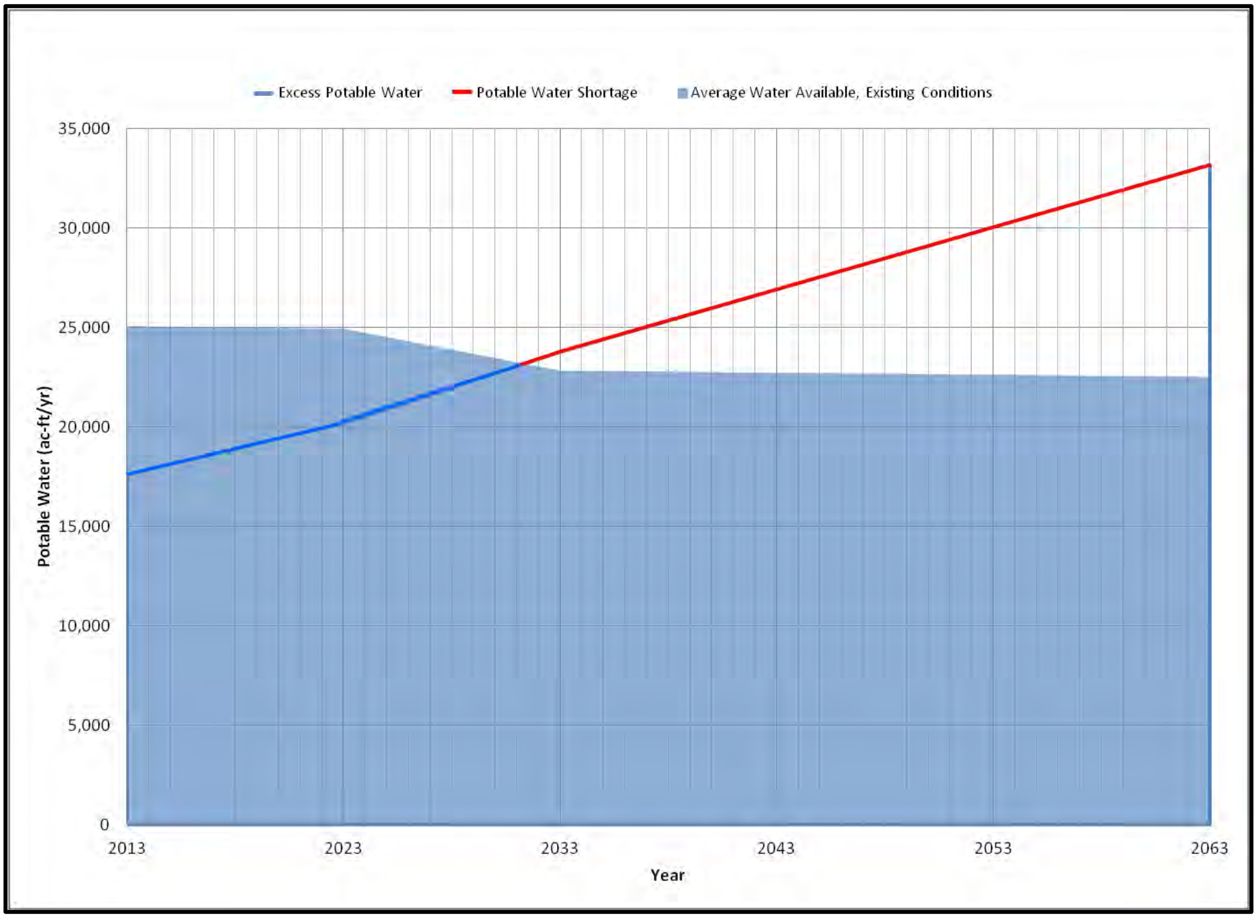

Figure 3-31
Potable Water Demand and Availability Projections

Compatibility of Recharge Water Quality with Aquifer

Recharge water quality is an important consideration for a successful ASR operation. If the chemistry of the recharge water is not compatible with the receiving formation, adverse effects from leaching or precipitation of minerals can occur. ASR injection wells can experience biofouling or plugging due to poor injection water quality. Injection wells typically have a small area through which water can enter the aquifer. The high injection flow across this area is vulnerable to losing hydraulic conductivity due to potential precipitates and biological growth. It is also important that the recharge water contains little to no suspended solids as this may also cause clogging. Back flushing, surging, chemical treatment, and filtering prior to injection are potential solutions to effectively remediate such issues.

The LWS pilot study did not directly evaluate the compatibility of recharge water with the receiving aquifer, as study limitations required water from the aquifer itself to be used for recharge. Biofouling or significant plugging would not be expected in a BOPU ASR operation, as treatment of recharge water will almost certainly be required prior to injection.

Ability to Demonstrate Control and Use of Recharge Water

To satisfy water rights requirements, it must be shown that recharge water is accumulating within BOPU's well field and is available for beneficial use by BOPU. The LWS pilot study was able to demonstrate control over water placed into aquifer storage via injection well. However, intervening clay lenses appeared to prevent the accumulation of water in the Ogallala aquifer.

The LWS pilot study demonstrated that existing wells in the Happy Jack well field with high transmissivity are located so that injected water would move in a general northeast direction and remain within the Happy Jack well field to be available to other BOPU wells. Similarly, the LWS study concluded that there are existing wells in the Bell well field with high transmissivity residing in an up-gradient location with respect to other wells in the Bell well field. Injection at these proposed locations could increase the saturated areas above the aquifer in unconfined locations as well as increase hydraulic head in locations where the aquifer is confined.

This ability to demonstrate control of the water will be a critical factor in the selection of specific sites for injection or infiltration so that the benefits of ASR are maximized for BOPU and not for adjacent users of the aquifer. The vicinity of the Bailey #5 well is one site that should be investigated as a possible injection/infiltration point. BOPU well field staff has observed significant differences in water levels between the Bailey # 5 well and other surrounding wells, suggesting that there may be some subsurface features there that serve as boundaries. If such boundary features exist, they may help in containment of any water that is placed there.

Ability to Obtain Permits for ASR Installation

The permitting process for both rapid infiltration basins and injection wells was outlined in the 2011 LWS study. Both technologies require permit approval from the SEO and WDEQ. In the case of rapid infiltration basins, the SEO would likely require the following:

- Application for Permit to Appropriate Surface Water
- Change of Place of Storage petition
- Change of Use petition, if SEO determines recharge is not a municipal use

For injection wells, the SEO would likely require:

- Application for Permit to Appropriate Ground Water
- Application for Permit to Appropriate Surface Water

The WDEQ (Water Quality Division) is likely to require the following for a rapid infiltration basin:

- Demonstration that the project would not cause a violation of ground water standards
- Demonstration that the design and construction of the rapid infiltration basin complies with Chapter 11 of the Wyoming Water Quality Rules and Regulations

For injection wells, the WDEQ (Underground Injection Control Program) would most likely require:

- Submittal of Underground Injection Control Permit, most likely as a Class V injection well.

There does not appear to be significant obstacles to permitting an injection well. However, if modifications to rapid infiltration basins that create a more direct pathway to the Ogallala are required, the WDEQ has indicated that a permit may not be obtainable.

Feasibility of Delivering Water to Recharge Areas

The LWS study concluded that only the Happy Jack and Bell well fields are suitable for ASR operations, citing unfavorable hydrogeology in the Federal and Borie fields, along with TCE contamination in the Borie field. The Belvoir Ranch was also eliminated from consideration due to TCE contamination issues and the lack of permitted municipal use. With the recent completion of the TCE treatment system, extraction and use of recharged water from the Borie and Belvoir fields would no longer be an issue. However, the potential for recharged water to increase TCE mobility remains as an obstacle to ASR in these areas. Further, the use of ASR in the Federal and Borie well fields is not likely to compare favorably to the Happy Jack and Bell well fields due to the proximity of the Happy Jack and Bell well fields to BOPU's existing raw water pipelines and treatment plants. Existing raw water pipelines cross both well fields, and all wells are located within 5 miles of the Sherard Treatment Plant or Round Top storage facility.

It appears feasible to deliver recharge water to the two well fields that have been identified as primary candidates for ASR, the Happy Jack and Bell fields. The cost of capital improvements necessary to deliver water to recharge areas is dependent upon the ASR technology chosen. Previous studies (LWS, 2011) have concluded that two technologies, rapid infiltration basins and injection wells, show the most promise for ASR. If rapid infiltration basins are used, capital costs will be much lower, as raw water supplies can likely be used for recharge water. Rapid infiltration basins could be constructed in close proximity to raw water pipelines that cross the Happy Jack and Bell well fields. For injection wells, the water must meet potable drinking water standards and would require treated water from the Sherard Treatment Plant to be piped back to the injection well sites. Additional costs would be incurred to retrofit existing wells for ASR or to install new wells for ASR. Figure 3-32 shows a schematic representation of the existing water supply infrastructure and potential infrastructure needed for implementation of ASR.

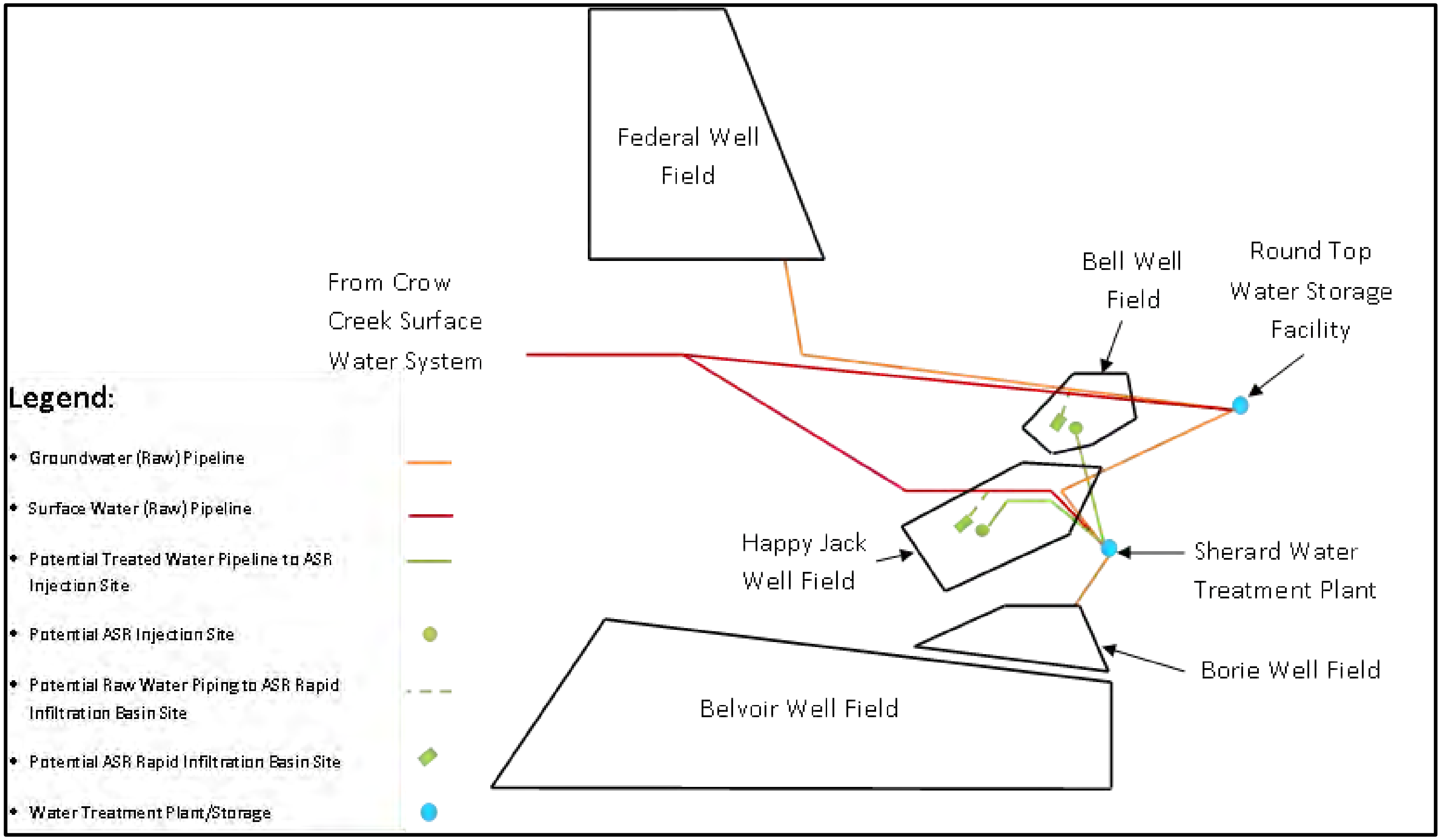

Figure 3-32: Schematic of Existing Water Supply Infrastructure and Potential ASR Infrastructure

3.6.4 Conclusions and Recommendations

ASR appears to be a promising method for BOPU to minimize declines in water levels in the Ogallala aquifer and thereby maintain current production levels. The two well fields most amenable to ASR currently provide approximately 60% of the total well field production, so efforts to maximize the productivity of these well fields can have significant effects on the ground water contribution to BOPU's water supply.

Previous investigations have demonstrated the potential for successful implementation of ASR at BOPU well fields. However, significant questions and data gaps remain that warrant further investigation prior to full-scale implementation. The most critical obstacle to full-scale implementation appears to be the lack of long-term water supply, as the current water supply system is projected to experience shortfalls starting after 2033. However, near-term water supplies appear adequate for an ASR program, and the benefits of implementing ASR in the near-term may improve long-term water production and help provide more time for BOPU to identify and develop new water supply sources.

It is recommended that an additional pilot scale study be conducted to satisfy the remaining questions and concerns. The additional study should consider the following:

- Further analysis of ASR should continue to focus on the Happy Jack and Bell well fields, due to their favorable hydrogeology, location relative to the existing BOPU infrastructure, and lack of TCE contamination issues. Specific injection/infiltration sites should be identified and evaluated. Since the Sherard Water Treatment Plant has the ability to treat TCE and potential for expanded treatment, future studies should include the potential for the use of the Belvoir Ranch for ASR.
- The pilot study should focus on ASR injection wells, until it can be shown that permitting of modified rapid infiltration basins (modified to create a higher permeability pathway through intervening clay layers) can be successful.
- The study should evaluate higher capacity injection, either by installing a new injection well or by retrofitting an existing well, that can accommodate the required injection and pumping piping.
- An analysis of the chemistry of the recharge source and aquifer water for compatibility must be included in the ASR study to ensure that adverse effects such as biofouling or plugging of injection or production wells will not occur.
- The study should include a detailed evaluation of the capital costs required for full-scale implementation.

3.7 Federal and Bell Well Field Connection Evaluation

Groundwater from the Federal and Bell well fields can currently be delivered to Round Top Storage Tank, where chlorine is added for distribution as potable water. In addition, output from the Bell Well field is currently sent via a both a 10-inch and a 14-inch pipeline to the Happy Jack Well field. From there it is sent to the Sherard WTP through the Happy Jack pipeline. Although this is a functioning system, it is not the ideal solution since there are current impacts which have not been studied in detail to the Bell and Happy Jack well outputs. The theoretical impact of the current configuration would indicate that the dynamic pressures are increased, particularly at the Happy Jack field, most likely pushing the pump curves into a much less than optimum output. A more direct line would isolate the flows from the Bell field, as well as allowing BOPU to bring in the output from the Federal Well Field.

Should an adverse event affect the ability of BOPU to receive water from the Stage I/II source through the Crystal Lake Reservoir transmission pipelines, BOPU needs the ability to send groundwater from the Federal and Bell well fields to the Sherard WTP to provide emergency water delivery. In addition, under non-emergency conditions, an interconnecting pipeline allows for blending of surface and groundwater. In case of a treatment plant failure or restriction, an interconnection could allow for provision of potable water through the King II reservoir at the Sherard WTP with only the required addition of chlorine. This interconnection would thus provide flexibility and redundancy to the current system of water treatment and distribution.

Based on the data collected for the 2003 Master Plan, which has been re-verified for this report, the peak pumping rates for Bell well fields are 2,550 gpm and for Federal well fields 2,585 gpm. These pumping rates are well above the sustainable production figures of 1,700 gpm (Bell) and 600 gpm (Federal), but the intertie pipeline is designed to provide transmission for the peak pumping capacity of the Federal Well Field output, which could be required for a short duration emergency. Well output is expected to fall slightly over time as the aquifers are drawn down, as indicated in the static well monitoring reports up to 2013.

3.7.1 Pipeline Routes, Rights of Way, and Terrain.

A hydraulic analysis indicates the piping from the Federal Well Field to Round Top has adequate capacity for the Federal Well field output although the 12-inch diameter pipeline from Federal to Wye is a bit undersized for peak outputs. The existing 24-inch pipeline from the Bell well field to Round Top could handle the Federal output to get the raw water to the Bell field by reversing the current flow pattern. From the Bell field southward, a new pipeline option is proposed to take the Federal Well field output directly to the King Tank at the Sherard WTP. Preliminary evaluation suggests that a 20-inch diameter pipeline would achieve this. The routing has been chosen to follow existing pipeline routes, ending by paralleling the transmission mains coming from Crystal Lake Reservoir along Happy Jack Road into the treatment plant. This routing minimizes the additional disturbance areas and should allow construction largely within existing rights-of-ways. The terms for those rights-of-ways may require modification to allow construction of an additional transmission main. The proposed alignment and current property ownership is shown on Figure 3-33 while the profile for the alignment is shown on Figure 3-34.

3.7.2 Materials and Controls

Based on the initial pipe routing and expected pressure regimes, C-905 class DR18 PVC piping would be the most economical alternative for the well-field intertie pipeline.

3.7.3 Flow Conditions, Pressures, and Capacities

The elevation at the King II Tank at the Sherard WTP is approximately 10 feet higher than the elevation at Round Top, resulting in a static pressure increase to pump to Sherard WTP of about 4.5 psi. The current system allows the Bell well field output to flow via a parallel 10-inch and 14-inch or larger piping to the Happy Jack well field and thence to the King tank. Since pressures in this system are slightly above the pressures resulting from gravity flow from the Round Top tank, Federal well field water at the Round Top tank cannot flow and combine with the Bell field flow. Flows through a 24-inch main would be reasonable, even at the maximum groundwater output combined from the Federal and Bell well fields. The dynamic losses at 5,135 gpm (7.4 mgd) through the proposed 24-inch transmission main would be an additional 30 feet (13 psi). Based on this analysis, an examination of the impact to pump curves was performed in the Bell well field with an increase in pumping pressure of 18 psi (40 feet of head).

A look at the available pump curves for the Bell well field shows several of the pumps to be similar 3-stage units, so the 40 feet of head results in an increase per stage of 13.3 feet. In one case, the theoretical output for an un-modified pump results in a reduction of 30%. A restoration or upgrade of pump impellors to the maximum for the pump housing would compensate for the increased head in several cases without a reduction in output. It also appears from the handwritten notes on the pump curves that the actual total dynamic head for several of the pumps is below the design head, also allowing some leeway in adding head to the pump without destroying the efficiency.

3.7.4 Cost and Economic Feasibility

A preliminary cost opinion, including allowances for right-of-way adjustments, appurtenances, control valves, air evacuation installations, and cathodic protection on metallic appurtenances or pipes would indicate this project would cost approximately $3.92 million, using a 16-inch diameter pipeline and incorporating a low head, high capacity booster station. The project would be technically feasible. The need for a booster station is not entirely clear, since pump modifications appear feasible to overcome the increase in dynamic head to the wells that were studied, however, not all pump curves were available during the course of this investigation. The cost of individual well pump modifications are assumed to be staggered over time, targeting first the pumps most impacted by the proposed modifications. If the pressure directly from the

Federal well field were used to push water over to the King tank, a cross connection around Round Top tank would have to be installed and the residual pressure from the elevation difference to the Federal field would allow the sustainable yield (about 600 gpm or 0.864 mgd) to flow by gravity, not requiring a booster. Peak output of the Federal well field has a loss in the 12-inch section of transmission pipeline which essentially precludes gravity flow all the way to the King tank.

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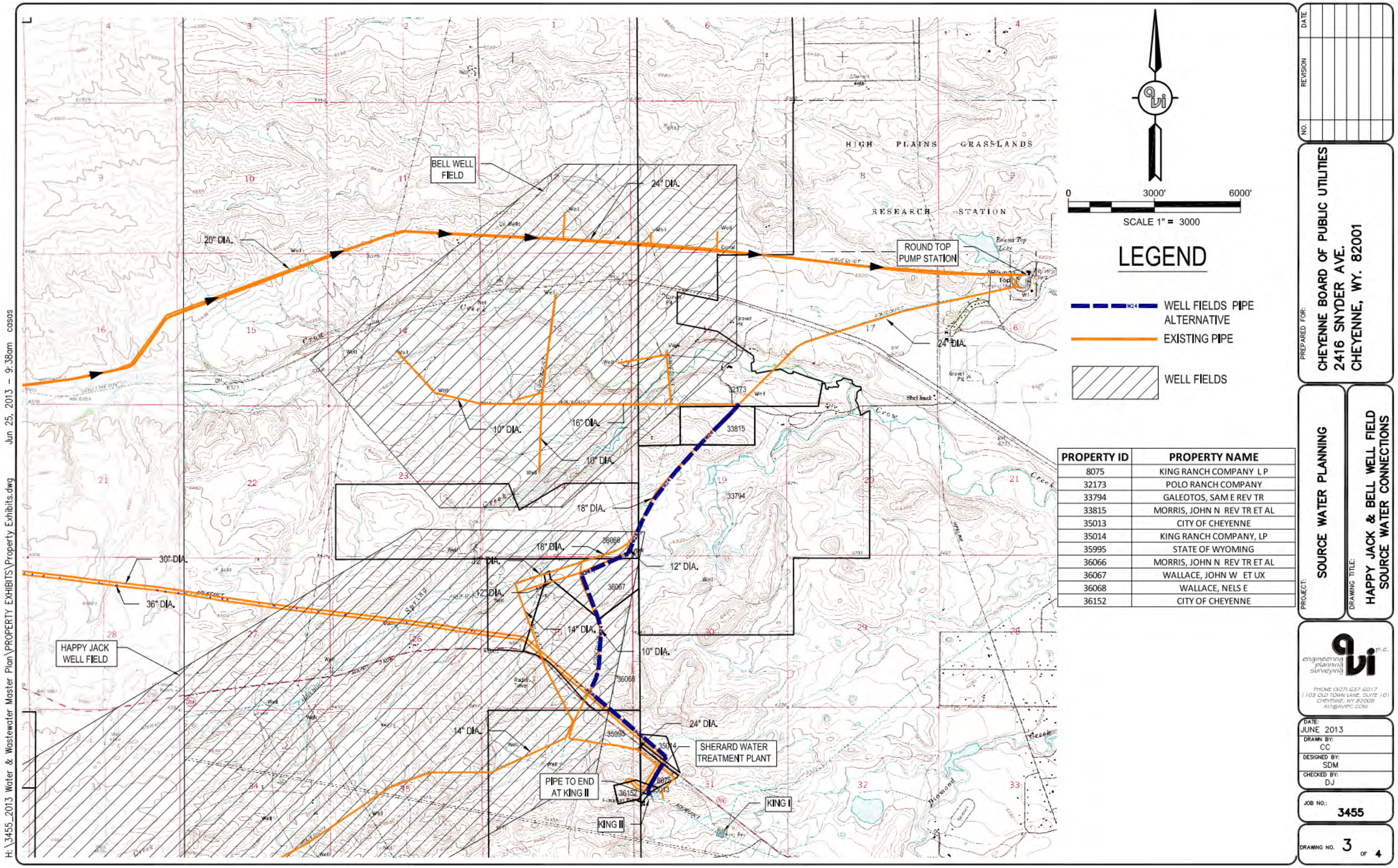
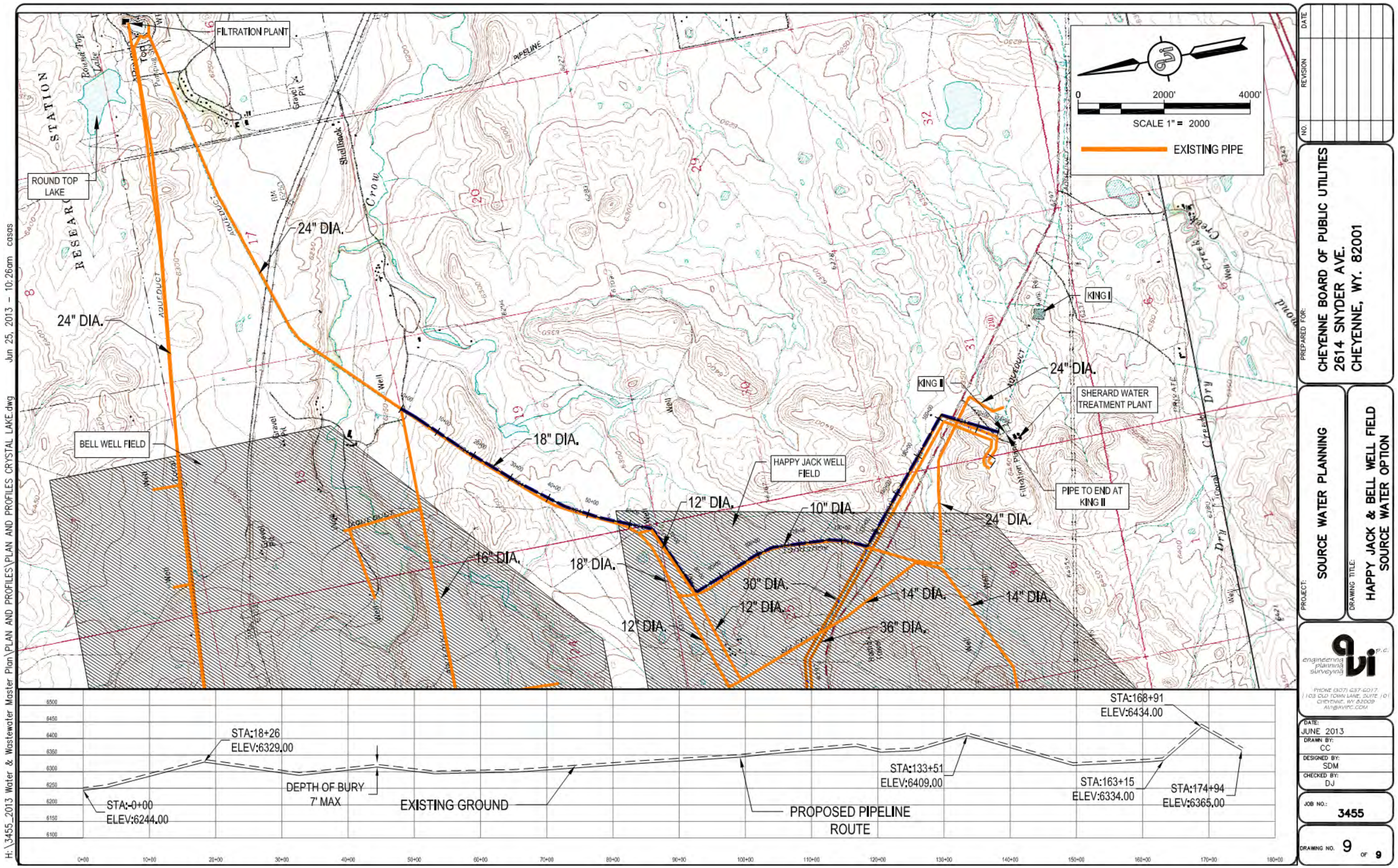
Figure 3-33
Federal and Bell Well Field Connection Potential Alignment Alternative



PROPERTY ID	PROPERTY NAME
8075	KING RANCH COMPANY L P
32173	POLO RANCH COMPANY
33794	GALEOTOS, SAM E REV TR
33815	MORRIS, JOHN N REV TR ET AL
35013	CITY OF CHEYENNE
35014	KING RANCH COMPANY, LP
35995	STATE OF WYOMING
36066	MORRIS, JOHN N REV TR ET AL
36067	WALLACE, JOHN W ET UX
36068	WALLACE, NELS E
36152	CITY OF CHEYENNE

Figure 3-34
Federal and Bell Well Field Connection Profile for the Potential Alignment



3.8 Granite Springs and Crystal Lake Bypass Pipeline Evaluation

Currently all Stage I and Stage II water from the raw water collection and storage system is discharged to Crystal Lake and Granite Springs Reservoirs for temporary storage before being withdrawn from Crystal Lake Reservoir through the transmission pipelines to the Sherard WTP. If either reservoir were rendered inoperable because of infrastructure failure or contamination, BOPU's ability to deliver water during high demand periods would be severely compromised. This evaluation considers the feasible options for bypassing the reservoirs using a direct connection for the raw water system upstream of the reservoirs to the existing transmission pipeline below the reservoirs. The alignment options and current property ownership information is shown on Figure 3-35.

3.8.1 Possible Pipeline Routes

Two scenarios have been evaluated. The first route (Alternative 1a) essentially follows the Crystal Lake road (Laramie County Road 210) from a point in the southwest corner of Section 27, Township 14N, Range 70W (elevation 7,500 feet), to intercept the Sherard transmission pipeline in the SE Quarter of Section 25 (elevation 6,770 feet), near Hecla, Wyoming, a distance of 15,240 feet (2.89 mi.). A variation on this route (Alternative 1b) is to cut away from the roadway northward along Granite Springs Road in the southwest Quarter of Section 25 towards Crystal Lake Reservoir, along a drainage swale eastward to connect to the Sherard transmission pipeline near the center of Section 25, a distance of 14,043 feet (2.66 mi.). This would avoid a slight hill along the roadway and result in a reduction of pipeline distance (about 1,200 feet less). This option may have some potential access difficulties over private lands along the drainage swale near the Sherard transmission pipeline but does allow the bypass flow to intersect the 50-inch transmission pipeline (Crystal Canyon Pipeline) before it splits to feed the 36-inch and 30-inch transmission pipelines to Sherard WTP.

A second route intercepts the current source discharge pipeline near Crystal Lake Reservoir in the northwest corner of Section 26. An initial alignment studied a route down the west edge of Crystal Lake Reservoir, then skirting the Lake around the south side and back up the east and down into the canyon below the Crystal Lake Reservoir Dam to the Sherard transmission pipeline. This route is more circuitous than the first route, and has additional challenges of bedrock and very steep slopes at the end of the route, but could potentially be largely constructed in Granite Springs Road—a gravel recreation access road. It is approximately 7,652 feet (1.45 mi) total distance around the lake. A more careful examination of conditions in the field has indicated that this route would not be practical because of the rock outcroppings and steepness of the slopes for the direct route to outlet works area, and that the dam abutments could be at risk if the pipeline is tunneled through the rock in this area. Based on these considerations, the option is not shown in Figure 3-35 to Figure 3-38.

More practical variations on this route are to route the pipeline south from the southeast tip of Crystal Lake Reservoir (Alternative 2a) to Crystal Lake Road and take the same route toward Hecla as given in option 1a for a distance of 10,532 feet (1.99 mi.). Alternatively, from the southeast tip of Crystal Lake Reservoir, a route (Alternative 2b) could be established toward the north east and then eastward through the drainage swale to connect with the 50-inch transmission line near the center of Section 25, a distance of 8,848 feet (1.67 mi.), using the same route discussed in option 1b. In both these options, it was assumed that the pressure (14 psi) required to overcome the rise out of the lake area (up to 33 feet of elevation) could be gained from the existing piping as it comes down the hill from the Source Transmission Mains and that a pump station would not be required to operate. The rise would require installation of an air release valve. The proposed alignments are shown on Figure 3-35.

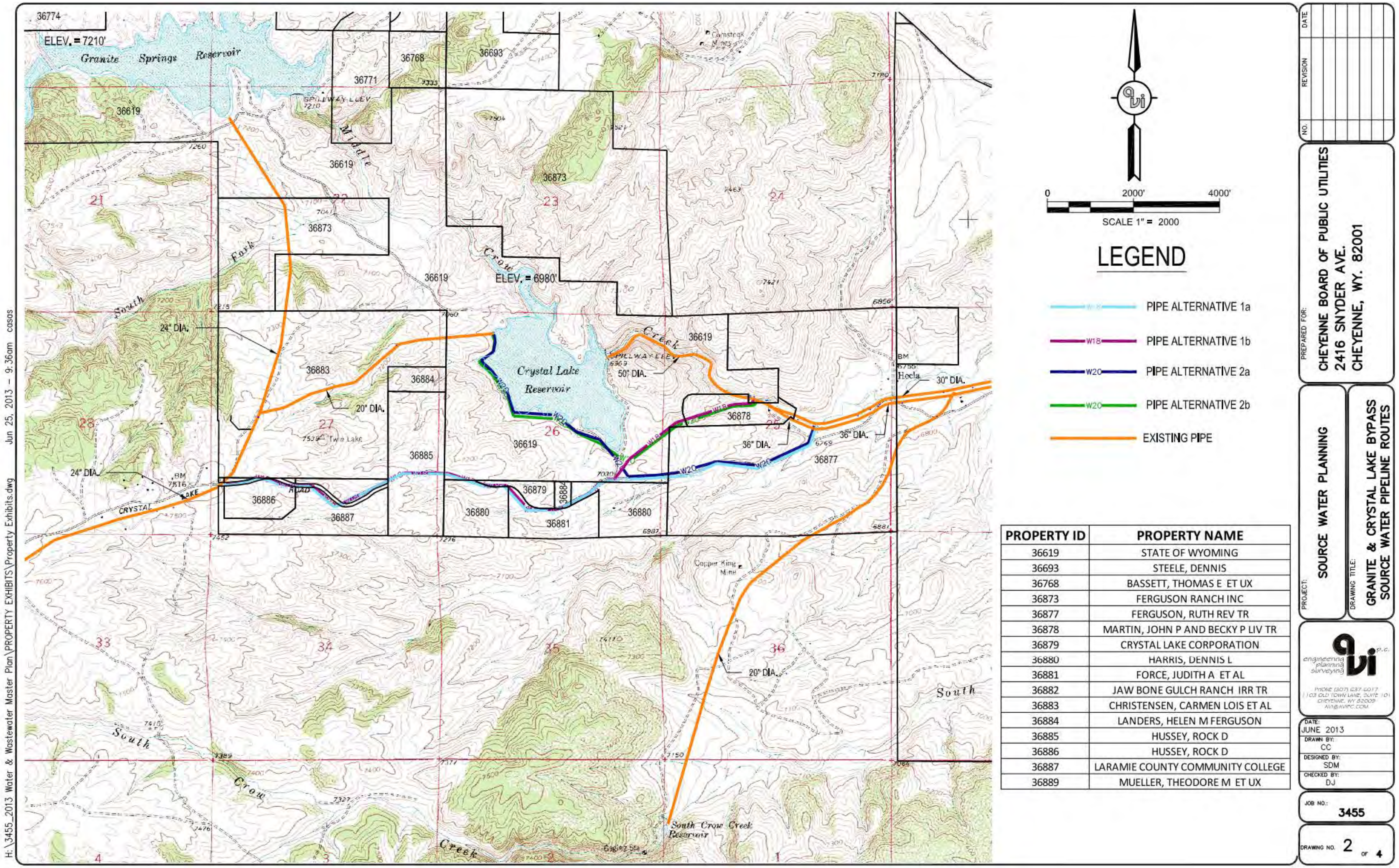
3.8.2 Rights of Way

Most of the proposed routes fall within private lands but may make use of the corridor which includes County Road 210 (Crystal Lake Road). This has the advantage of using previously disturbed areas, minimizing new impacts to lands and wildlife. The optional routing which uses a natural drainage running on the east side of Crystal Lake Reservoir northward from the road to Middle Crow Creek (to connect there to the transmission pipeline) runs across a private parcel. The options following the lake shore or Granite Springs Road falls entirely within property already owned by the City of Cheyenne, so would pose the least difficulty from a right-of-way perspective.

3.8.3 Terrain

The shortest distance and least total elevation change routing is Alternative 2b, with the pipeline passing around the Crystal Lake Reservoir on its south side. The elevation change for this option is approximately 190 feet, though most of that occurs in the final leg going east of the reservoir down into Crystal Canyon. The alternatives which follow the Crystal Lake Road generally have a slope from west to east in the range of 4.5 to 5.3 percent. Conceptual profiles are shown for each of the options in Figure 3-36 to Figure 3-39.

Figure 3-35
Granite Springs and Crystal Lake Bypass Pipeline Options



PROPERTY ID	PROPERTY NAME
36619	STATE OF WYOMING
36693	STEELE, DENNIS
36768	BASSETT, THOMAS E ET UX
36873	FERGUSON RANCH INC
36877	FERGUSON, RUTH REV TR
36878	MARTIN, JOHN P AND BECKY P LIV TR
36879	CRYSTAL LAKE CORPORATION
36880	HARRIS, DENNIS L
36881	FORCE, JUDITH A ET AL
36882	JAW BONE GULCH RANCH IRR TR
36883	CHRISTENSEN, CARMEN LOIS ET AL
36884	LANDERS, HELEN M FERGUSON
36885	HUSSEY, ROCK D
36886	HUSSEY, ROCK D
36887	LARAMIE COUNTY COMMUNITY COLLEGE
36889	MUELLER, THEODORE M ET UX

Figure 3-36

Granite Springs and Crystal Lake Bypass Pipeline Alternative 1a

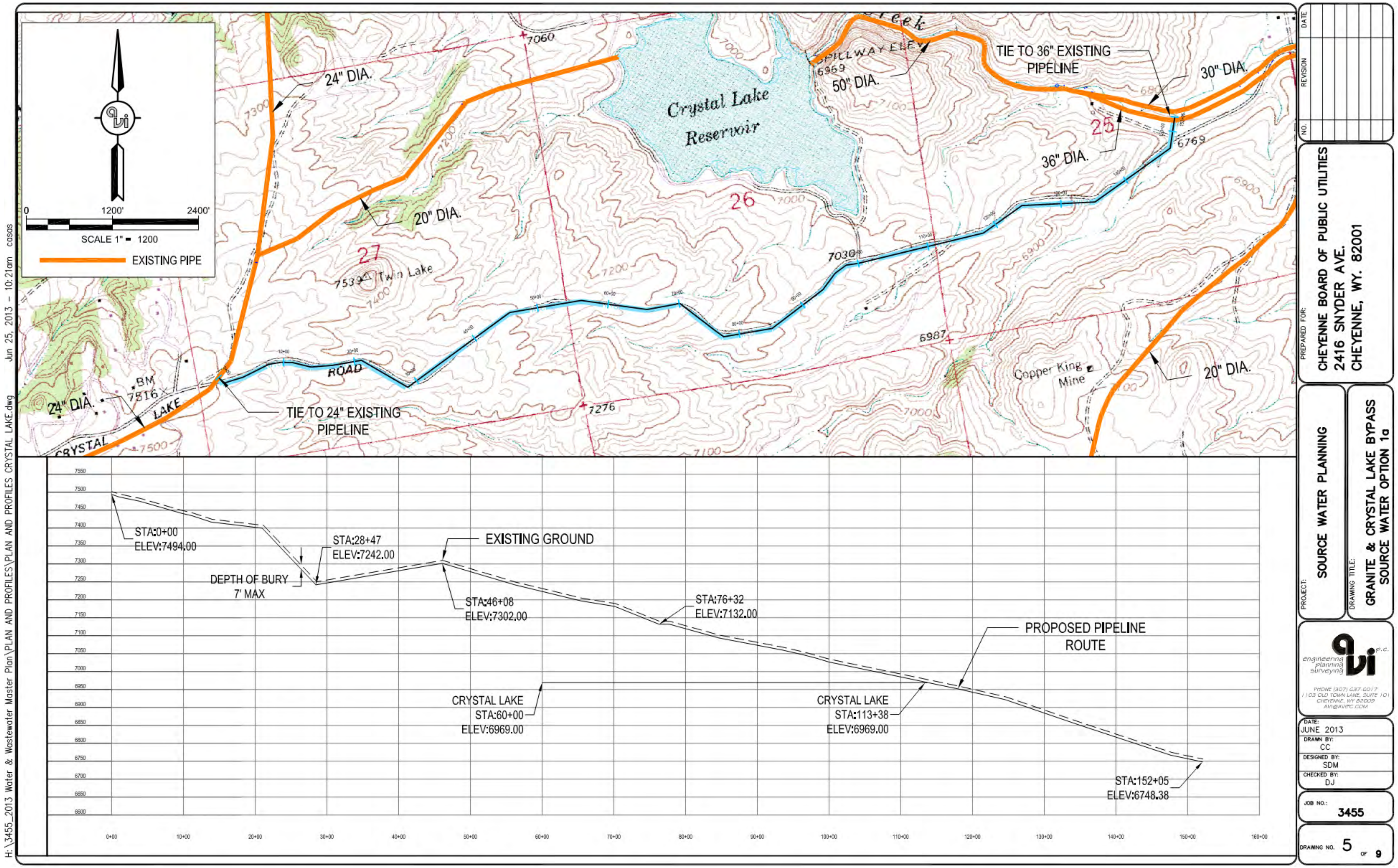


Figure 3-37
Granite Springs and Crystal Lake Bypass Pipeline Alternative 1b

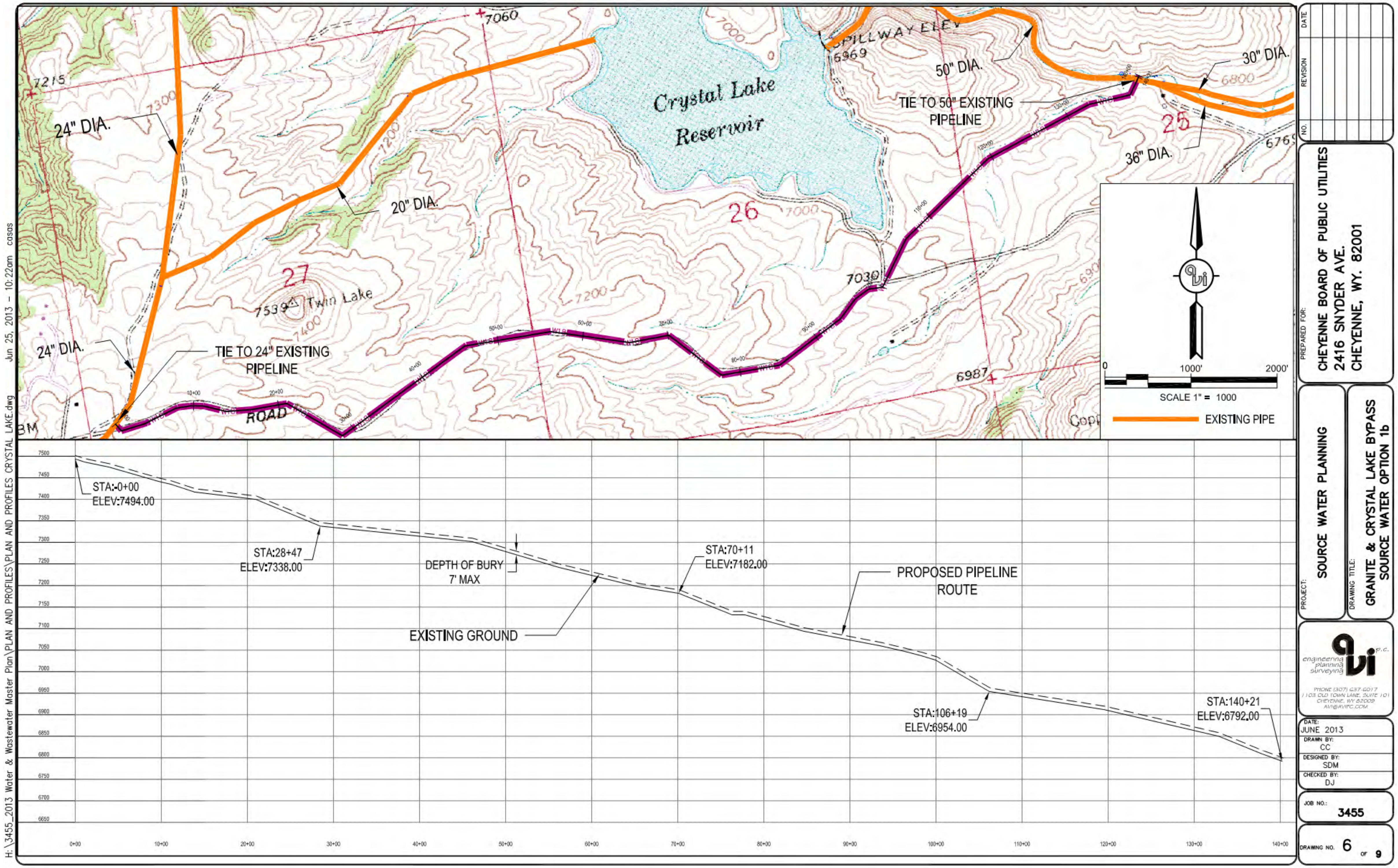


Figure 3-38

Granite Springs and Crystal Lake Bypass Pipeline Alternative 2a

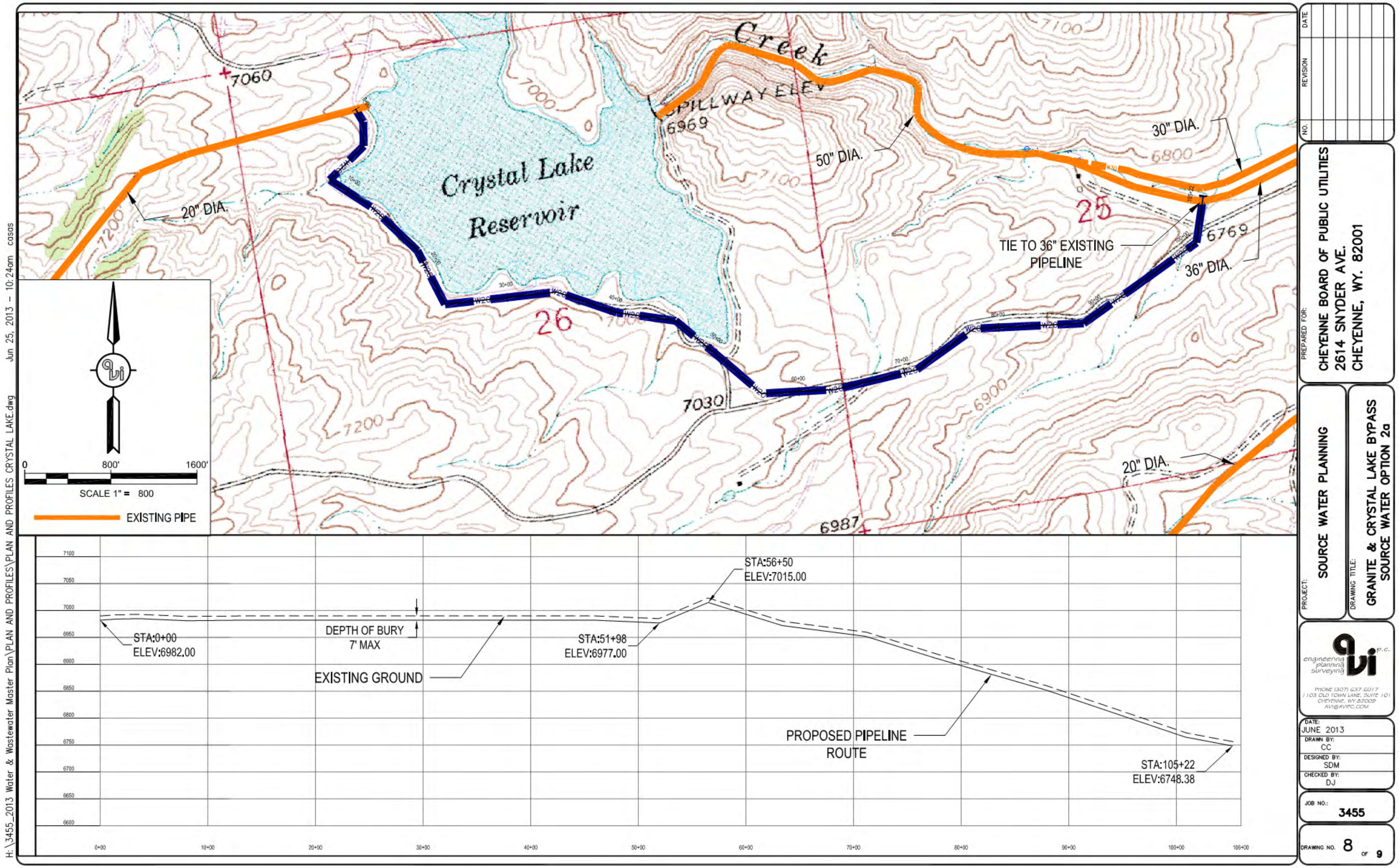
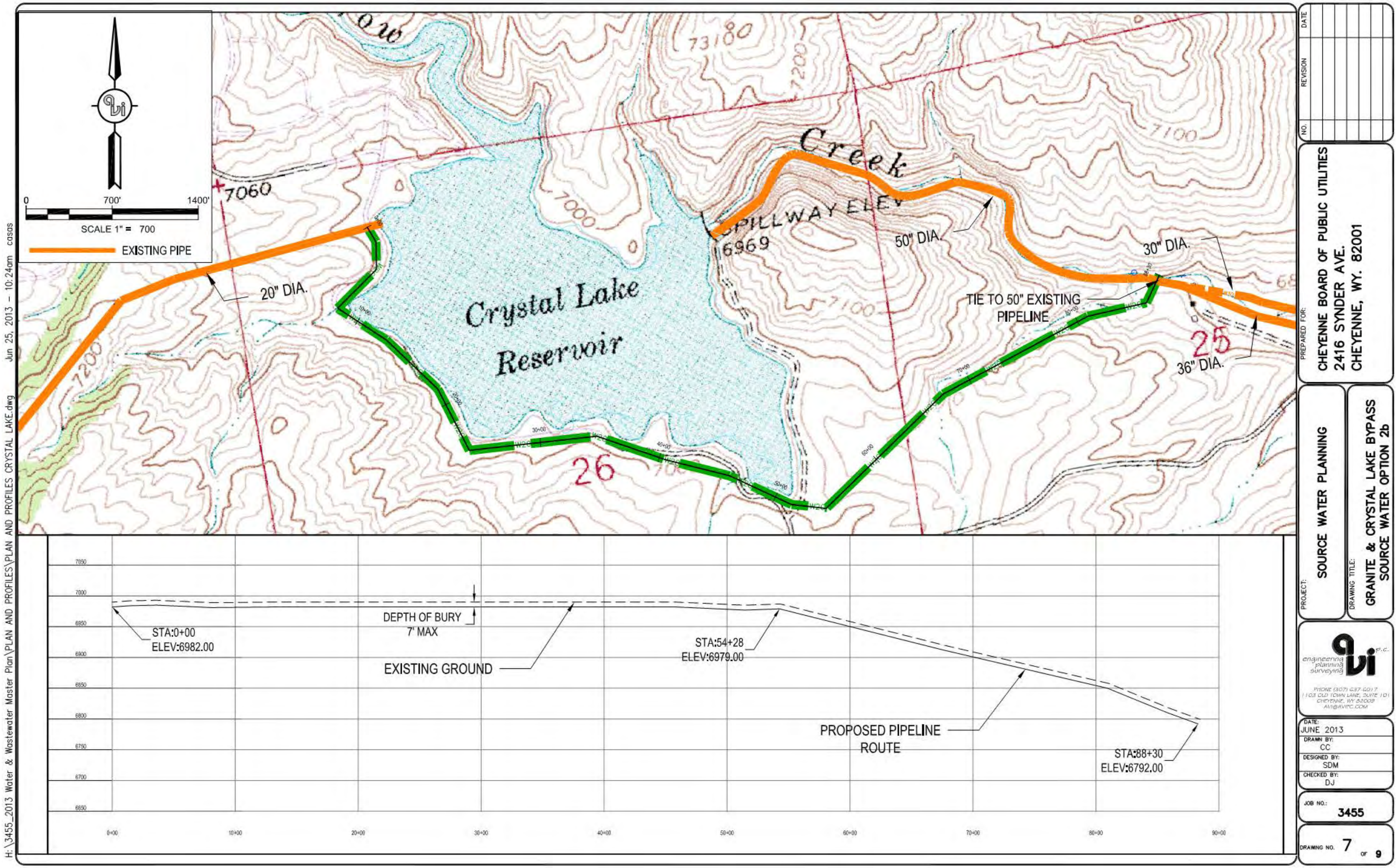


Figure 3-39
Granite Springs and Crystal Lake Bypass Pipeline Alternative 2b



3.8.4 Materials and Pipeline Controls

Pipeline materials in current use are generally cathodically protected, concrete lined steel or ductile iron pipe. The new bypass pipeline could use any of these materials, which yield a long lifespan with low maintenance requirements. If the pipeline is designed to keep operating and transient pressures below 250 psi, PVC pipe could be considered, particularly for 24-inch and lesser diameter pipelines.

3.8.5 Operating Regimes

The bypass pipeline would be designed to take the entire source (Stage I/II) output to the transmission pipeline which delivers raw water from Crystal Lake Reservoir to the Sherard WTP. This operating regime would be used in the event Crystal Lake Reservoir, (and to a lesser extent, Granite Spring Reservoir), would have to be taken off line due to contamination of the reservoir by either natural or man-induced causes. Certain maintenance activities which might require drawdown of either reservoir could also trigger use of the bypass pipeline; for example to work on the outlet works.

3.8.6 Flows

Design flow of the 30-inch source transmission pipeline is 20 to 30 mgd, though the current discharge lines into the reservoirs appear to be individually less than 20 mgd. The bypass pipeline should be designed to accommodate at least 20 mgd.

3.8.7 Pressures

The pressure at the tie-in for option 1 was assumed to be 0 psi (atmospheric) giving a residual pressure at Hecla of around 300 to 320 psi based on elevations and flows, requiring the strategic installation of a PRV. Pressures should be adjusted to simulate having the reservoir on-line so that the transmission pipeline and all appurtenances would operate normally.

3.8.8 Transients

Operation of valves along the bypass pipelines could result in transients, particularly if the valves are operated too rapidly. It is recommended that speed limited valves, along with a possible damper and extra-strength pipe near the main valves be used to absorb the excess energy resulting from operation of the valves. Normal procedure to remove the bypass from service would be to open the lake outlets, then slowly close the intake end of the bypass pipeline first, before closing the outlet end. Under normal conditions, there is little risk in allowing the outlet end of the bypass to remain open as water would only flow back to static lake elevation. To place the bypass into service, the valve operation order given above should be reversed.

3.8.9 Cost estimates

The estimated cost for construction for each of the bypass route options is shown in Table 3-26. These cost estimates are based on Means cost estimating documentation and do not include property and permit acquisition for the selected alignment.

Table 3-26
Estimated Costs for Crystal Lake Bypass Options

Option	Route	Diameter (inches)	Material	Distance (feet)	Unit Cost ($/ft_	Concept Cost
1a	Crystal Lake Rd to Hecla	18	PVC	15240	120	$ 1,828,800
1b	Crystal Lake Rd through NE Sec.	18	PVC	14043	120	$ 1,685,200
2a	Around lake to CL Rd to Hecla	20	PVC	10532	140	$ 1,474,500
2b	Around lake through NE Sec.	20	PVC	8835	140	$ 1,236,900

3.9 Crystal Lake Dredging Evaluation

3.9.1 Preliminary Assumptions

The purpose of the Crystal Lake Reservoir dredging evaluation is to evaluate the feasibility and economic benefits of restoring the water supply capacity of the Crystal Lake Reservoir (Reservoir) by removing accumulated sediment. Dredging extents and operations may be limited in order to protect aeration pipelines, outlet works, and other reservoir infrastructure. The Crystal Lake Reservoir dredging evaluation is based on limited available data regarding the sediment currently deposited within the reservoir. Although a bathymetric map is available that delineates existing water depth contours and elevations, specific data related to the extent, thickness, and physical characteristics of the sediment is not available for this dredging feasibility evaluation. Therefore, several assumptions have been made in order to conceptually evaluate three different reservoir dredging options. These options are shown in Figure 3-40.

Since the bathymetric map only indicates existing water depth based on a pool elevation of 6,972 feet, it is not possible to determine specific areas impacted by sediment deposition. Halligan Reservoir, located on the North Fork of the Cache la Poudre River in Colorado, may be an appropriate analog for potential sedimentation in Crystal Lake. Halligan Reservoir is a concrete arch dam, constructed approximately in the early 1910s and currently impounds 6,400 acre-feet. The upstream basin is granitic bedrock. In 1996, flushing of large amounts of sedimentation from the reservoir devastated downstream habitat. Since then the sedimentation dynamics of this reservoir have been extensively studied (for example Rathburn et al, 2005[2]). Sedimentation in Halligan largely originates as suspended solids. Sedimentation rates during snowmelt range from 2 to 554 grams per second. Total sedimentation in the reservoir, estimated in 2003 after the flushing event, was calculated as 310 acre-feet.

This evaluation assumes that all areas of the reservoir that have existing water depths located above the existing water supply intake elevation of approximately 6,925 feet would be included in the potential dredging limits. Areas with water depths below the intake elevation will be considered to be in the dead storage zone and therefore not considered for dredging. Approximately 104 acres of the total 134 surface acre reservoir have water depths that are equal to or shallower than the 6,925 feet intake elevation. It is assumed that all existing water depth contours extend down to the top of the soft sediment. For estimating purposes, an area of 100 surface acres has been selected for evaluation since the near shore or littoral zone of the reservoir will be excluded from any dredging as either a no-cut buffer or simply due to the likely absence of soft sediment. Since sediment measurements are not available, average sediment

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[2] Rathburn, S.L., J.B. Finley, S.M. Klein, B.R. Whitman. 2005. "Assessing reservoir sedimentation using bathymetic comparison and sediment loading measurements", presented as the 2005 Hydrology Days, Colorado State University.

thickness values of three (3) and five (5) feet have been selected for the estimated 100 acre potential dredging area evaluation. It is assumed that some areas may have thinner sediment deposits whereas other areas may have thicker and more significant amounts of deposition.

In addition, a third dredging option targets the approximate twenty (20) surface acre area located at the shallowest and westernmost portion of the reservoir immediately adjacent to the inlet point of Middle Crow Creek. Due to its location relative to the tributary entry point, this area has a high probability of sediment deposition and water depths are generally less than 40 feet. For estimating purposes, an average of five feet of sediment deposited throughout the 20 acre area is assumed.

Based on the three potential sediment volume scenarios described above, potential dredging quantities for the three options can be estimated by multiplying the number of surface acres times the average sediment thickness times 1,613.33 cubic yards per acre-ft. The smaller 20 acre area located at the upstream end of the reservoir would include approximately 161,333 cubic yards (101 acre-feet); the 100 acre live pool area with an average of three feet of sediment would include dredging approximately 483,999 cubic yards (300 acre-feet); and the 100 acre area with a five ft. average thickness would include the removal of approximately 806,665 cubic yards (500 acre-feet). While these are assumed sediment thicknesses, this range provides a bracket similar to sedimentation rates documented in Halligan reservoir.

There are three primary means of dredging that can be considered for Crystal Lake Reservoir:

- Reservoir drawdown and excavation (e.g. mechanical “dry” dredging)
- Mechanical wet dredging
- Hydraulic dredging

3.9.2 Reservoir Drawdown and Excavation

In this approach, the reservoir is completely or partially drained, the exposed bottom materials are allowed to dry out, and earth-moving equipment is brought in to remove the unwanted sediment. This method requires building a haul road to provide access for heavy trucks into the reservoir. In certain applications, this approach can be a viable alternative to other forms of dredging, but has a greater environmental impact, and takes the longest to complete due to the impact of unanticipated runoff events. The drained reservoir is unsightly and is not usable during the drying phase, which can last several weeks to several months. Typically, the sediment never truly dries; excavation occurs under muddy conditions. The method is highly susceptible to weather conditions and even a small storm event can affect the project schedule by re-wetting the sediments. Tributaries into the lake must be re-routed through channels or using pumps and pipes. The tributary routing system must have the capacity to handle sudden runoff from major storm events, otherwise the work area can be flooded. The raw surface water supply to Sherard would need to bypass Crystal Lake, requiring the construction of the conceptual bypass pipelines described in Section 3.8. Once the project is complete, it may take several months to a year or more for the reservoir to fill back to capacity, which is generally not acceptable for a water supply reservoir. Lake biota and fish stocks must then be replenished and re-established.

This method is considered risky and unsuitable for Crystal Lake Reservoir primarily due to the loss of water supply storage while sediment removal work and reservoir re-filling is being conducted.

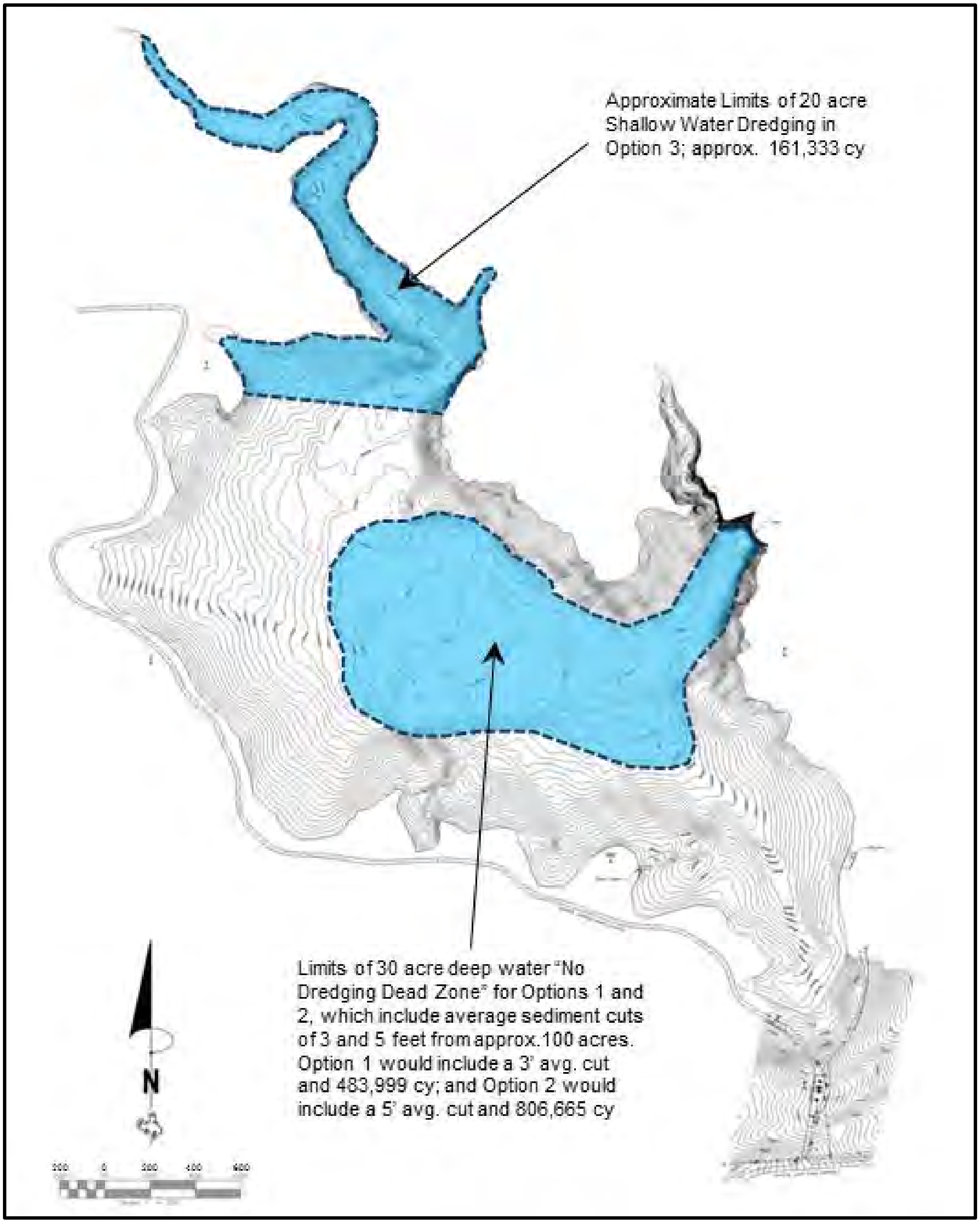

Figure 3-40
Crystal Lake Reservoir Potential Dredging Options

3.9.3 Summary of Potential Wet Dredging Options

Mechanical Wet Dredging

Mechanical wet dredging uses an excavator (such as a track hoe), dragline or a clamshell to remove the sediments. The dredging is conducted from shore, or from a barge in the water. Mechanical dredging often creates increased levels of turbidity (suspended sediment) within the water; specialized "environmental" or closed clamshell buckets can be used to reduce the turbidity, however these add cost and add water to the dredged material and are not generally required unless the sediment is contaminated. Mechanical wet dredging produces a plastic to fluid mud of varying consistency; the material can be handled and transported without dewatering although there is risk of spillage if the mud is too fluid. In addition, many facilities that accept dredged material for beneficial reuse or placement will not accept material with high water content. Examples of mechanical wet dredging are shown Figure 3-41. Variations of mechanical wet dredging are discussed below.

Figure 3-41
Examples of Mechanical Wet Dredging

Mechanical Dredging from Shore – The excavator or clamshell works from the shoreline of the lake. Surrounding trees and other obstructions are removed, and the shoreline must be leveled. The excavator places the dredged material into trucks for transport. Haul roads are constructed around the shoreline so trucks have access to the excavating equipment. Dredging is typically limited to near shore areas within reach of the equipment – typically 30 to 40 feet (a maximum of about 50 to 60 feet with specialized equipment). This approach is feasible only for small ponds and cove areas with good shoreline access for equipment.

Mechanical dredging from shore is not considered feasible for the Reservoir due to the environmental impacts of constructing a road at the water's edge around the Reservoir, the potential impact to sensitive littoral habitat within the Reservoir from dredging along the shoreline, and the inability of this method to access the middle of the Reservoir.

Mechanical Dredging from Barge – The excavator or clamshell operates from a barge or floating platform in the water. Sediment is placed into another barge for temporary containment. When the sediment containment barge is full, it is moved to a lakeside staging area, where another excavator removes the material onto the shore or into trucks. The method requires handling the sediment several times. Clamshell dredging can be expensive, and is generally used for deep water river and harbor dredging. Mechanical dredging requires good lakeside access to deploy the barges and excavating equipment, and to provide truck access for removing the sediment and hauling it to the placement site. The dredged material from mechanical wet dredging can be muddy or even soupy in consistency and can generally be handled by equipment without additional dewatering. However, if the material is too wet, it may require some settling and decanting of excess water prior to transport or placement. The wet sediment is heavy, and trucks may be required to run at 30% to 50% less volume than capacity, increasing the number of truck trips. Again, a haul road is needed to access the lakeshore staging area.

Mechanical dredging from a barge with subsequent truck transport of the sediment would require a lakeside staging area suitable for heavy truck traffic, an upland site suitable for the placement and temporary containment of wet sediment, and potentially hauling to a final destination.

Another option for mechanical dredging from barge includes in-water placement. The excavator or clamshell operates from a support barge in the water. Sediment is placed into another containment barge. When the containment barge is full, it is moved to another in-water location, and the sediment is deposited within the same water body. The placement would target inactive storage areas or possibly be used to restore shorelines. Typically, the barge hauling the sediment has a hopper type opening on the bottom. When the barge is positioned over the placement site, the hopper is opened and the sediment drops to the bottom. This method is typically used in dredging sand from harbors, estuaries, and oceanfront areas, but may also be suitable for large lakes and navigable rivers where fine-grained silts and clays are absent and there are deep water areas. In some cases, the sediment may be placed into the water by another excavator in order to achieve some specific shape or beneficial reuse, such as wetland or island creation, reef creation, or other habitat. The placement of the dredged material back into the water can cause increases in turbidity, particularly when the material contains fine-grained silts and clays.

The release of dredged material back into the water is not a typical practice for water supply reservoirs. The placement of sediment within the deeper areas of Crystal Lake Reservoir would impact deeper aquatic habitat and potentially impact water quality. With recent regulations by the EPA regarding anti-degradatation requirements, the placement of substantial volumes of sediment within the lake would pose substantial regulatory hurdles.

Mechanical Dredging with the dredged material placed back into the Reservoir within the "dead zone" or deep water area below the water intake is not feasible due to anticipated concerns from federal, state, and local regulatory agencies regarding impacts to water quality and aquatic habitat.

Hydraulic Dredging

A hydraulic dredge (also called a suction dredge) works like a floating vacuum. The typical dredge used for lakes is about the size of a small houseboat, and consists of a diesel motor, pump, and small operators cab (see Figure 3-42 as an example). A boom with a rotating cutterhead or horizontal auger is lowered into the sediment to loosen the bottom material. A suction hose attached to the boom pulls in the loosened sediment – much like a carpet sweeper attachment on a household vacuum. The sediment slurry is then pumped through a temporary HDPE (high density polyethylene) pipeline to an offsite location for dewatering. The dredge pipeline typically floats on the water surface and/or is laid on the ground surface to reach the dewatering site. Depending on the location and set-up of the dewatering area, there may also be a return pipeline for the water used to carry the sediment. Alternatively, the water may be discharged to the nearest surface water – however such a setup typically requires an additional layer of regulatory review and permitting.

Figure 3-42
Example of a Hydraulic Dredge

Hydraulic dredges typically generate less turbidity than mechanical dredges. There is minimal disturbance of the lakeshore, and the main dewatering area does not have to be at the lakeside. However, there must be an access point to put the dredge into the water. Large hydraulic dredges could be lifted into the lake with a crane, whereas many of the smaller dredges could be launched from a boat trailer. Hydraulic dredging operations also require a lakeside staging area where the pipeline pieces can be delivered and assembled, and to support the dredging operation.

A hydraulic dredge can pump the material in the pipeline a short distance. If the distance is greater than one mile, or involves pumping over any substantial hills, one or more booster pumps may be necessary. The booster pumps must be accessible for operations, fueling, and maintenance. The booster pump stations can be on land or on a barge in the water, depending on pipeline configuration and location of the dewatering site(s). Pipelines are typically placed on the surface, and although temporary, need to be solidly constructed to avoid leaks. The pipelines can be routed under roadways using cut and fill, jack and bore or horizontal directional drilling technique. Pipeline and pumping costs increase with distance, total elevation, number of roadway crossings, and if there are right-of-way issues or special permits required (e.g. stream crossings, wetland issues, etc.).

Hydraulic dredging is generally the least obtrusive of the available dredging methods for lakes and reservoirs. However, because hydraulic dredging relies on pumping water at high velocities to move the sediment, some form of dewatering (e.g. separating the sediment from the water used to transport the sediment) is necessary. Hydraulic dredging is a feasible option for dredging the Reservoir provided the dredged material can be stored and dewatered cost-effectively.

3.9.4 Sediment Flushing through Outlet Works

Flushing reservoir sediment through the existing outlet works is not considered to be a feasible or environmentally acceptable option for Crystal Lake Reservoir. The process of rapid discharge by opening the outlet works would generally discharge locally deposited sediment located within close proximity to the upstream outlet works openings. Additional agitation could increase the extent and volume of sediment to be discharged, but this process would create significant water quality impacts within the reservoir and downstream of the reservoir. Sediment deposited in the upper end of the reservoir would not be easily accessed for flushing due to the distance from the outlet works and the probability of re-deposition due to the morphometry of the reservoir. In order to be reasonably effective over a larger area, multiple flushing and partial drawdown efforts would likely be required in conjunction with physical bottom disturbances to initiate and optimize sediment mobility.

Although sediment bypassing is used in some reservoirs throughout the world for storage capacity maintenance, this process is often timed to coincide with significant storm events that include substantial suspended sediment loads. A major restriction to sediment flushing at Crystal Lake Reservoir would be state and federal regulatory restrictions that would prevent rapid discharges of sediment downstream due to the significant environmental impacts that would likely occur. Therefore, sediment flushing does not warrant additional consideration as a dredging alternative for Crystal Lake Reservoir.

3.9.5 Sediment Dewatering Options

Generally, the most cost effective approach to dredge and dewater this volume of sediment is to hydraulically pump the sediment and water slurry into an earthen Sediment Dewatering Facility (SDF), with examples shown in Figure 3-43. Construction of SDFs are dependent on securing or leasing a sufficiently sized parcel of open land that is nearly level or gently sloping and is located outside of the floodplain, with no wetlands on the site. It is also desirable to be within one to two miles of the targeted dredging area with pipeline access and a minimum number of road crossings. Longer pumping distances are certainly feasible, but a booster pump would be required and higher dredging costs would be incurred as a result.

Figure 3-43
Examples of Earthen Sediment Dewatering Facilities (SDF)

If a suitable upland storage and dewatering site cannot be secured, the use of alternative dewatering methods such as geotextile tubes (see photos in Figure 3-44) or an on-site mechanical dewatering system (e.g. centrifuges, clarifiers, belt presses, etc.) would be required. Both of these alternative dewatering approaches are generally more expensive than constructing a suitably sized earthen dewatering facility. However, in the event an upland sediment dewatering facility (SDF) site cannot be acquired or leased, these alternative dewatering approaches may be required. The dewatered material would have to be hauled to one or more locations, preferably for beneficial reuse. Geotextile tubes are large geotextile fabric tubes approximately 60 feet in circumference and 100 feet or more in length. They are generally placed in a series and hydraulically filled with dredged sediment and water. Depending upon the total volume proposed to be dredged, a sufficient amount of land area would be required for a short period of time to allow the excess water to discharge back into the waterway and allow the dredged sediment to dry and consolidate within the tubes for eventual off-site hauling.

Figure 3-44
Examples of Dewatering Alternatives to SDF: Geotextile Tubes and Mechanical Dewatering Systems

If sufficient land area does not allow the temporary and potentially repeated placement of geotextile tubes, an onsite mechanical dewatering system can be used on a smaller land area to rapidly dewater the sediment for temporary stockpiling and subsequent off-site hauling. A similar dewatering system was recently utilized in Delavan, WI to dewater approximately 45,330 cubic yards of sediment within a 100 foot by 200 foot area behind a Town Fire Station. The dewatered sediment was stock piled and then hauled to an approved soil placement site on a daily basis since available stock pile space was limited. The volume of the sediment as measured within the lake was reduced by nearly 40 percent after being dewatered, which greatly reduced the number of truck loads required.

3.9.6 Estimates of Probable Cost

The following estimates of probable cost have been developed for three project options with hydraulic dredging and conventional dewatering with an earthen sediment dewatering facility and have been based on other similar projects that have been completed. More accurate costs can be determined prior to actual project implementation by requesting bids from several appropriate contractors.

Table 3-27

Estimate of Probable Costs – Hydraulic Dredging and Sediment Dewatering Facility

Sediment Removal Work Task	Quantity	Estimated Cost
Hydraulic Dredging – ($12.00/CY)	163,333 - 806,665 CY	$1,960,000 - $9,680,000
Dredge and Pipeline Mobilization	1 LS	$200,000 - $300,000
Construct Sediment Dewatering Facility (20 to 100 acres)	1 LS	$600,000 - $3,000,000
Site Grading and Reclamation after Dredging	1 LS	$100,000 - $400,000
Subtotal		$2,860,000 - $13,380,000
Contingency (~10%)		$286,000 - $1,338,000
Subtotal incl. Contingency		$3,146,000- $14,718,000
Engineering, Permitting (~15%)		$ 472,000 - $2,208,000
Total Estimated Cost for Dredging and Dewatering		$ 3,618,000 - $16,926,000 ($33,900 to $35,700 per acre-foot)

Table 3-28

Estimate of Probable Costs – Hydraulic Dredging and Alternative Dewatering

Sediment Removal Work Task	Quantity	Estimated Cost
Hydraulic Dredging – ($12.00/cy)	163,333 - 806,665 CY	$1,960,000 - $9,680,000
Mobilization: Dredge, Pipeline, Dewatering Equip.	1 LS	$250,000 - $350,000
Dewatering (Mechanical or Geotextile Tubes; ($10.00/cy)	163,333 - 806,665 CY	$1,633,000 - $8,067,000
Hauling to Off-Site Facility* ($10.00/cy)	98,000 - 484,000 CY	$980,000 - $4,840,000
Subtotal		$4,823,000 - $22,937,000
Contingency (~10%)		$483,000 - $2,294,000
Subtotal incl. Contingency		$ 5,306,000 - $25,231,000
Engineering, Permitting (~15%)		$796,000 - $3,785,000
Total Estimated Cost for Dredging & Dewatering		$6,102,000 - $29,016,000 ($58,000 to $60,300 per acre-foot)

* Assumes 40% volume reduction from dewatering

Table 3-29
Estimate of Probable Costs – Wet Mechanical Dredging and Hauling

Sediment Removal Work Task	Quantity	Estimated Cost
Wet Mechanical Dredging – ($20.00 /cy)	163,333 - 806,665 CY	$3,266,660 - $16,133,300
Mobilization: Excav. Equip., Site Prep, Haul Roads	1 LS	$200,000 - $300,000
Temporary Material Storage, Dewatering, Loading	1 LS	$200,000 - $300,000
Hauling Sediment to Off-Site Facility ($10/cy)	163,333 - 806,665 CY	$1,633,330 - $8,066,650
Subtotal		$5,300,000 - $24,800,000
Contingency (~10%)		$530,000 - $2,480,000
Subtotal incl. Contingency		$5,830,000 - $27,280,000
Engineering, Permitting (~15%)		$875,000 - $4,092,000
Total Estimated Cost for Dredging & Dewatering		$6,705,000 - $31,372,000 ($62,700 to $66,200 per acre-foot)

* For wet excavation, hauling will be based on total dredging volume due to high water content

3.9.7 Recommended Dredging Approach and Estimated Cost

For estimating purposes, a 12-inch or 14-inch small portable hydraulic dredge working a double shift is assumed. Since an earthen sediment dewatering facility (SDF) is typically more cost effective than alternative dewatering methods such as geotextile tubes and mechanical dewatering systems, investigating the available of suitable land within proximity to the reservoir is recommended. The unit costs for this approach ranges from $33,900 to $35,700 per acre-foot of restored storage. Project duration is assumed to require 6 to 18 months to dredge 163,333 to 806,665 CY. The property will be needed for an additional season to allow for site improvements before dredging and/or restoration after the operation is complete.

The reservoir storage capacity that can be restored if one of the dredging options is implemented ranges from 101.2 to 500.0 acre feet or 32.9 to 162.9 million gallons. If reservoir dredging is determined to be a potentially feasible water supply enhancement alternative, it will be critical to complete a reservoir sedimentation survey and a detailed dredging feasibility study prior to planning and implementation.

3.9.8 Project Permitting

Dredging the Reservoir will require federal, state, and local permits. Federal and state permits will be required for all in-water activity and activity that affects regulated waters such as streams and wetlands. The federal and state permits are issued on a project basis, i.e. each agency's permit would address all regulated activity in the Reservoir, at the staging site, along the dredge pipeline route, and at the dewatering and placement sites. Local permits and approvals will be required for changes in land use or land disturbance activity. The local permits are issued on a site specific basis, i.e. there would be a permit for the staging area and another permit for each dewatering site(s) and placement site (if required).

3.9.9 Water Quality Management

Water quality impacts that may occur during dredging will be managed and controlled by placing floating turbidity barriers with extended curtain lengths (typically 10 to 20 feet) around the active dredging area and immediately upstream of the water supply intakes to isolate and contain dredging induced turbidity and any contaminants that may be temporarily re-suspended into the water column. It is likely that the hypolimnetic aeration system can remain functional for most or all of the dredging operation so that manganese is controlled by maintaining an oxygenated hypolimnion.

3.9.10 Dredging Impacts on Projected Potable Water Supply Deficits

The potential Crystal Lake Dredging was evaluated for improvements to the projected potable supply deficits. An increase in storage of 500 acre-feet was assumed; the degree of storage increase will need to be verified using refined sediment surveys and a dredging feasibility study. Table 3-30 shows the frequencies of drought levels under this proposed condition for year 2033 projected demands. There is an improvement of 2 months for Level 5 droughts, 7 months for Level 4, and 6 months for Level 3. The distribution of the Level 5 years is shown in Figure 3-45 There are no appreciable changes in drought frequencies and annual shortage distributions for year 2063. Table 3-31 and Figure 3-46 shows these results.

Table 3-30
Proposed Crystal Lake Dredging Impacts on Drought Level Frequency, Year 2033 Projected Demands

Drought Level	Existing Condition Frequency [%]	Proposed Condition Frequency [%]
Level 1: No Drought	6%	7%
Level 2: Mild Drought	38%	39%
Level 3: Moderate Drought	29%	29%
Level 4: Severe Drought	26%	25%
Level 5: Emergency	1%	1%

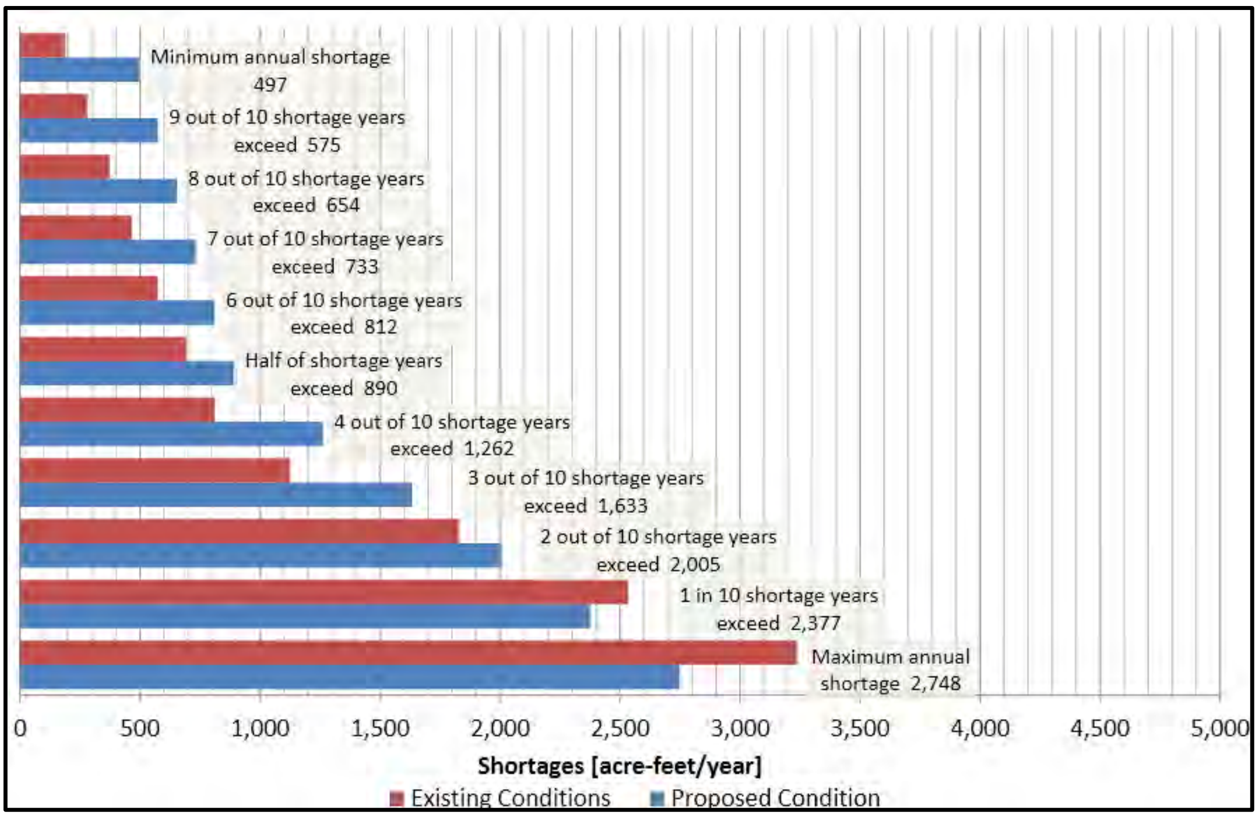

Figure 3-45
Distribution of Annual Potable Shortages using Proposed Crystal Lake Dredging and Year 2033 Projected Demands

Table 3-31
Proposed Crystal Lake Dredging Impacts on Drought Level Frequency, Year 2063 Projected Demands

Drought Level	Existing Condition Frequency [%]	Proposed Condition Frequency [%]
Level 1: No Drought	0%	0%
Level 2: Mild Drought	0%	0%
Level 3: Moderate Drought	3%	3%
Level 4: Severe Drought	50%	50%
Level 5: Emergency	47%	47%

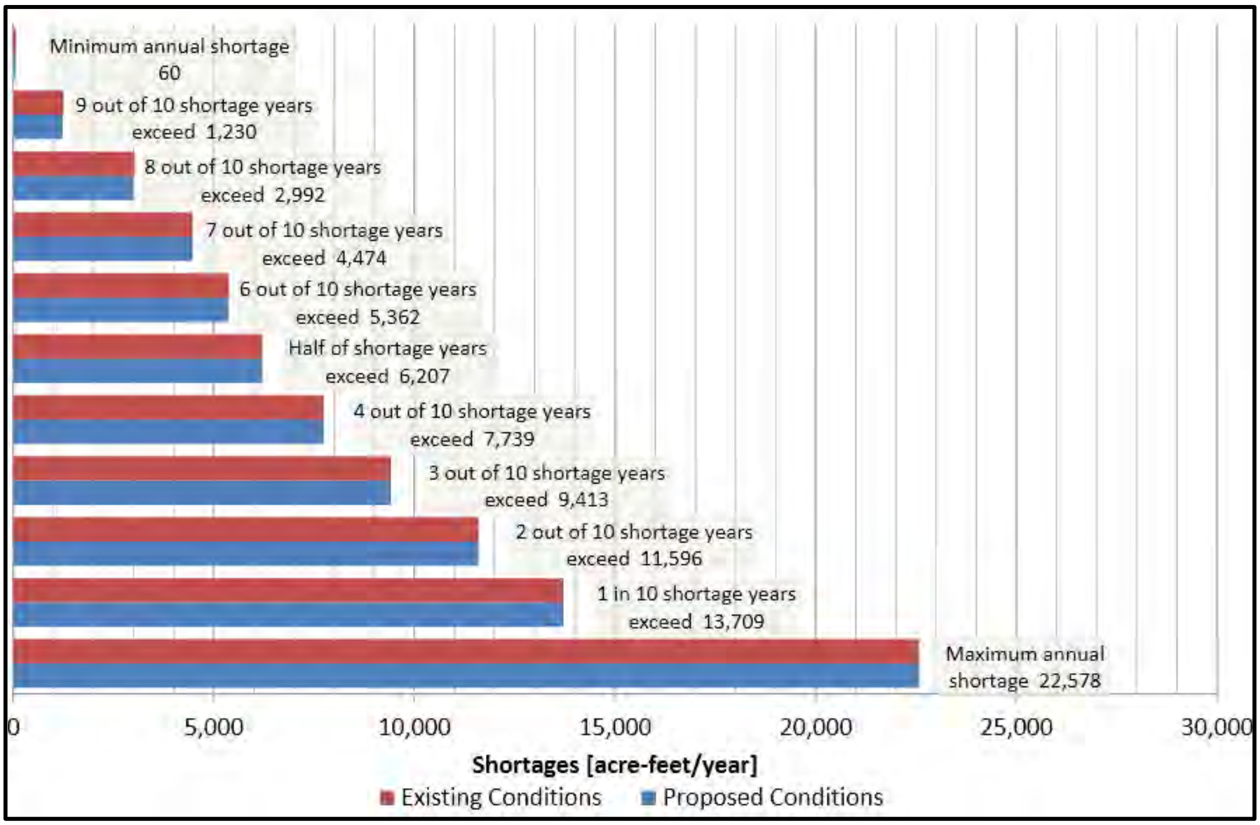

Figure 3-46
Distribution of Annual Potable Shortages using Proposed Crystal Lake Dredging and Year 2063 Projected Demands

3.10 Granite Springs and Crystal Lake Enlargement Evaluation

3.10.1 Summary

This section addresses the feasibility of enlarging the Crystal Lake and Granite Springs reservoirs to increase BOPU's overall raw water storage capacity. The combined Granite and Crystal Reservoirs have a storage capacity of around 8,700 acre-feet. A preliminary evaluation suggests that Crystal Lake Reservoir can be raised by approximately 14.5 feet and Granite Springs Reservoir can be raised by approximately 11.5 feet. Modifications would likely include: raising the crest of the dams, flattening the downstream face of both dams, removing the existing spillways and constructing newly designed spillways, raising and extending the saddle dike at Crystal Lake, raising and extending the release guide dike at Granite Springs, and moving or extending the outlet works at both projects. Additional discussions related to the individual projects are provided in the following sections.

Raising both Crystal Lake and Granite Springs Reservoirs will provide approximately 4,170 acre-feet of additional storage. Therefore, reservoir enlargements will only partially address the projected shortages and supplemental water supply will be needed. Raising either of the reservoirs by more than 14.5 and 11.5 feet, respectively, may also be technically feasible but will present additional (greater) technical challenges including the need for substantial modifications to the upper abutments of the concrete dams, significant embankment extensions at Crystal Lake spillway, and new saddle dams at both sites. At this point the enlargement configuration that balances cost and benefit considerations is not readily apparent. Instead of enlarging the reservoirs, it may be beneficial to consider a new water storage reservoir or enlargement of other facilities. Systematic evaluation of a range of alternatives, including a cost-benefit analysis should be considered to identify the most economic approach for addressing projected future water needs.

As part of assessing the feasibility of enlarging the two reservoirs, HDR collected and reviewed available information from the City and the SEO was collected and reviewed to identify baseline conditions of the two projects and identify if there are any outstanding dam safety issues that would affect the feasibility of modifying the dams and appurtenant features. However, important baseline information to adequately assess the feasibility of enlarging the dams and modifying appurtenant structures was available. Recommended analyses and investigations necessary to establish the feasibility of enlarging Crystal Lake and Granite Springs Reservoirs and to adequately develop the concepts and estimate the cost of the modifications are provided in Section 3.10.4. Since adequate information is not available, concepts and cost estimates of potential modifications were developed based on assumed conditions and engineering judgment. These assumptions should be verified and the concepts adjusted as necessary.

During the review of the available information, a potential design deficiency was identified in the spillway system of Granite Springs Reservoir. Based on the drawings, the wall to the right of the spillway is at elevation 7,213.5 feet and the elevations of the top of the fuse plugs are 7,218.5 feet and 7,219.5 feet. This implies that water will flow as much as 5 feet over the top of the wall to the right of the spillway before the first fuse plug is breached. There are no identifiable erosion protection measures installed downstream of the wall and the foundation conditions of the spillway and surrounding areas are unknown, so an evaluation of the design of the spillway and fuse plug system by a qualified dam safety engineer is recommended.

3.10.2 Granite Springs Reservoir Enlargement Evaluation

Granite Springs Project Information and History

Granite Springs Dam is one of several facilities of the City water supply system and is located in Laramie County, Wyoming, approximately 20 miles west of the City. The reservoir is in Curt Gowdy State Park and is approximately 2 miles upstream of Crystal Lake Reservoir. The dam impounds water from Middle Crow Creek and a water delivery pipeline from Lake Owen.

Granite Springs Dam is a rubble masonry arch dam completed in 1904. The dam has a maximum height of about 90 feet, a constant 300 foot radius at the crest centerline and a vertical upstream face. The crest is approximately 420 feet long and 10 feet wide at elevation 7,210.5 feet. A 1.25-foot wide, 12-foot high parapet wall exists along the upstream side of the crest to an elevation of 7,222.5 feet. The dam creates a reservoir with a capacity of approximately 5,220 acre-feet.

The existing spillway consists of a stepped spillway with a grouted riprap stilling basin. The spillway crest elevation is 7,210.5 feet. East of the spillway are two fuse plugs with top elevations of 7,218.5 feet and 7,219.5 feet. The base elevations of the fuse plugs are 7,208.0 feet. A spillway release guide dike exists to the east of the fuse plugs. Key data for the project is summarized as follows:

- Downstream Hazard Description: Class I – High Hazard
- Dam Crest Elevation: 7,210.5 feet
- Dam Height: 90 feet
- Top of Parapet Wall Elevation: 7,222.5 feet
- Spillway Crest Elevation: 7,210.5 feet
- Normal Maximum Pool Elevation: 7,210.5 feet
- Probable Maximum Flood Pool Elevation: Unknown
- Reservoir Volume: 5,220 acre-feet

Documentation related to the project was collected from BOPU and from the SEO. Available information on the Granite Springs Reservoir is very limited. There is little to no information on the dam prior to 1983. The following documents were available for review:

- Engineering and Environmental Evaluation, Crystal Lake and Granite Springs Dams, Woodward-Clyde Consultants, December 1983.

- Contract Documents for Rehabilitation of Granite and Crystal Dams and Spillways, Harza Engineering Company, 1985.
- Inspection Report for Granite Springs and Crystal Lake Dams, Wyoming State Engineer's Office, May 2012.

The December 1983 report was a study to evaluate the structural and hydrologic concerns for both Crystal Lake Dam and Granite Springs Dam and to present possible solutions. The primary concern for both projects was their inability to safely pass flood flows larger than a 100-year flood. Solutions proposed by Woodward-Clyde Consultants included increasing the height of both dams and adding spillway capacity to both dams so that the probable maximum flood flows would not overtop the dams.

In 1985, a contract was completed by Harza Engineering Company (Harza) which included remediation for both Crystal Lake and Granite Springs Dams. For Granite Springs Dam the following items were completed:

- Adding a new 12-foot high parapet wall to the crest of the dam.
- Removal of the existing spillway and construction of a new spillway and fuse plugs.
- Modifications to the outlet works.

BOPU reported that the spillway at Granite Springs Dam was reconstructed in 2010 but the contract documents for this work were not available.

Engineering analyses and basis of design calculations supporting the 1985 and 2010 modifications were not available for review. Further analyses and investigations are necessary to assess if there are any outstanding dam safety issues that would affect the feasibility of modifying the dam and appurtenant features.

Granite Springs Site Visit

A site visit was conducted on May 6th, 2013. The HDR personnel conducting the inspection were Elena Sossenkina, P.E. and David Isley. Bill Ray, with the City of Cheyenne Board of Public Utilities (BOPU), accompanied HDR on the site visit. The weather was mostly sunny and about 60° F. During the site visit, HDR assessed the practical limits of raising the reservoir and looked for any safety concerns which may affect modifying the project features. Observations of the project features, O&M recommendations, and photos taken during the site visit are provided in Appendix 3-C.

Based on the topography of the site, the Granite Springs Dam can be raised by approximately 10 feet. Modifications would likely include: raising the crest of the dam, flattening the downstream face, removing the existing spillway and constructing newly designed spillways, raising and extending the release guide dike, and moving or extending the outlet works. Raising the reservoir is primarily limited by the dam and spillway abutments. Raising the dam by more than 10 feet may be feasible but will present greater technical challenges including the need for substantial modifications to the dam abutments. In addition, raising the dam by more the 10 feet, and subsequently raising the reservoir, will also include substantial modifications to the spillway abutments and new saddle dams around the project site. Based on information available, raising the reservoir by 10 feet at Granite Springs would provide an additional capacity of 1,879 acre-feet. Additional storage can be gained without increasing the height of the dam by adding length to the spillway and adding a spillway over the dam. The additional storage gained by modifying the spillways is estimated at 300 acre-feet. The total potential increase in storage for Granite Springs Reservoir is estimated at 2,180 acre-feet. The design concepts for raising the dam 10 feet and modifying the spillways are discussed below.

Granite Springs Enlargement Concepts

The design concepts discussed in this Section are for raising the dam by 10 feet to elevation 7232.5 and for the modifications to other site features. The dam raise is primarily limited by the site topography as discussed above. The assumptions being used to develop the design concepts are discussed within.

Hydrology

A hydrologic study was performed for Granite Springs Reservoir in 1983 by Woodward-Clyde Consultants. The peak inflow into Granite Springs Reservoir was determined to be 29,643 cubic feet per second (cfs). The dam and spillway were modified in 1985 but there was no design documentation available for review. It was assumed that the design outflow of the existing spillway system is the same as the peak inflow. Therefore, the assumption is being made that the design outflow for the new spillway is 29,643 cfs.

Dam Concept

The modifications for raising the dam consist of adding roller compacted concrete (RCC) to the crest and to the downstream face. The RCC will be an integral part of the existing masonry rubble dam. At any given cross section of the dam an RCC lift must be at least 10 feet in width for constructability. Within the new RCC structure, a 100-foot ogee crest spillway would be placed approximately centered between the abutments. The spillway allows for controlled flow over the dam and into the creek below. The elevation of the spillway crest is discussed in the next section. With the construction of the new RCC dam and spillway, the outlet works pipes and building would be moved downstream and modified so as to not interfere with the spillway flow.

Typical RCC gravity dams have a downstream slope of 0.5-0.8 horizontal to 1 vertical. The slope is a function of the stability and foundation of the dam. For this concept, a downstream slope of 0.7 horizontal to 1.0 vertical is used. The design concepts are shown on Figure 3-47 and Figure 3-48.

Spillway Concept

The spillway modification for Granite Springs Reservoir consists of: 1) removal of the existing spillway and fuse plugs, 2) construction of an RCC overflow spillway with an ogee weir crest, 3) construction of a training wall to direct flow towards the existing grouted riprap and spillway release channel, and 4) construction of a saddle dike adjacent to the spillway to raise the ground elevation to the top of the dam elevation.

The length of the new spillway would be 250 feet, which is limited by the site topography. The alignment of the ogee crest will be rotated counterclockwise slightly from the existing spillway/fuse plug alignment in order to butt up against higher ground. The design concepts for the new spillway are shown on Figure 3-47 and Figure 3-48.

The elevation of the spillway crest for both spillways was estimated using the equation for discharge over an uncontrolled overflow ogee crest. Based on a combined spillway length of 350 feet and a design outflow of 29,643 cfs, the elevation of the crest of the spillway is determined to be 7,222. This elevation includes 1.5 feet of residual freeboard. Elevation 7,222 is the new normal maximum pool level (NMPL). This increases the reservoir from elevation 7210.5 to elevation 7,222; an increase of 11.5 feet. Based on information available, placing the normal maximum pool level at elevation 7,222 results in an increase in storage capacity of an estimated 2,180 acre-feet.

There are other spillway designs which should be considered as more information becomes available. One option in particular would be to construct a labyrinth spillway in place of the RCC overflow. A labyrinth spillway would allow for additional storage capacity without raising the dam, but would be more costly than the RCC overflow. The labyrinth spillway would require a bedrock foundation and would be constructed entirely of conventional concrete.

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Figure 3-47
Granite Springs Reservoir Enlargement Concepts Overview

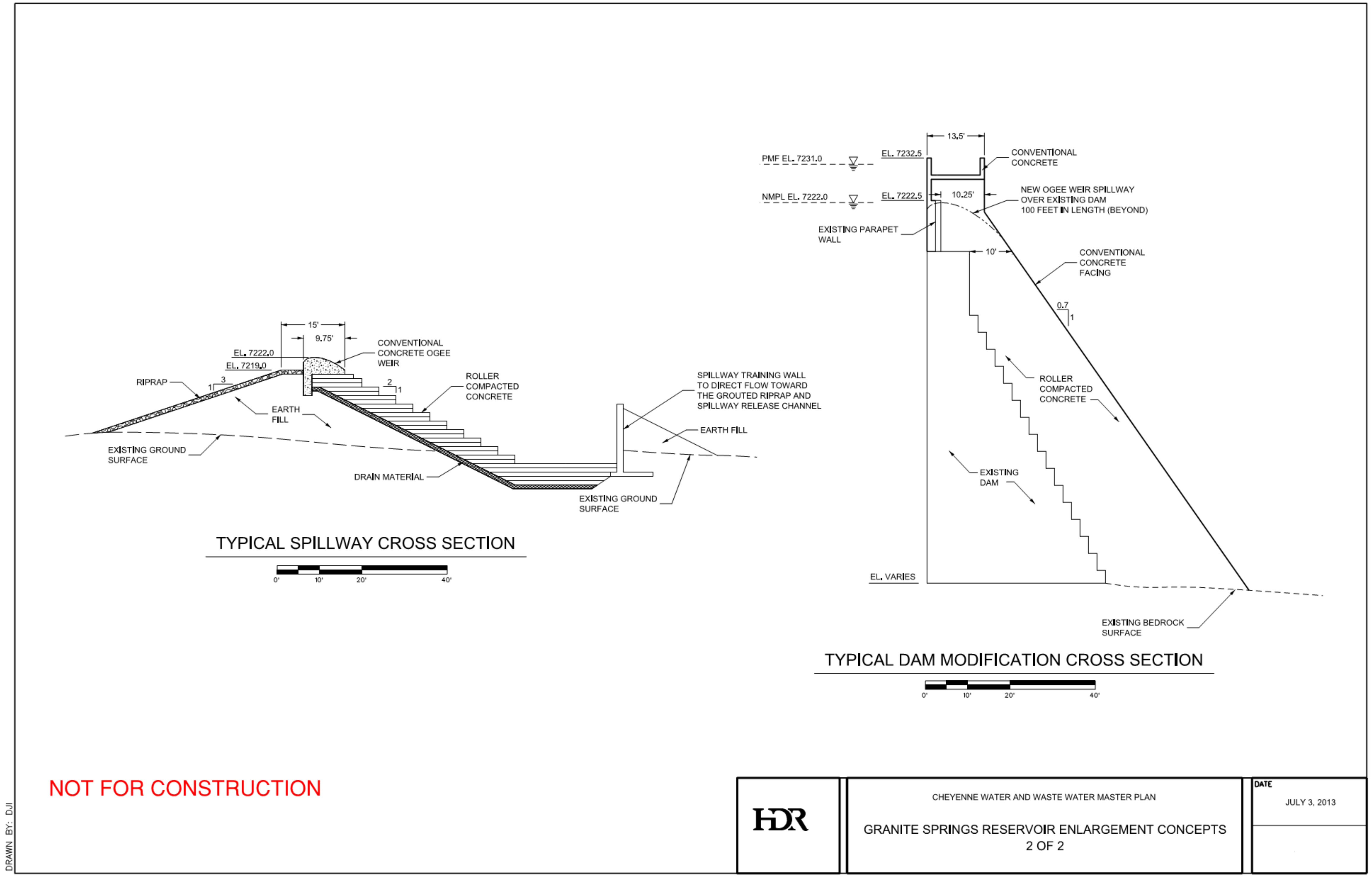

Figure 3-48
Granite Springs Reservoir Enlargement Cross Sections Concepts

Planning Level Cost Estimates

Planning level cost estimates for enlarging Granite Springs Reservoir are provided in Table 3-32. These costs estimates are based on 2013 dollars.

Table 3-32
Planning Level Cost Estimate for Enlarging Granite Springs Reservoir

Item	Quantity	Units	Unit Price	Extension
Major Construction Features				
Roller Compacted Concrete (RCC)	20,400	CY	$80/CY	$1,632,000
Conventional Concrete	4,000	CY	$370/CY	$1,480,000
Earth Fill	12,300	CY	$10/CY	$123,000
Drainage Materials	700	CY	$40/CY	$28,000
Outlet Works Modifications	1	LS	$ 650,000	$ 650,000
Subtotal				**$3,913,000**
Planning Level Direct Construction Costs (PLDC)				**$3,913,000**
Mobilization/Demobilization @ ~10% of PLDC				$391,000
Unscheduled Items @ ~15% of (PLDC + Mobilization)				$646,000
Construction Contingencies at ~20% of (PLDC + Mobilization+ Unscheduled Items)				$1,000,000
Subtotal				**$5,950,000**
Planning Level Estimate of Construction Cost (PLCC) (July 2013)				**$5,950,000**
Project Engineering and Construction Management Costs				
Engineering Design @ ~10% of PLCC				$594,000
Construction Management @ ~12% of PLCC				$713,000
Permitting @ ~ 4% of PLCC				$238,000
Subtotal				**$1,545,000**
Planning Level Estimate of Project Costs (July 2013)				**$7,484,000**
Additional Reservoir Storage Volume (acre-feet)				**2,200**
Additional Storage Cost (per acre-feet)				**$3,400**

Estimated Design and Construction Schedule

Table 3-33 provides an estimated design and construction schedule for the Granite Springs modifications discussed within this report.

Table 3-33
Granite Springs Enlargement Design and Construction Schedule

Item	Time Frame
Feasibility Studies and Design	9 - 24 months
Permitting	
Preliminary Design	
Final Design	6 - 9 months
Bidding and Contractor Selection	3 months
Construction	2 years
Total Estimated Design and Construction Time Frame	**3.5 - 5 years**

3.10.3 Crystal Lake Reservoir Enlargement Evaluation

Crystal Lake Project Information and History

Crystal Lake Dam is one of several facilities of the City water supply system. Crystal Lake Dam is located in Laramie County, Wyoming, approximately 20 miles west of the City. The reservoir is in Curt Gowdy State Park and is approximately 2 miles downstream of Granite Springs Reservoir. The dam impounds water from Granite Springs Reservoir, South Fork Middle Crow Creek and a water delivery pipeline from Lake Owen.

Crystal Lake Dam is a concrete arch dam constructed in about 1910. The dam has a maximum height of about 98 feet, a constant 93.5 foot radius at the crest centerline and a vertical upstream face. The crest is approximately 190 feet long and 7 feet wide at elevation 6,973.0 feet. A 1-foot wide, 10-foot high parapet wall exists along the upstream side of the crest to an elevation of 6,983.0 feet. The dam creates a reservoir with a capacity of approximately 3,410 acre-feet.

The existing spillway consists of an ogee weir and a stilling basin similar to a Bureau of Reclamation Type IV stilling basin. The ogee crest elevation is 6,971.0 feet. Adjacent to the spillway are three fuse plugs with top elevations of 6,978.0 feet, 6,979.0 feet and 6,980.0 feet. The base elevation of the fuse plugs are all 6,971.0 feet.

A saddle dike exists on both sides of the spillway fuse plugs. The original saddle dike was approximately 23 feet high and 550 long (Woodward-Clyde Consultants, December 1983).

When the existing spillway was constructed, it was placed through the saddle dike. The existing spillway and fuse plugs total 370 feet in length. The saddle dike, as it exists now, is approximately 90 feet in length on either side of the spillway fuse plugs and serves as the right and left abutment of the southern and northern fuse plugs, respectively. Key information for Crystal Lake Dam is summarized as follows:

- Downstream Hazard Description: Class I – High Hazard
- Dam Crest Elevation: 6,973.0 feet
- Dam Height: 98 feet
- Top of Parapet Wall Elevation: 6,983.0 feet
- Spillway Crest Elevation: 6,971.0 feet
- Normal Maximum Pool Elevation: 6,971.0 feet
- Probable Maximum Flood Pool Elevation: Unknown
- Reservoir Volume: 3,410 acre-feet

Documentation related to the project was collected from BOPU and from SEO. Available information on the dams is limited. There is little to no information prior to 1978. The following documents were available for review:

- Portions of *Phase I Evaluation Report*, Crystal Lake Dam, U.S. Army Corps of Engineers, 27 March 1978.
- *Interim Report Engineering Evaluation,* Crystal Lake Dam, Woodward-Clyde Consultants, April 1983.
- *Engineering and Environmental Evaluation, Crystal Lake and Granite Springs Dams,* Woodward-Clyde Consultants, December 1983.
- *Contract Documents for Rehabilitation of Granite and Crystal Dams and Spillways*, Harza Engineering Company, 1985.
- Inspection Report for Granite Springs and Crystal Lake Dams, Wyoming State Engineer's Office, May 2012.

The complete 1978 Phase I Evaluation Report was not available for review. The SEO only had a small portion of the report on file. The two 1983 evaluations were in response to concerns brought up in the 1978 Phase I Evaluation Report. The April 1983 report evaluated concerns with Crystal Lake Dam including: 1) water seeps between the dam concrete and the abutment rock, 2) weathering and erosional deterioration of the abutment rock immediately downstream of the dam, 3) stability of the dam due to erosion of closely jointed rock that could severely erode if the dam were to overtop 4) significant deterioration of the upstream and downstream face of the dam, and 5) three radial cracks in the upper 15 feet of the dam affecting the internal stresses in the dam. Woodward-Clyde concluded that: 1) abutment remedial work was needed including grouting, installation of abutment drains, rock bolting and placement of mass concrete in areas of rock deterioration or where blocks of rock have fallen out, 2) the dam should not be allowed to overtop, 3) there was limited deterioration of the internal concrete, 4) no evidence of alkali-aggregate reactions or sulfate reactions were observed, and 5) Crystal Lake Dam has structural capacity to resist loadings imposed by hydrostatic forces resulting from the normal water level at minimum temperatures providing the radial cracks in the top 15 feet of the dam can transmit arch forces across them, and that the cracks should be repaired.

The December 1983 report was a study to evaluate the structural and hydrologic concerns identified in the 1978 Phase I Evaluation Report for both Crystal Lake Dam and Granite Springs Dam and to present possible solutions. The primary concern for Crystal Lake Dam was its inability to safely pass any flows larger than the 100 year flood. Solutions proposed by Woodward-Clyde included increasing the height of the dam and adding spillway capacity so that the probable maximum flood flows would not overtop the dam.

In 1985, a contract was completed by Harza Engineering Company (Harza) which included remediation for both Crystal Lake and Granite Springs Dams. For Crystal Lake Dam the following items were completed:

- Removal of the existing 2.5-foot high parapet wall and replacing it with a new 10-foot high wall on the crest.
- Resurfacing of the entire downstream face and crest, and resurfacing of the upstream face down to elevation 6,945.0 feet.
- Installation of perforated PVC pipe downstream of the three existing radial cracks and placement of waterstops on the upstream side of the radial cracks.
- Removal of the existing spillway features on the right abutment of the dam.
- Construction of a new spillway and fuse plugs within the existing saddle dike to the right of the dam.
- Modifications to the outlet works.

In 2008, a contract was completed by Black & Veatch Corporation which modified the outlet works to Crystal Lake Dam and added additional pipeline downstream of the dam.

The modifications completed subsequent to the 1978 and 1983 evaluations may have addressed some of the concerns raised at that time. However, engineering analyses and basis of design calculations supporting these modifications were not available for review. In addition, there is no evidence that any remediation of the abutments at Crystal Lake Dam was completed as recommended in the April 1983 report, or that the abutments were evaluated and determined to be adequate in any subsequent studies. A future study which includes an analysis of the abutment conditions is recommended.

Crystal Lake Site Visit

A site visit was conducted by HDR on May 6th, 2013. The HDR personnel conducting the inspection were Elena Sossenkina, P.E. and David Isley. Bill Ray, with the City of Cheyenne

Board of Public Utilities (BOPU), accompanied HDR on the site visit. The weather was mostly sunny and about 60° F. During the site visit, HDR assessed the practical limits of raising the reservoir and looked for any safety concerns which may affect modifying the project features. Observations of the project features, O&M recommendations, and photos taken during the site visit are provided in Appendix 3-D.

Based on the topography of the site it is possible to raise Crystal Lake Dam by approximately 10 feet. Raising the dam by more than 10 feet may be technically feasible but will present greater technical challenges, including the need for substantial modifications to the upper abutments of the dam. In addition, raising the dam more than 10 feet, and subsequently raising the reservoir, will require modifications to other sight features including significant extensions of the spillway saddle dike, and new saddle dams around the project site. Based on information available, raising the reservoir by 10 feet at Crystal Lake would provide an additional capacity of 1,341 acre-feet. Additional reservoir storage can be gained without increasing the height of the dam by adding length to the spillway. In the area of the existing spillway, there appears to be enough area for an estimated 800-foot length spillway. The additional storage gained by lengthening the spillway is estimated at 650 acre-feet. The total increase in storage for Crystal Lake Reservoir is estimated at 1,990 acre-feet. The design concepts for raising the dam 10 feet and modifying the spillway are discussed below.

Crystal Lake Enlargement Concepts

The design concepts discussed are for raising the dam by 10 feet to elevation 6993.0 and for the modifications to other site features. The dam raise is primarily limited by the site topography as discussed above. The assumptions being used to develop the design concepts are discussed below.

Hydrology

A hydrologic study was performed for Crystal Lake Reservoir in 1983 by Woodward-Clyde Consultants. The peak inflow into Crystal Lake Reservoir was determined to be 36,812 cubic feet per second (cfs). The dam and spillway were modified in 1985 but there was no design documentation available for review. Since there is no design information available, HDR assumes that the design outflow of the existing spillway system is the same as the peak inflow. Therefore, the assumption is being made that the design outflow for the new spillway is 36,812 cfs.

Dam Concept

The modifications for raising the dam consist of adding roller compacted concrete (RCC) to the crest and to the downstream face. The RCC will be an integral part of the existing concrete dam. At any given cross section of the dam an RCC lift must be at least 10 feet in width for constructability. With the construction of the new RCC, the outlet works building and pipes are moved downstream.

Typical RCC gravity dams have a downstream slope of 0.5-0.8 horizontal to 1 vertical. The slope is a function of the stability and foundation of the dam. For this concept, a downstream slope of 0.7 horizontal to 1.0 vertical is used. The design concepts are shown on Figure 3-49 and Figure 3-50.

Spillway Concept

The spillway modification for Crystal Lake Reservoir consists of: 1) removal of the existing spillway and fuse plugs, 2) construction of an RCC overflow spillway with an ogee weir crest, and 3) construction of saddle dikes adjacent to the spillway to raise the ground elevation to the top of the dam elevation. The length of the new spillway is 800 feet, which is estimated by site topography. The design concepts for the new spillway are shown on Figure 3-49 and Figure 3-50.

The elevation of the spillway crest was estimated using the equation for discharge over an uncontrolled overflow ogee crest. Based on a spillway length of 800 feet and a design outflow of 36,812 cfs, the elevation of the crest of the spillway is determined to be 6,985.5. This elevation includes 1.5 feet of residual freeboard. Elevation 6,985.5 is the new normal maximum pool level (NMPL). This increases the reservoir from elevation 6,971 to elevation 6,985.5; an increase of 14.5 feet. Based on information available, placing the normal maximum pool level at elevation 6,985.5 would result in an increase in storage capacity of an estimated 1,990 acre-feet.

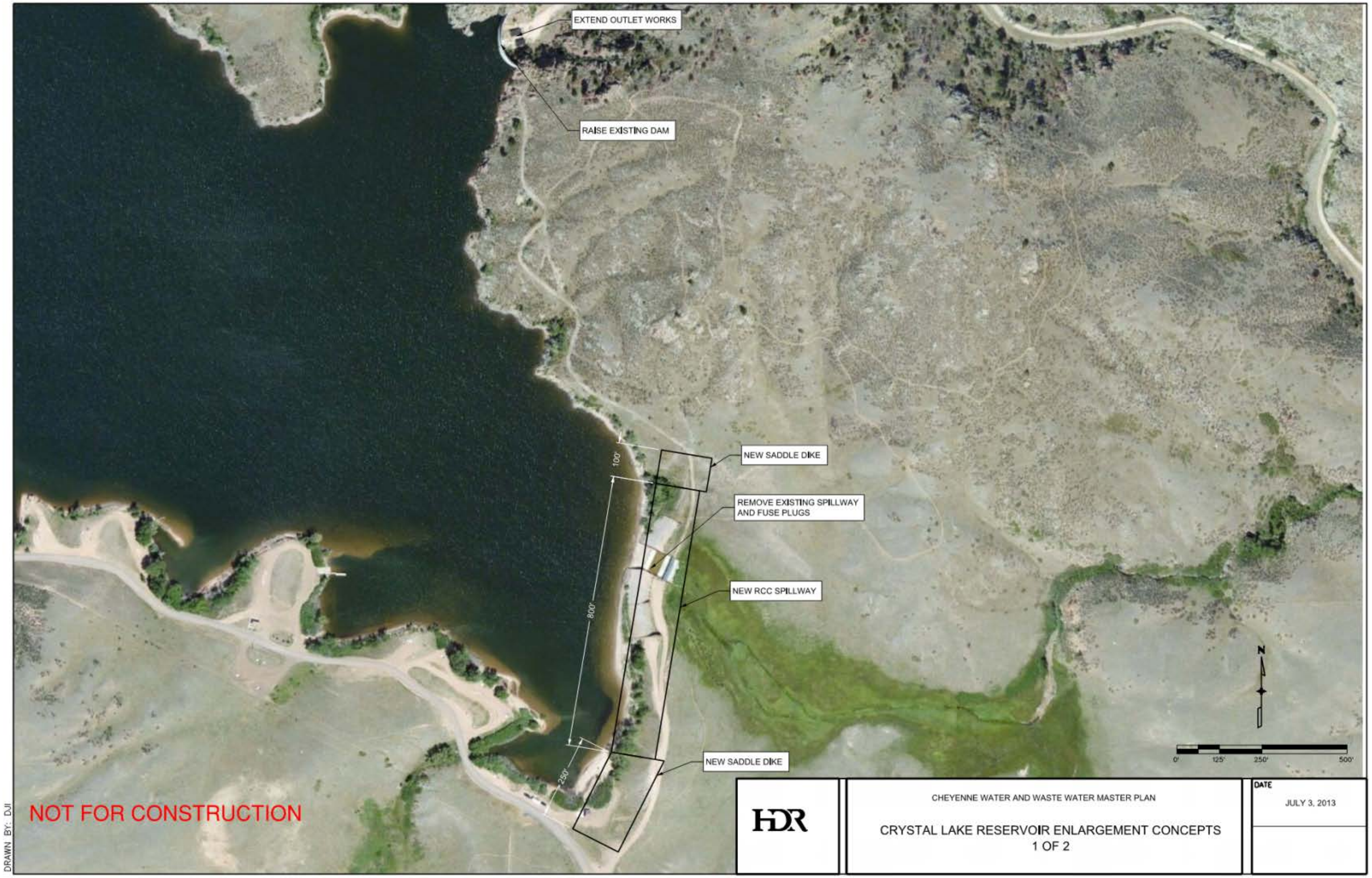

Figure 3-49
Crystal Lake Reservoir Enlargement Concepts Overview

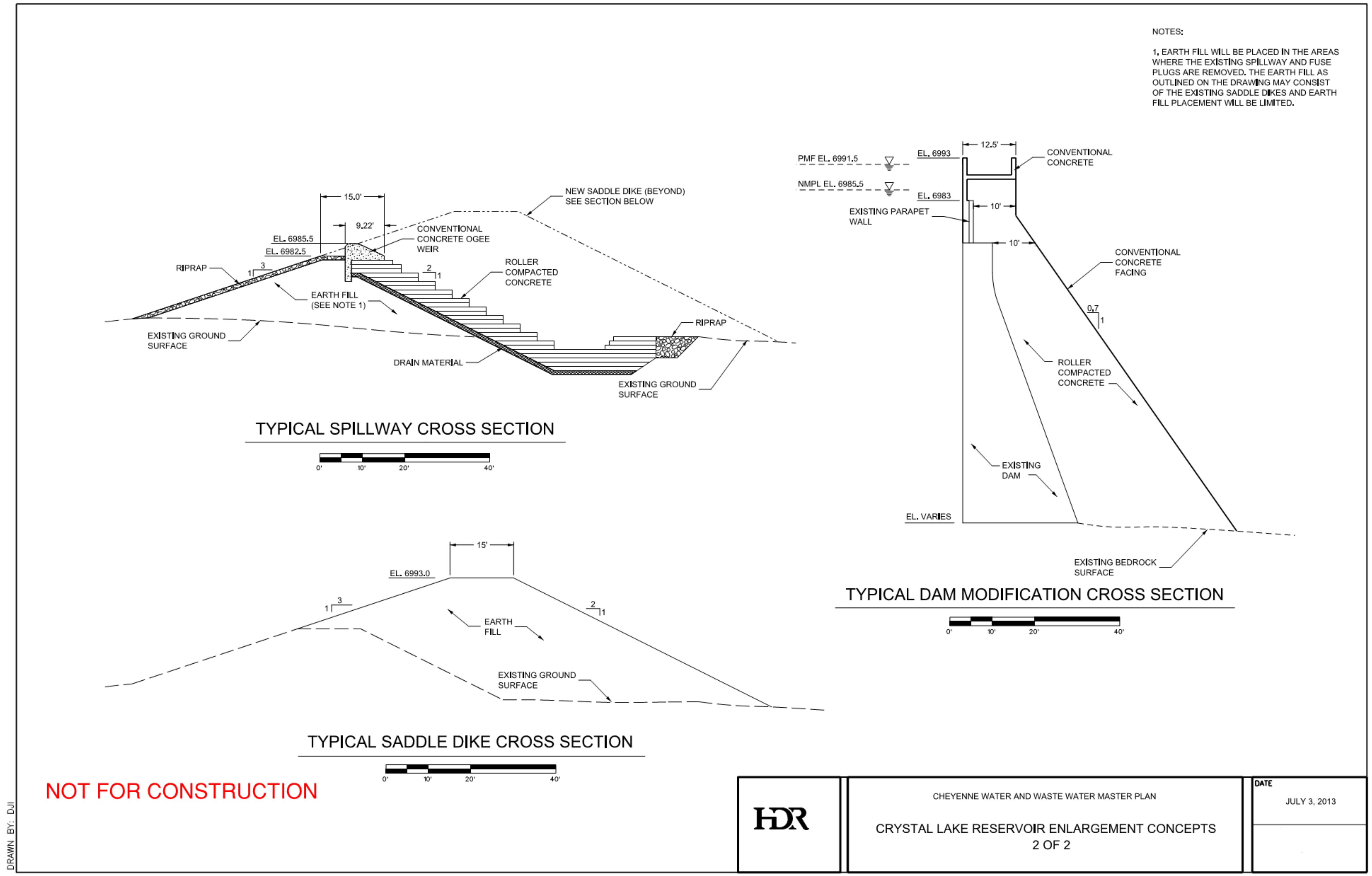

Figure 3-50
Crystal Lake Reservoir Enlargement Cross Sections Concept

Planning Level Cost Estimates

Planning level cost estimates for enlarging Crystal Lake Reservoir are provided in Table 3-34. These costs estimates are based on 2013 dollars.

Table 3-34
Planning Level Cost Estimate for Enlarging Crystal Lake Reservoir

Item	Quantity	Units	Unit Price	Extension
Major Construction Features				
Roller Compacted Concrete (RCC)	22,800	CY	$80/CY	$1,824,000
Conventional Concrete	2,400	CY	$370/CY	$888,000
Earth Fill	29,900	CY	$10/CY	$299,000
Drainage Materials	2,100	CY	$40/CY	$84,000
Outlet Works Modifications	1	LS	$1,400,000	$1,400,000
Subtotal				**$4,495,000**
Planning Level Direct Construction Costs (PLDC)				**$4,495,000**
Mobilization/Demobilization @ ~10% of PLDC				$450,000
Unscheduled Items @ ~15% of (PLDC + Mobilization)				$742,000
Construction Contingencies at ~20% of (PLDC + Mobilization+ Unscheduled Items)				$1,137,000
Subtotal				**$6,824,000**
Planning Level Estimate of Construction Cost (PLCC) (July 2013)				**$6,824,000**
Project Engineering and Construction Management Costs				
Engineering Design @ 10% of PLCC				$682,000
Construction Management @ 12% of PLCC				$819,000
Permitting @ 4% of PLCC				$273,000
Subtotal				**$1,774,000**
Planning Level Estimate of Project Costs (July 2013)				**$8,598,000**
Additional Reservoir Storage Volume (acre-feet)				**2,000**
Additional Storage Cost (per acre-feet)				**$4,000**

Estimated Design and Construction Schedule

Table 3-35 provides an estimated design and construction schedule for the Crystal Lake modifications discussed within this report.

Table 3-35
Crystal Lake Enlargement Design and Construction Schedule

Item	Time Frame
Feasibility Studies and Design	9 - 24 months
Permitting	
Preliminary Design	
Final Design	6 - 9 months
Bidding and Contractor Selection	3 months
Construction	2 years
Total Estimated Design and Construction Time Frame	**3.5 - 5 years**

3.10.4 Recommendations for Additional Studies

Following is a list of additional analyses and investigations necessary to adequately assess the feasibility of raising the dams. In addition, these studies will be integral to future project design.

- Hydrologic and hydraulic analyses of the projects to develop the inflow design floods for the reservoirs and to determine the capability of the current features to safely pass the probable maximum flood flows. (Estimated cost: $70,000 for both projects)
- Geotechnical and geophysical investigations of the foundation and abutments for the dams, spillways, fuse plugs, dikes and adjacent areas. (Estimated cost: $80,000 for both projects)
- Stability and stress analyses of the dams based on the current configuration and on updated hydraulic loads. (Estimated cost: $60,000 for both projects)
- Site specific seismic hazard assessment. (Estimated cost: $20,000 for both projects)

3.10.5 Lake Enlargement Impacts on Projected Potable Supply Deficits

The potential Granite Springs and Crystal Lake Enlargement was evaluated for improvements to the projected potable supply deficits. Storage was increased in both reservoirs based on the 10 foot dam raise. Table 3-36 shows the frequencies of drought levels under this proposed condition for year 2033 projected demands. There is an improvement of 6 months for Level 5 droughts, 35 months for Level 4 droughts, and 37 months for Level 3. The distribution of the Level 5 years is shown in Figure 3-51. There are minimal appreciable changes in drought frequencies and annual shortage distributions for year 2063. Table 3-37 and Figure 3-52 shows these results. Higher frequency shortages can be curtailed with the dam raises, but during lower frequency shortages there is limited available water to fill the expanded storage.

Table 3-36
Proposed Lake Enlargement Impacts on Drought Level Frequency, Year 2033 Projected Demands

Drought Level	Existing Conditions Frequency [%]	Proposed Conditions Frequency [%]
Level 1: No Drought	6%	12%
Level 2: Mild Drought	38%	40%
Level 3: Moderate Drought	29%	25%
Level 4: Severe Drought	26%	22%
Level 5: Emergency	1%	1%

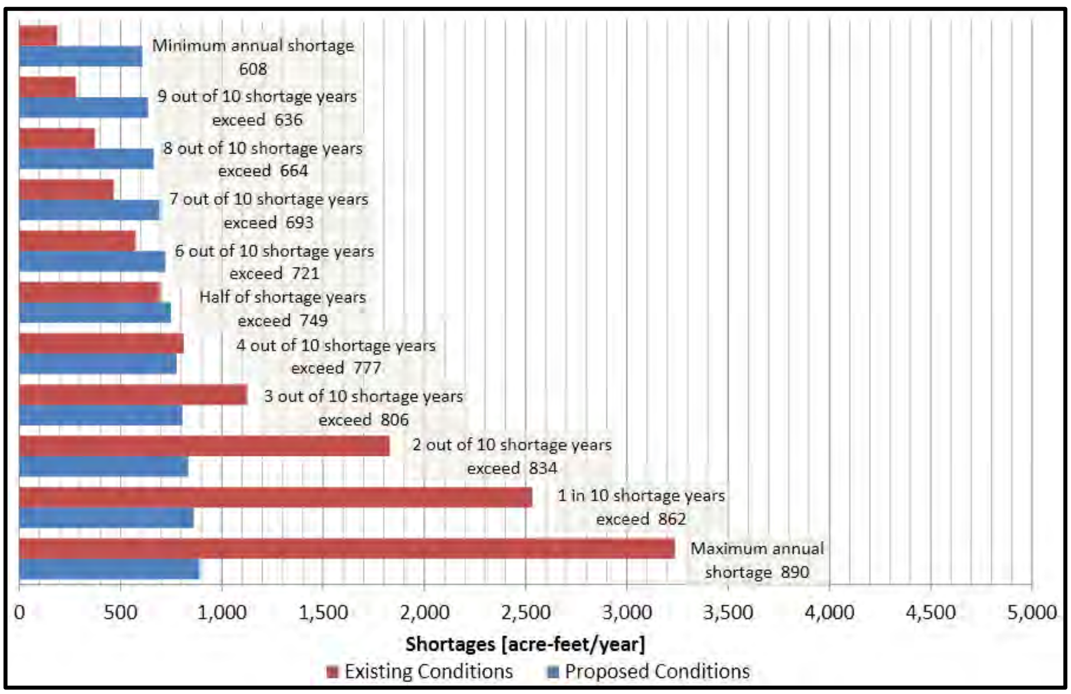

Figure 3-51
Distribution of Annual Potable Shortages using Proposed Lake Enlargement and Year 2033 Projected Demands

Table 3-37
Proposed Lake Enlargement Impacts on Drought Level Frequency, Year 2063 Projected Demands

Drought Level	Existing Conditions Frequency [%]	Proposed Conditions Frequency [%]
Level 1: No Drought	0%	0%
Level 2: Mild Drought	0%	0%
Level 3: Moderate Drought	3%	3%
Level 4: Severe Drought	50%	50%
Level 5: Emergency	47%	47%

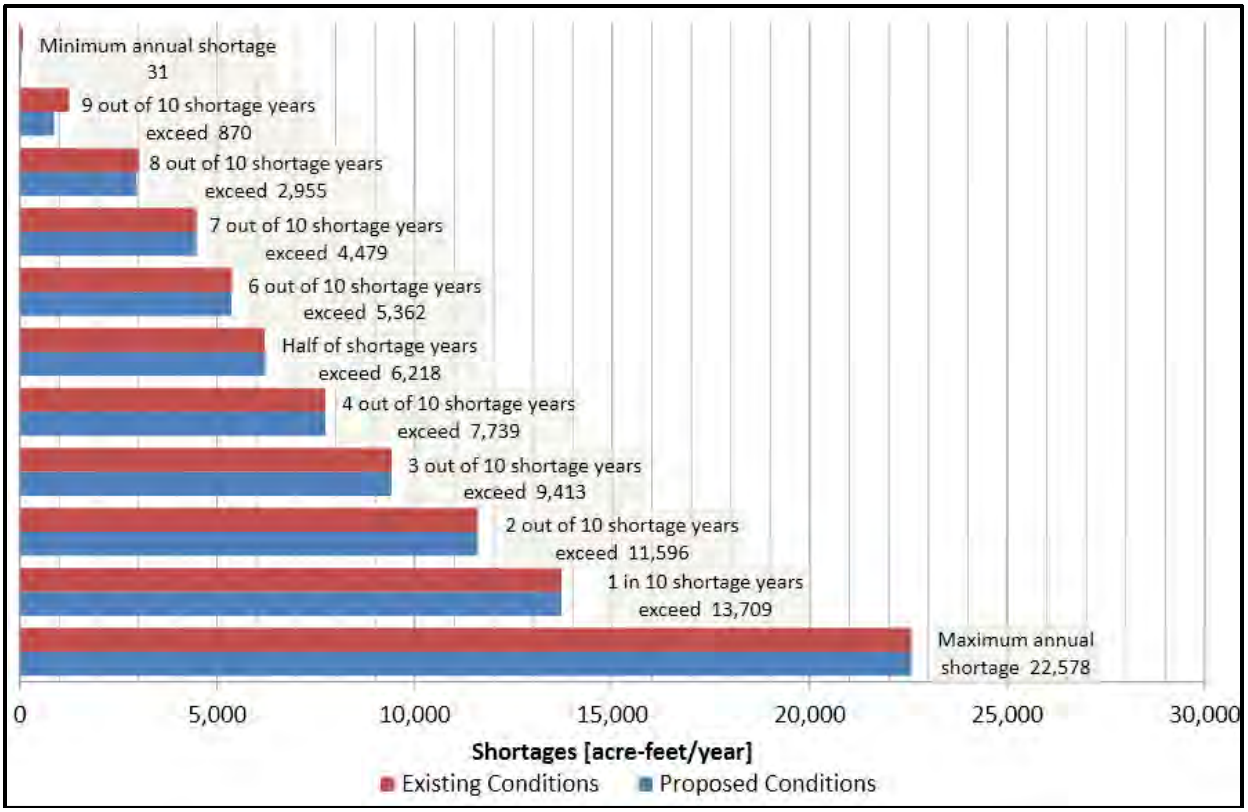

Figure 3-52
Distribution of Annual Potable Shortages using Proposed Lake Enlargement and Year 2063 Projected Demands

3.11 Recommendations

As the City's population and demands for water increase, additional sources of water will be required. Water demand deficit modeling predicts how the City may see water shortages start to appear in future years if supplies remain as they are. By 2033, shortages will be regular occurrences during drought periods. By 2063, shortages are predicted to be nearly continuous.

There is no single answer to ensuring the sustainability of the City's water supply. Additional surface water sources, increased storage, and increased groundwater supplies will all need to play a role, as well as conservation and innovative use on the demand side. Aquifer Storage and Recovery (ASR) projects may be needed to maintain production in the City's existing well fields. The City will be faced with many challenges in dealing with these issues. New surface water sources and storage are limited and very expensive, so the City will likely need to rely on groundwater to make up larger percentages of source water in future years.

One way to evaluate the options for obtaining additional water supplies is to determine the cost per ac-ft to develop the water. Based on the evaluations in this plan, the relative costs per ac-ft for the supply alternatives are shown in Table 3-38. These implementation costs are planning level only and will be refined as BOPU moves forward with studies and evaluations that can refine the requirements to implement each alternative. A summary of planning and implementation projects for the near term is shown in Table 3-39.

Table 3-38. Comparison of Supplemental Supply Alternatives by Cost

Option	Budget Cost	Production (ac-ft/yr)	Unit Cost ($/ac-ft)
North Crow Pipeline Alt 3	$1,200,000	490	$2,400
Granite Springs Raise	$7,484,300	2180	$3,400
Crystal Lake Raise	$8,597,500	1990	$4,300
New Well	$1,050,000	200	$5,300
Dredging (Hydraulic with SDF)	$3,618,000	100	$35,700
ASR	Unknown	Unknown	Unknown
New Surface Reservoir	Unknown	Unknown	Unknown

3.11.1 Planning Program Recommendations for Existing to Near Term (2013 to 2023)

The practicality of future implementation program items can be best addressed with feasibility, optimization, and evaluation studies. These studies cover future groundwater development and surface water resource storage, maintenance and expansion.

Groundwater Expansion

Future expansion and development of groundwater is recommended to occur primarily in the Bell well field on the Polo Ranch and the western Belvoir Ranch, with potential new well fields yet to be identified north, outside of the LCWCA, and northeast, inside the LCWCA.

The existing trend is toward increasing control and limitation of groundwater development near the City. In the event that BOPU develops additional groundwater supplies, such as the Belvoir Ranch, the administration of the well field production limits by the SEO is not known precisely. The SEO may consider that any future additional development in the Tertiary Aquifer would be included within the well field production limits. However, if BOPU were to develop groundwater from a different aquifer system, a technical argument can be made to the SEO that the new well field should not be included in the production limits set for the Federal, Bell, Happy Jack, and Borie well fields. Regardless, the expansion of existing well fields and/or the development of new groundwater resources capable of exceeding the established well field production limits will require that BOPU negotiate and apply to the SEO for an enlargement of the aggregate well field production limits. Well field optimization studies should be conducted to determine the number of wells that can be constructed and their locations within an identified well field area.

Aquifer Storage and Recovery

Approximately 10,000 ac-ft/yr may be currently available for aquifer storage and recovery (ASR). This amount will decrease until no significant excess water will be available in 2033, assuming no additional surface water sources are added to BOPU portfolio, so evaluation of ASR should be completed in the near term to take advantage of available water. Further analysis of ASR should continue to focus on the Happy Jack and Bell well fields, due to their favorable hydrogeology, location relative to the existing BOPU infrastructure, and lack of TCE contamination issues. Specific injection/infiltration sites should be identified and evaluated. The pilot study should focus on ASR injection wells until it can be shown that permitting of modified rapid infiltration basins (modified to create a higher permeability pathway through intervening clay layers) can be successful. The study should evaluate higher capacity injection, either by installing a new injection well or by retrofitting an existing well that can accommodate the required injection and pumping piping. Chemical compatibility must be included in the ASR study. The study should include a detailed evaluation of the capital costs required for full-scale implementation.

Dredging Crystal Lake

Crystal Lake dredging may restore between 100 and 500 acre-feet of storage at a cost of $6.7 to $31.4 million. A dredging project may extend between 6 to 18 months, which may affect reservoir operations depending on the dredging method selected. The estimate of storage restoration determined here is based on assumptions of sediment thickness and deposition. The actual potential for reservoir storage increase will differ from these assumptions. To determine whether reservoir dredging is a potentially feasible water supply enhancement alternative, it will be critical to complete a reservoir sedimentation survey and a detailed dredging feasibility study prior to planning and implementation. The sediment survey would selectively sample sediment depth measurements at elevations above the dead pool. A dual frequency fathometer could be applied to measure sediment thickness, assuming the sediment is not stratified. Sediment thickness transects would be performed in the upper end of the reservoir. The sediment survey would also include sediment core sampling and characterization of the sediment, such as grain size distribution and chemical contamination. Due to the relatively high costs of dredging compared to anticipated storage improvements, Crystal Lake dredging implementation is not recommended at this time. Selective dredging may be appropriate for specific water quality or outlet work maintenance objectives.

Raising the Dams at Crystal Lake and Granite Springs Reservoirs

Further potential for storage expansion in Crystal Lake and Granite Springs reservoirs through dam height increase may be possible and cost effective. The age of both dams has implications on dam safety. Additional analyses and investigations necessary to adequately assess the feasibility of raising the dams and dam integrity and safety will be needed in order for BOPU to properly decide on subsequent implementation. The studies identified are:

- Hydrologic and hydraulic analyses of the projects to develop the inflow design floods for the reservoirs and to determine the capability of the current features to safely pass the probable maximum flood flows.
- Geotechnical and geophysical investigations of the foundation and abutments for the dams, spillways, fuse plugs, dikes and adjacent areas.
- Stability and stress analyses of the dams based on the current configuration and on updated hydraulic loads.
- Site specific seismic hazard assessment.

New Reservoir Storage

Further expansion of surface water storage may take the form of new reservoirs. A feasibility study to evaluate potential sites amenable to future dams will provide BOPU with information for decisions regarding new reservoirs. Each dam site would be assessed as to the potential for range of dam heights, storage and reservoir surface area. The same or similar analysis listed above would be part of the feasibility study. Hydrologic and operational analysis and modeling would be needed to assess the potential of fill and draft from new storage based on existing in-basin yields and Stage I/II flows. The degree of surface water losses from seepage and evaporation would be assessed as part of the operational review.

3.11.2 Implementation Program Recommendations for Existing to Near Term (2013 to 2023)

Implementation Program Recommendations consist of negotiations related to existing and future groundwater use and system reliability improvements. Three on-going projects from 2013 are retained in the near-term program, including on-going well field improvements, Little Snake Stage II collection mains, and hydropower on the raw water pipeline at Sherard WTP.

Groundwater Projects

Based on actual pumping rates in 2012 at individual wells, production at the Koppes #4 well exceeds the adjudicated water right. BOPU should file an enlargement for this well. The City should re-engage Dyno-Nobel in discussions that explore private-public partnership use of groundwater. The goal of these discussions would be to transfer control of groundwater withdrawals to BOPU so that the overall drawdown of the well field is under BOPU's management. Both these wells are a proven resource, and their proximity to the Sherard WTP would likely make this option economically favorable. Knowing the outcome of negotiations in the near-term will aid in planning other well fields and piping infrastructure.

Pipeline Projects

New pipeline options for delivery of North Crow Creek water to Sherard WTP, delivery of Federal and Bell well fields water to the King II tank, and bypassing Granite Springs and Crystal Lake reservoirs to deliver water to Sherard WTP adds reliability and flexibility to portions of the raw water collection system. The North Crow Creek Raw Water Collection System could provide 2 mgd for 100 days per year for additional potable treatment. The Federal and Bell well fields connection would provide flexibility and redundancy to the current system of water treatment and distribution. The Granite Springs and Crystal Lake Bypass Pipeline would permit the bypass of Stage I/II waters directly to Sherard WTP in the event that one or both of these reservoirs were rendered inoperable because of infrastructure failure or contamination. Prioritization of these three pipelines by operations staff argues for building the Crystal/Granite bypass first, the Federal and Bell well fields pipeline second, and the North Crow collection pipeline third. All three pipelines are scheduled for construction in the near term, with the option of altering the order of construction by BOPU if desired.

3.11.3 Implementation Program Recommendations for Mid Term (2023 to 2033)

In the 2033 planning period, potable supply shortages are projected during specific periods of drought. Development of additional wells is currently the recommended source for supplemental supply, although the results from the new reservoir evaluation may cause BOPU to consider a new reservoir in place of groundwater sources. For planning purposes, each well is assumed to produce 200 acre-feet per year using a 6-month pumping period. The worst year drought shortage is estimated at 8,772 acre-feet per year. Using the future supplemental well fields only during a Level 5 Drought Emergency would therefore require 44 additional wells to meet the peak drought year. If the future well fields were brought on-line prior to a Level 5 drought (for example, during years of Level 4 Severe Drought), the existing surface water resources would be extended as drought progresses. An additional supplemental supply of 5,000 acre-feet per year would be needed under this approach rather than a peak supply. This would be an additional 25 wells instead of 44 wells. Well development costs are roughly $700K to get the well started and another $350K to complete. These costs do not include pipeline expansions or operating and maintenance needs.

Based on available water, land ownership and proximity to existing infrastructure (and inferred lower costs), the options for new well fields are ranked as follows:

1. Expand the Existing Bell well field on the Polo Ranch. Explore the Lance/Fox Hills aquifer along with the High Plains aquifer.
2. Explore the Dyno-Nobel well field options.
3. Develop the Belvoir Ranch High Plains aquifer and explore the Lance/Fox Hills aquifer on the eastern portion of the ranch.
4. Develop the western Belvoir Ranch.
5. Develop other new well fields to the north, outside of the LCWCA.
6. Develop other new well fields to the northeast, inside the LCWCA.

One project that should remain on the horizon and be tracked for this time period is the Flaming Gorge Pipeline Project, which may bring water from Flaming Gorge Reservoir into eastern Wyoming and the front range of Colorado. Potential participants in this project are currently awaiting analysis of the availability of water from the project from the Bureau of Reclamation. This program could well carry over into the long term planning horizon depending on how it progresses. BOPU is a participant in the project with the objective of obtaining 5,000 to 10,000 acre feet of new annual supply between 2020 and 2060.

3.11.4 Implementation Program Recommendations for Long Term (2033 to 2063)

In the long term planning period, most months (63%) are projected to be a Level 5 Drought Emergency, with median annual shortages of 9,210 acre-feet per year. The potential additional well fields recommended for development in the midterm may serve to offset 20% to 40% of months with shortages. Reevaluation of this planning period in subsequent master planning efforts is recommended. Uncertainty in long term population forecasts, potential shifts in average annual snowpack and climate, and implementation projects which will occur over time will affect estimates of future water supply sustainability. Additional expansion and the development of surface water storage should be identified in future feasibility studies.

Table 3-39
Summary of Planning and Implementation Recommendations

Planning Period	Program	Estimated Cost [3]
Existing to Near Term (2013 to 2023)	**Planning**	
	Well field expansion into the Polo and Belvoir ranches (define sites for drilling, well depth, and well spacing)	$50,000 to $70,000
	ASR Pilot Study (Happy Jack and Bell well fields)	$75,000 to $100,000
	Safety of dams for evaluation of Crystal Lake and Granite Springs enlargement:	$225,000
	Feasibility of new surface reservoir storage	$40,000 to $80,000
	Implementation	
	Negotiate identified water right issue (Koppes #4)	$10,000
	Negotiate Private-public partnership groundwater agreements (Dyno-Nobel)	$10,000
	Federal and Bell well field connection pipeline and booster pump station	$3,920,000
	Crystal Lake and Granite Springs reservoirs bypass pipeline	$1,560,000[1]
	North Crow Creek raw water pipeline	$1,200,000[2]
Mid-Term (2023 - 2033)	**Implementation**	
	Construct additional 25 to 44 wells for use during drought years	$26,250,000 to $46,200,000
Long Term (2033 - 2063)	**Implementation**	
	Expand and develop additional surface water storage identified in feasibility studies	

(1) Cost shown is the mean cost of the available pipe alignments, providing some flexibility in the development of the final project to refine the details of the alignments and select the preferred pipeline location incorporating property and easement issues.

(2) Does not include potential costs for replacing existing pipe if condition warrants replacement.

(3) Estimates are in 2013 dollars

3.12 Capital Improvement Plan

From the recommended improvements in Section 3-11, a capital improvement plan was developed outlining the implementation phasing and cost of the source water supply and delivery projects.

The improvement projects are all assigned a capital improvement ID with the following format, Planning Period-System-Project Number:

- Planning Period-
    - 2013 – In Progress/Completed
    - NT – Near-term (2014-2023)
    - MT – Mid-term (2024-2033)
    - LT – Long-term (2034-2063)
- System-
    - WS – Water Supply
- Project Number
    - Sequential number for each project

3.12.1 Cost Estimating Assumptions

Cost estimates were developed for each of the capital improvement projects yearly from 2015 to 2023 and as a total cost for mid-term (2024-2033) projects. The current budget year 2014 and the cost estimates from the financial projections provided by BOPU were not changed. Cost estimates were not provided for the long-term projects since they too far in the future to be certain of their implementation or costs.

The cost estimates developed are order of magnitude costs to give an indication of probable cost to implement. It is normally expected that an estimate of this type would be accurate within +50% or -30%. A 30% design contingency was applied to the total construction costs and a 3.5% per year escalation rate was used to account for inflation.

3.12.2 Capital Improvement Plans by Planning Period

Table 3-40 and Table 3-41 present the near-term (2015-2023), mid-term (2024-2033), capital improvement plans for water supply and delivery, respectively. Table 3-40 includes 2013 projects for reference but those projects are not considered part of the near-term capital improvement plan as they are currently in progress or under construction. Prior to the pipeline improvement projects being implemented, the scope and sizing of each project should be verified via pre-design investigation and planning including field confirmations, hydraulic modeling, cost estimating and siting and/or alignment studies.

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Table 3-40
Capital Improvement Plan for the Near-Term for Raw Water Sources

Item #	CIP ID	Project	Adjusted Budget FY 2013	Proposed Budget FY 2014	Projection FY 2015	Projection FY 2016	Projection FY 2017	Projection FY 2018	Projection FY 2019	Projection FY 2020	Projection FY 2021	Projection FY 2022	Projection FY 2023	Near-term Expenditures Based on Year of Construction Dollars
1	2013-WS-1	Well Field Improvements	$810,160	$600,000	$621,000	$642,700	$665,200	$688,500	$712,600	$737,500	$763,300	$790,000	$817,700	$7,038,500
2	2013-WS-2	LS Stage II Collection Mains/Siphons	$1,096,176											$0
3	2013-WS-3	Sherard Plant Hydro-Electric Generator	$3,000,000											$0
4	NT-WS-1	Planning Well Field Expansion into the Belvoir and Polo Ranches					$80,300							$80,300
5	NT-WS-2	ASR Pilot Study (Happy Jack and Bell Well Fields)				$110,900								$110,900
6	NT-WS-3	Evaluate Safety of Dams at Crystal Lake and Granite Springs						$267,200						$267,200
7	NT-WS-4	Feasibility of New Surface Reservoir Storage					$91,800							$91,800
8	NT-WS-5	Negotiate Identified Water Right Issues (Koppes #4)			$10,700									$10,700
9	NT-WS-6	Negotiate Private-Public Partnership for Dyno-Nobel Well			$10,700									$10,700
10	NT-WS-7	Federal and Bell Well Field Pipeline and Booster Pump Station									$5,161,900			$5,161,900
11	NT-WS-8	Crystal Lake and Granite Springs Reservoirs Bypass Pipeline							$1,917,600					$1,917,600
12	NT-WS-9	North Crow Creek Raw Water Pipeline											$1,692,700	$1,692,700
		Total Projects	$4,906,300	$600,000	$642,400	$753,600	$837,300	$955,700	$2,630,200	$737,500	$5,925,200	$790,000	$2,510,400	$16,382,300
													Average Cost per Year (Over 10 Years)	**$1,638,200**

Table 3-41
Capital Improvement Plan for the Mid-Term for Raw Water Sources

Item #	CIP ID	Project	Cost Estimate 2024-2033
1	MT-WS-1	Belvoir Water Development (carry over from 2003 Master Plan)	$45,000,000
		Total Projects	$45,000,000
		Average Cost per Year (over 10 years)	**$4,500,000**

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Appendices

Volume 3 – Source Water Supply and Delivery

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Appendix 3-A
Surface Basin Yields

1. Introduction
   Figure 3A-1 shows the collection system. BOPU describes the collection system as follows[3]:

   Approximately 75 percent of the water used in Cheyenne comes from mountain streams in the Medicine Bow and Laramie Mountain Ranges.

   The majority of this water comes from a trans-basin trade system, known as Stage I/II. It is a three part system that moves water from one side of a mountain to another, trades water across a valley, and then pipes water across two mountain ranges to Cheyenne.

   In the first part of the system, BOPU collects water from streams west of the Continental Divide and transports the water to the east side of the Continental Divide by a tunnel. Two reservoirs, Hog Park and Seminoe Reservoirs, store the water until needed for trade purposes.

   The second part of the system trades water from Hog Park Reservoir and from Seminoe Reservoir for water in Rob Roy Reservoir. The trade exchanges water from the west side of the North Platte River Watershed for water on the east side.

   The third part of the system transports water from Rob Roy Reservoir to Granite Springs and Crystal Reservoirs. The water is piped by gravity down the Medicine Bow Mountains, across the Laramie River Valley and over the Laramie Mountains. Once over the top of Laramie Mountains, the water flows by gravity to Granite and Crystal Reservoirs.

   It's a vast, complex system. And it relies on the cooperation and assistance of many organizations such as the U.S. Forest Service, U.S. Bureau of Reclamation and the Wyoming State Engineer. The system also relies on snow.

   It takes snow in both the Sierra Madre Mountains and in the Medicine Bow Mountains to make the system work. Snow provides the water to collect, trade with and trade for.

   Snow and precipitation also provide water in the Laramie Mountains. This water [from snowmelt runoff and precipitation] flows into the oldest water resource for Cheyenne, Crow Creek. Today, Cheyenne collects water from Crow Creek in three reservoirs,

[3] BOPU, "Where does Cheyenne's water come from?", Cheyenne WY Official Website, http://www.cheyennecity.org/index.aspx?nid=1550. Accessed August 2013.

Crystal, Granite and [South Crow, Old North Crow, and] North Crow Reservoirs. Crow Creek provides approximately 25 percent of the water used in Cheyenne.

The remaining 25 percent of water used in Cheyenne comes from wells. Cheyenne has four well fields to the west and northwest of the City. These well fields contain 35 wells that deliver water to Cheyenne.

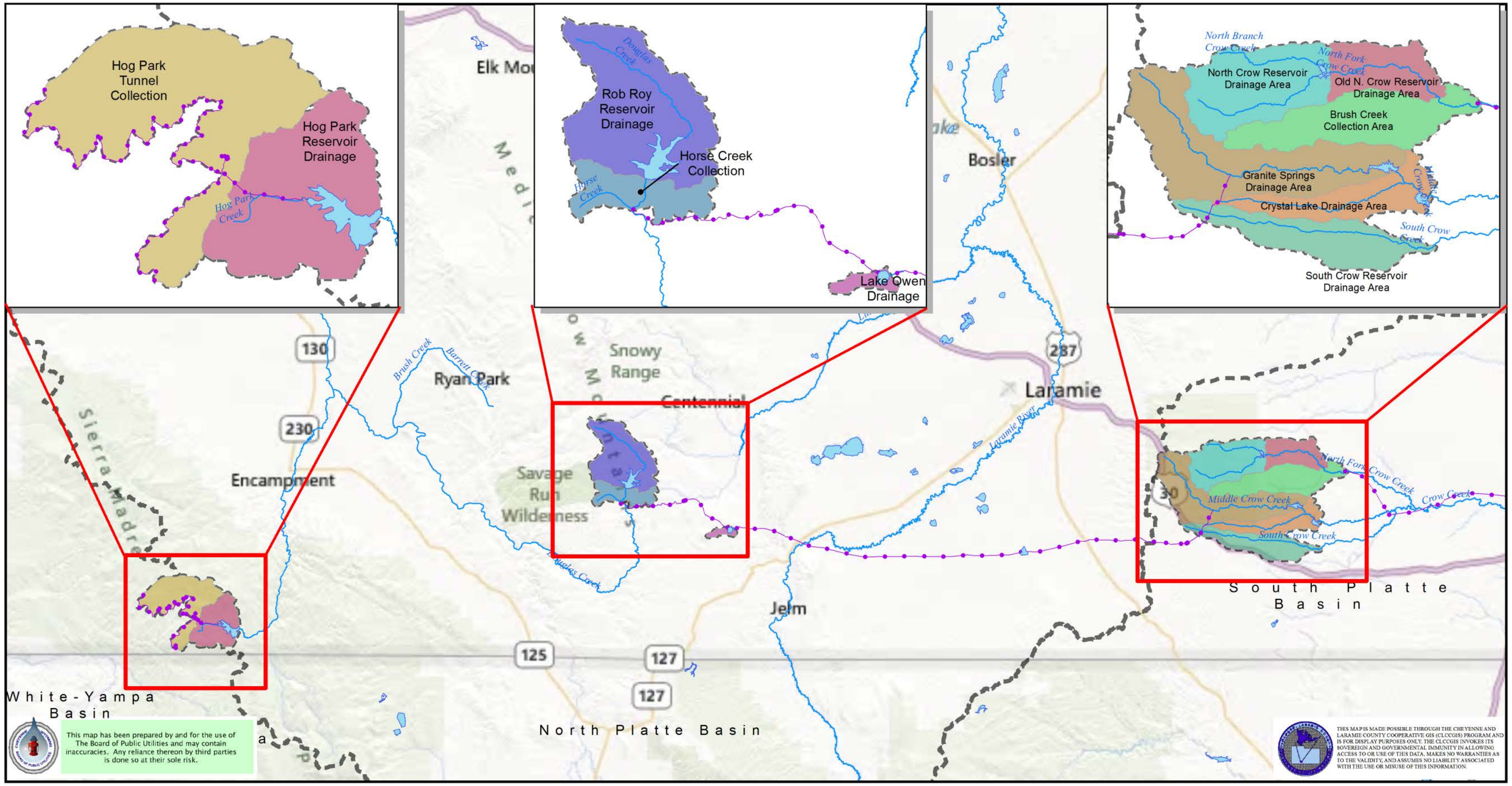
Flows from a 17 square mile collection area in the Yampa-White basin/Little Snake River watershed are brought into the North Platte basin using the Hog Park tunnel (HPT). These flows are stored in Hog Park reservoir, while natural flows from the 12 square mile reservoir drainage area (HPN) are passed through to Hog Park Creek. The Hog Park reservoir provides an exchange mechanism for import of water from the larger North Platte basin. This import is driven from the Stage I and II pipelines from the Rob Roy Reservoir. Rob Roy Reservoir captures a 21 square mile drainage area (RRN). The Stage I/II pipelines also can divert creek flows from Horse Creek (HCD) and other sources (PLD). The total drainage area of this creek collection system is 6 square miles.

Lake Owen has a drainage area of 1 square miles and additionally regulates the Stage I and II flows from Rob Roy. The natural lake is maintained at approximately a constant water surface. From Lake Owen, the Stage I and II pipeline transmits water to Granite Springs and Crystal Lake reservoirs. These reservoirs have a respective drainage area of 29 square miles (GRN) and 11 square miles (CRN), respectively. Lastly, several reservoirs store water for release in the North Crow and South Crow creek watersheds. These reservoirs, with estimated incremental drainage areas, are 61 square miles.

Figure 3A-2 shows the inflows, reservoirs, and water use locations in a schematic form as modeled in the Surface Water Supply System (SWSS) model. Inflow points in this schematic are:

- HPT – Hog Park Reservoir Tunnel
- HPN – Hog Park Reservoir Natural Inflow
- RRN – Rob Roy Reservoir Natural Inflow
- HCD – Horse Creek, diverted into the Stage I/II pipeline
- PLD – Other Douglas Creek tributaries, diverted into the Stage I/II pipeline
- LON – Lake Owen Natural Inflow
- GCN – Granite Springs Reservoir Natural Inflows
- CRN – Crystal Lake Reservoir Natural Inflows
- NCN – Upper North Crow Reservoir Natural Inflows
- ONN – Old North Crow Reservoir Natural Inflows
- BCD – Brush Creek Flows
- SCN – South Crow Creek Reservoir Natural Inflows
- Groundwater – contributions of aggregated well fields to supply potable demands

Figure 3A-1. Raw Water Collection System



Legend

Streams and Pipelines
- Rivers/Streams
- Ditches
- Pipelines

Drainage Area
- Basins
- Brush Creek Collection Area
- Crystal Lake Drainage Area
- Granite Springs Drainage Area
- Hog Park Reservoir Drainage
- Hog Park Tunnel Collection
- Horse Creek Collection
- Lake Owen Drainage
- North Crow Reservoir Drainage Area
- Old N. Crow Reservoir Drainage Area
- Rob Roy Reservoir Drainage
- South Crow Reservoir Drainage Area

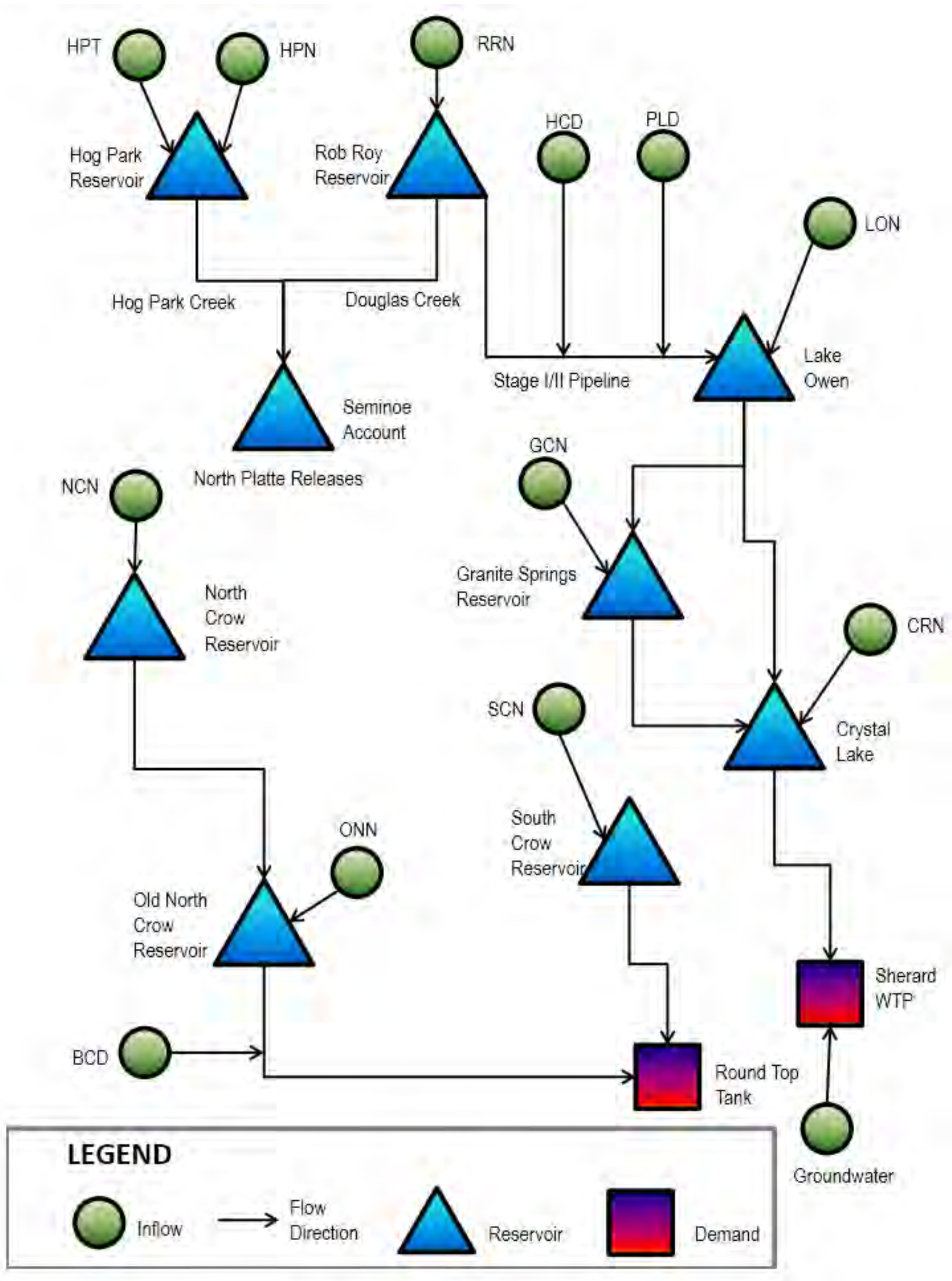
1 inch = 10 miles

N

Figure 3A-1
Surface Raw Water Collection System

Last Updated: 6/30/2013
2013 Water and Wastewater Master Plans

Figure 3A-2. Surface Water Supply System Model Schematic



2. Water Supply Synthesis

The 2003 Master Plan developed a methodology for calculating natural flows in the collection system and applied it to estimate flows for a period from water year 1933 to 2002. This same methodology is applied here to estimate additional flows for water years 2003 to 2012. This extended period covers a long-term and severe drought in the collection system area. This drought may be continuing at the time of this writing[4]. The 2003 Master Plan did not fully describe the natural flow methodology in that several regression and water budget equations were not documented. Additionally, documented equations developed for the Middle Crow drainage area were incorrect as either equation constants were rounded too much or had decimal values when log values were required. These equations are provided and corrected here. Unless stated otherwise, measurements are from BOPU's SCADA system.

2.1. Little Snake River and Hog Park Creek

Flows are estimated for the Hog Park Tunnel collection area and the direct drainage to Hog Park reservoir. Both areas are either measured directly or have sufficient measurements to estimate natural flows using a water budget approach.

2.1.1. Stage I/II Diversions to Hog Park Reservoir

A portion of the Yampa-White basin/Little Snake River watershed is collected and transferred to the Hog Park Creek watershed using the Hog Park Tunnel (HPT). The flows through the tunnel are directly measured by BOPU. Diverted flows may be limited based on the availability of natural flows and the available storage space in Hog Park Reservoir. During low flow years downstream water rights may prevent BOPU from diverting flows into the tunnel. Conversely, there may be times where Hog Park Reservoir has limited or no available storage; BOPU could shutdown the tunnel operations in this condition as well. The measured BOPU flows may underestimate the potential for tunnel flow diversions in the later case.

The potential HPT flows can be estimated using measured stream flows at the USGS gage 09253000 Little Snake River near Slater, CO. The calculated potential HPT flows are then compared to BOPU measured flow. Months where the calculated potential flows exceed the measured flow are indication that BOPU could have diverted more but choose not to based on the storage conditions in Hog Park Reservoir. The maximum of BOPU measured tunnel flow and the calculated potential flows are used for modeling.

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[4] National Drought Mitigation Center. 2013. U.S. National Drought Monitor, Available online at: http://droughtmonitor.unl.edu/. Accessed August 6, 2013.

The calculated potential tunnel flows are estimated as follows:

Q_{HPT}[April to July] = -16492631.38 / Q_{Slater}[April to July] + 26693.90844 Equ. 1
Q_{HPT}[May to July] = -12431890.71 / Q_{Slater}[May to July] + 25249.23198 Equ. 2
Q_{HPT}[June to July] = -4181879.837/ Q_{Slater}[June to July] + 18238.53906 Equ. 3
Q_{HPT}[May to June] = -9299707.339/ Q_{Slater}[May to June] + 22452.19591 Equ. 4
Q_{HPT}[April] = Q_{HPT}[April to July] - Q_{HPT}[May to July] Equ. 5
Q_{HPT}[May] = Q_{HPT}[May to July] - Q_{HPT}[June to July] Equ. 6
Q_{HPT}[July] = Q_{HPT}[May to July] - Q_{HPT}[May to June] Equ. 7
Q_{HPT}[June] = Q_{HPT}[June to July] - Q_{HPT}[July] Equ. 8

Cumulative flows at Slater in acre-feet are used in Equations 1 to 4 to estimate the potential tunnel flow volumes in acre-feet. In Equation 1, for example, the cumulative volume from April to July at Slater provides an estimate of the April to July potential tunnel diversions. Monthly potential diversion flows are calculated in Equations 5 to 8 by selective subtraction of the various cumulative volumes.

2.1.2. Hog Park Natural Inflow

The natural runoff above Hog Park reservoir is estimated using a water budget. Storage and releases from the reservoir are measured. Evaporation is estimated using the surface area of the reservoir and a presumed evaporation rate. Spills are estimated based on water surface elevation above the spillway invert.

The Hog Park reservoir natural inflow is estimated using Equation 9.

$Q_{HPN}[t] = (S[t+1] – S[t]) + E[t] + Q_{HogParkCreek}[t] + Q_{Spill}[t] – Q_{HPT}[t]$ Equ. 9

Where:

- $Q_{HPN}[t]$ is the Hog Park Reservoir natural inflow estimate
- (S[t+1] – S [t]) is the change in reservoir storage
- E[t] is the estimated evaporation from the reservoir. The evaporation rate is a set rate by month, shown in Table 3A-1.
- $Q_{HogParkCreek}[t]$ is the reservoir release to Hog Park Creek, not including spills. The flows are measured and supplied as part of the US Forest Service report. Minimum flows in the creek are 15 cfs. If the estimate for Q_{HPN} is more than the minimum flow then the excess is passed through the reservoir.
- $Q_{Spill}[t]$ is the estimated reservoir spill calculated in acre-feet per month as 1.9835*(60.22*X^2+135.43*X), where X is the water level elevation minus the spillway elevation of 8,455 ft amsl.
- $Q_{HPT}[t]$ is the measured Hog Park Tunnel inflows

Table 3A-1. Hog Park Reservoir Evaporation Rates

Month	Evaporation Rate [ft/day]	Month	Evaporation Rate [ft/day]
Jan	0.0006	Jul	0.0055
Feb	0.0006	Aug	0.0052
Mar	0.0013	Sep	0.0037
Apr	0.0032	Oct	0.0023
May	0.0039	Nov	0.0010
Jun	0.0050	Dec	0.0006

2.2. Douglas Creek Basin

The Douglas Creek collection area consists of direct drainage to Rob Roy reservoir and the Horse Creek and other creek collection areas that are diverted into the State I/II pipelines. The reservoir inflows are estimated from measured storage and release flows to the Stage I/II pipeline and Douglas Creek if available. Otherwise, natural inflows are estimated from long-term USGS stream gage information. The creek collection area is estimated using a drainage-area ratio to the reservoir inflows.

2.2.1. Rob Roy Reservoir Natural Inflow

The natural inflow drainage area above Rob Roy Reservoir is currently ungaged and approximately 21 square miles. The drainage area ranges from an elevation of 10,400 feet to 9,400 feet at the reservoir outlet. Monthly estimates of natural flows (RRN) into Rob Roy can be calculated using two methods depending on the available data. The preferred approach is to calculate natural inflow using a water budget. The water budget requires monthly measurements of reservoir storage and outflows. If measured data is not available, a regression equation using in-watershed USGS gages operated from 1955 to 1965 and long-term USGS stream gages can be used.

If measurements on the reservoir storage and releases to both Douglas Creek and the Stage I/II pipeline are available, the estimated inflows to the reservoir can be calculated using a water budget. The water budget for Rob Roy natural inflows is:

$$Q_{RRN}[t] = (S[t+1] – S[t]) + E[t] + Q_{DouglasCreek}[t] + Q_{StageI/II}[t] \qquad \text{Equ. 10}$$

Where:

- $Q_{RRN}[t]$ is Roy Rob natural inflow estimate
- $(S[t+1] – S[t])$ is the change in reservoir storage

- E[t] is the estimated evaporation from the reservoir. The evaporation rate is a set rate by month, shown in Table 3A-2.
- $Q_{DouglasCreek}$[t] is the reservoir release to Douglas Creek. The flows are measured are supplied as part of the US Forest Service report. Minimum flows in the creek are 5.5 cfs but may be more with spills and other reservoir operations
- $Q_{StageI/II}$[t] is the reservoir release to Lake Owen through the pipeline

Table 3A-2. Rob Roy Reservoir Evaporation Rates

Month	Evaporation Rate [ft/day]	Month	Evaporation Rate [ft/day]
Jan	0.0006	Jul	0.0056
Feb	0.0006	Aug	0.0052
Mar	0.0013	Sep	0.0037
Apr	0.0030	Oct	0.0023
May	0.0039	Nov	0.0010
Jun	0.0054	Dec	0.0006

If water budget data is not available a regression equation can be applied that links reservoir inflows to other long term USGS stream gage sites to the drainage area. The RRN water year annual (October to September) runoff total is estimated from the April to September volume measured at USGS Gage 06661000 Little Laramie River near Filmore, WY

("Little Laramie"). The USGS stops measuring at the Little Laramie gage site during winter, although the April to September volume is equally correlated with annual Rob Roy inflow volumes. This regression relationship is Equation 11. Next June (equation 12) and July (equation 13) RRN volumes, which typically make up 53% of the annual runoff, are estimated from the same months of Little Laramie volumes. The RRN volumes for the remaining months are estimated from monthly volumes at the Little Laramie gage (Equation 14). For months when this gage is inactive (e.g., during winter), a similar relationship using the Brush Creek gage (USGS 06622700 North Brush Creek near Saratoga, WY) is used. The estimates produced with Equation 14 have more uncertainty than estimates produced by equations 11 to 13. An adjustment factor is calculated using equation 15. The estimate of June and July is subtracted from the annual runoff estimate and compared to the estimate calculated with equation 14. The adjustment factor is then multiplied by equation 14 values.

Q_{RRN}[October to September]=6054.2275+ 0.29675185 * $Q_{LittleLaramie}$[April to Sep]	Equ. 11
Q_{RRN} [June] = 0.004215215 * ($Q_{LittleLaramie}$[June] ^ 1.4346282)	Equ. 12
Q_{RRN} [July] = 0.27503197 * ($Q_{LittleLaramie}$[July] ^ 0.93952316)	Equ. 13
Q_{RRN} [month] = 92.824931 + (0.20914491 * $Q_{LittleLaramie}$[month]) + (4.94E-06 * $Q_{LittleLaramie}$[month]^2) Or Q_{RRN}[month] = 255.72724 + (0.24881859 * $Q_{BushCreek}$[month]) + (3.00E-05 * $Q_{BushCreek}$[month])^2	Equ. 14
RRNAdjustment = (Q_{RRN}[Oct] + Q_{RRN}[Nov] + Q_{RRN}[Dec] + Q_{RRN}[Jan] + Q_{RRN}[Feb] + Q_{RRN}[Mar] + Q_{RRN}[Apr] + Q_{RRN}[May] + Q_{RRN}[Aug] + Q_{RRN}[Sep]) / (Q_{RRN}[October to September] - Q_{RRN}[June] - Q_{RRN}[July])	Equ. 15

2.2.2. Horse Creek collection system

Flows for the Horse Creek (HCD) and other creek collection area are estimated using a drainage-area ratio to the computed RRN flows. These ratios are shown in Equations 16 to 17.

Q_{HCN}[t] = 0.12240353 * Q_{RRN}[t] Equ. 16

Q_{PLD}[t] = 0.063412398 * Q_{RRN}[t] Equ. 17

2.2.3. Lake Owen

Lake Owen receives flow from the Stage I and II pipelines and direct drainage to the lake. The natural flows from the direct drainage (LON) is estimated using a water budget approach, as shown in Equation 18.

Q_{LON}[t] = (S[t+1] – S[t]) + Q_{in}[t] – E[t] – Q_{Spill}[t] – Q_{Out}[t] Equ. 18

Where:

- Q_{LON}[t] is Lake Owen natural inflow estimate
- (S[t+1] – S [t]) is the change in reservoir storage

- $Q_{in}[t]$ is Stage I and II pipeline inflows from Rob Roy reservoir and collection area
- $E[t]$ is the estimated evaporation from the reservoir. The evaporation rate is a set rate by month, shown in Table 3A-3.
- $Q_{Spill}[t]$ is the estimated reservoir spill. The spill is estimated as a function of water surface elevation above the spillway invert
- $Q_{out}[t]$ is the reservoir release to the Stage I and II pipelines toward Granite and Crystal reservoirs

Table 3A-3. Lake Owen Evaporation Rates

Month	Evaporation Rate [ft/day]	Month	Evaporation Rate [ft/day]
Jan	0.0006	Jul	0.0055
Feb	0.0006	Aug	0.0052
Mar	0.0013	Sep	0.0037
Apr	0.003	Oct	0.0023
May	0.0039	Nov	0.001
Jun	0.005	Dec	0.0006

2.3. Crow Creek Basin

Natural flow estimates are provided for reservoirs in the Middle Crow Creek, South Crow Creek, and the Granite and Crystal reservoir drainage areas. The flows for the Middle Crow and South Crow creek area are estimated from regression equations. The Granite and Crystal reservoir inflows are estimated from a combination of water budget and drainage-area ratios. While flows on the North Crow are not directly calculated, the inflows for the reservoirs in the North Crow watershed are estimated from flows on the Middle Crow Creek.

2.3.1. Middle Crow Creek

Natural flows for the Middle Crow Creek are estimated using regression equations. Precipitation measurements from the National Weather Service climate station at Hecla (Hecla 1E) is one of the inputs to the regression equations. The total number of rainfall, in inches, at Hecla 1E from October to June is used to estimate the Middle Crow Creek flow volumes in acre-feet for April to June (Equation 19). The same precipitation from April to June is also used to calculate May and June runoff volumes (equations 20 and 21). The April runoff volume is the difference between equations 19 and equations 20 to 21. The equations 19 to 22 are from the 2003 Master Plan methodology but are corrected for rounding and one incorrect coefficient.

The remaining months' Middle Crow runoff volume is based on the monthly rainfall amount and an autocorrelation from the previous month Middle Crow runoff. These relationships are shown in Equations 23 to 31.

Q_{MCrow}[April to June]=10^1.1054053 * P_{Hecla}[October to June]^2.1523599	Equ. 19
Q_{MCrow}[May] = 10^0.56828325 * P_{Hecla}[October to June]^2.3344576	Equ. 20
Q_{MCrow}[June] = 10^- 0.65575695 * P_{Hecla}[October to June]^3.242943	Equ. 21
Q_{MCrow}[April] = Q_{MCrow}[April to June] - Q_{MCrow}[May] - Q_{MCrow}[June]	Equ. 22
Q_{MCrow}[July] = (-8.5346153 + 0.37814792 * Q_{MCrow}[June]) + (-12.781858 + 5.1757027 * P_{Hecla}[July])	Equ. 23
Q_{MCrow}[August] = (3.2382459 * (Q_{MCrow}[July]^0.65842365)) + (-54.174401 + 27.873879 * P_{Hecla}[August])	Equ. 24
Q_{MCrow}[September] = (12.120377 * (Q_{MCrow}[Aug]^ 0.48275104)) + (-67.31534 + 43.196843 * P_{Hecla}[September])	Equ. 25
Q_{MCrow}[October] = (20.544347 * (Q_{MCrow}[September] ^0.43911833)) + (-64.480145 + 65.029898 * P_{Hecla}[October])	Equ. 26
Q_{MCrow}[November] = (21.477095 + 0.87142392 * Q_{MCrow}[October]) + (15.882355 + -26.820423 * P_{Hecla}[November])	Equ. 27
Q_{MCrow}[December] = (-19.409201 + 1.0372286 * Q_{MCrow}[November]) + (3.5817639 + -9.3162646 * P_{Hecla}[December])	Equ. 28
Q_{MCrow}[January] = (-15.873663 + 1.0157183 * Q_{MCrow}[December]) + (2.8623884 + -7.5277043 * P_{Hecla}[January])	Equ. 29
Q_{MCrow}[February] = (8.2174334 + 0.84505537 * Q_{MCrow}[January]) + (-2.5415994 + 6.4710368 * P_{Hecla}[February])	Equ. 30
Q_{MCrow}[March] = (67.267949 + 0.82294999 * Q_{MCrow}[February]) + (7.1851592 + -7.773718 * P_{Hecla}[March])	Equ. 31

Notes: Q_{MCrow} is the runoff volume of Middle Crow Creek in acre-feet. P_{Hecla} is the rainfall precipitation of the Hecla 1E station in inches.

2.3.2. South Crow Creek

The runoff for South Crow Creek is estimated using regression relationships with runoff of the Middle Crow Creek. The water year (September to October) annual estimate for South Crow uses the water year annual Middle Crow runoff estimate (Equation 32). Similarly, the April, May, and June runoffs are estimated from the same month Middle Crow runoff estimate (Equations 33 to 35). The runoff of these months are derived from snow-melt and highly correlated between the watersheds. Other months, affected by rainfall events, are less correlated. The total runoff from non-snow melt months is estimated from Equation 36. The individual monthly estimates for the other months are based on equation 37. Values from this last equation are further modified through multiplying using an adjustment factor calculated with Equation 38.

$Q_{SCrow}[Annual] = 4.6538293 * (Q_{MCrow}[Annual] ^\wedge 0.65577694)$	Equ. 32
$Q_{SCrow}[April] = -8.7690721 + (0.41684371 * Q_{MCrow}[April])$	Equ. 33
$Q_{SCrow}[May] = 0.66599255 + (0.25134282 * Q_{MCrow}[May])$	Equ. 34
$Q_{SCrow}[June] = -9.936115 + (0.25175903 * Q_{MCrow}[June])$	Equ. 35
Q_{SCrow}[October to March, July to September] = $Q_{SCrow}[Annual] - Q_{SCrow}[April] - Q_{SCrow}[May] - Q_{SCrow}[June]$	Equ. 36
$Q_{SCrow}[Month] = 0.2215 * (Q_{MCrow}[Month])$	Equ.37
$Adjustment_{SCrow} = (Q_{SCrow}[Oct] + Q_{SCrow}[Nov] + Q_{SCrow}[Dec] + Q_{SCrow}[Jan] + Q_{SCrow}[Feb] + Q_{SCrow}[Mar] + Q_{SCrow}[Apr] + Q_{SCrow}[May] + Q_{SCrow}[Aug] + Q_{SCrow}[Sep]) /$ (Q_{SCrow}[October to March, July to September])	Equ. 38

Notes: Runoff values for Q_{MCrow} and Q_{SCrow} are in acre-feet.

2.3.3. Upper North Crow Reservoir Inflow

The natural runoff estimate for the Upper North Crow Reservoir (NCN) is calculated using a drainage-area ratio to the Middle Crow Creek natural runoff. This relationship is provided in Equation 39.

$Q_{NCN}[t] = Q_{MCrow}[t] * 0.565482399$ Equ. 39

Notes: Runoff values for Q_{NCN} and Q_{MCrow} are in acre-feet.

2.3.4. Brush Creek and Upper North Crow Reservoir Inflow

The natural runoff estimates for the Upper North Crow Reservoir (ONN) diversion dam and the Brush Creek collection area are calculated using a drainage-area ratio with the Upper North Crow Reservoir natural runoff. These relationships are provided in Equations 40 to 41.

$Q_{BCD}[t] = Q_{NCN}[t] * 0.272687894$ Equ. 40

$Q_{ONN}[t] = Q_{NCN}[t] * 0.156641276$ Equ. 41

Notes: Runoff values for Q_{NCN} and Q_{MCrow} are in acre-feet.

2.3.5. South Crow Creek Reservoir Inflow

The natural runoff estimate for the South Crow Creek Reservoir (SCN) is calculated using a drainage-area ratio to the South Crow Creek natural runoff. This relationship is provided in Equation 42.

$$Q_{SCN}[t] = Q_{SCrow}[t] * 0.9832917 \qquad \text{Equ. 42}$$

Notes: Runoff values for Q_{SCN} and Q_{SCrow} are in acre-feet.

2.3.6. Granite Springs and Crystal Lake Natural Inflow

The natural runoff volumes for Granite Springs and Crystal Lake reservoirs are estimated with a combination of water budget and drainage-area ratios. Limited flow measurements between the Granite Springs and Crystal Lake reservoirs are available, such as how Stage I and II flows are distributed. The water budget is applied to the combined Granite Springs and Crystal Lake drainage area (GRNCRN) as shown in Equation 43. The water budget uses the following factors:

- $Q_{GRNCRN}[t]$ is combined natural runoff for both Granite Springs and Crystal Lake reservoirs' drainage areas
- $(S_{GR}[t+1] – S_{GR}[t])$ is the change in the Granite Springs reservoir storage
- $(S_{CR}[t+1] – S_{CR}[t])$ is the change in the Crystal Lake reservoir storage
- $Q_{in}[t]$ is Stage I and II pipeline inflows from Lake Owen
- EGR[t] is the estimated evaporation from the Granite Springs Reservoir. The evaporation rate is a set rate by month, shown in Table 3A-4.
- ECR[t] is the estimated evaporation from the Crystal Lake. The evaporation rate is a set rate by month, and is the same as the Granite Springs Reservoir evaporation rates.
- $Q_{SpillCR}[t]$ is the estimated reservoir spill from Crystal Lake. The spill is estimated as a function of water surface elevation above the spillway invert. The estimated spill from Granite Springs Reservoir is not needed as this loss from Granite Springs will be recaptured by Crystal Lake.
- $Q_{out}[t]$ is the Crystal Lake reservoir release to the Stage I and II pipelines toward Sherard WTP.

Table 3A-4. Granite Springs and Crystal Lake Evaporation Rates

Month	Evaporation Rate [ft/day]	Month	Evaporation Rate [ft/day]
Jan	0.0023	Jul	0.0146
Feb	0.0023	Aug	0.0133
Mar	0.0033	Sep	0.0101
Apr	0.0070	Oct	0.0065
May	0.0098	Nov	0.0034
Jun	0.0115	Dec	0.0022

The combined runoff estimate is split between Granite Springs Reservoir and Crystal Lake using a drainage-area ratio. These ratios are provided in Equations 44 and 45.

$Q_{GRNCRN}[t] = (S_{GR}[t+1] - S_{GR}[t]) + (S_{CR}[t+1] - S_{CR}[t]) + E_{GR}[t] + E_{CR}[t] + Q_{SpillCR}[t] + Q_{out}[t] - Q_{in}[t]$	Equ. 43
$Q_{GRN}[t] = Q_{GRNCRN}[t] * 0.879606889$	Equ. 44
$Q_{CRN}[t] = Q_{GRNCRN}[t] * 0.120393111$	Equ. 45

3. Estimated Runoff for 2003 to 2012

The methodology described in Appendix 3A Item 2 was applied to estimate natural runoff for water years 2003 to 2012. The monthly runoff values are presented in Appendix 3A Item 3.1. The long-term average runoff from the previous 2003 Master Plan is presented along with a revised long-term average for water years 1933 to 2012. A recent drought from 2000 to 2008 has been reconstructed as part of these runoff estimates. The comparison of the recent drought to past droughts is described in Appendix 3A Item 3.2.

3.1. Runoff Estimates

Runoff estimates are provided in the following Tables 3A-4 to 3A-17 for:

- Potential Hog Park Reservoir Tunnel (HPT)
- Hog Park Reservoir Natural Inflow (HPN)
- Rob Roy Reservoir Natural Inflow (RRN)
- Horse Creek Flow (HCD)
- Other Douglas Creek Diversions (PLD)
- Lake Owen Natural Inflow (LON)
- Middle Crow Creek Flows

- South Crow Creek Flows
- Upper North Crow Reservoir Natural Inflows (NCN)
- Upper North Crow Reservoir Natural Inflows (ONN)
- Brush Creek Flows (BCD)
- Granite Springs Reservoir Natural Inflows (GRN)
- Crystal Lake Reservoir Natural Inflows (CRN)
- South Crow Creek Reservoir Natural Inflows (SCN)

Table 3A-4. Potential Hog Park Reservoir Tunnel Flow Data [ac-ft]													
Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
2003	0	0	0	0	0	737	757	5,585	10,389	1,033	0	0	18,501
2004	0	0	0	0	0	66	1,726	7,581	5,987	126	0	0	15,486
2005	0	0	0	0	0	657	565	6,561	11,998	1,504	0	0	21,285
2006	0	0	0	0	0	0	1,390	9,603	8,252	1,206	0	0	20,451
2007	34	0	0	0	0	36	1,875	10,247	3,932	311	0	0	16,435
2008	0	0	0	0	0	0	443	5,579	13,782	2,063	0	0	21,867
2009	0	0	0	0	0	0	727	6,531	13,271	1,960	75	7	22,571
2010	0	0	0	0	0	0	633	5,251	13,872	1,948	0	0	21,704
2011	30	0	0	0	0	0	855	5,934	14,652	2,539	0	0	24,010
2012	0	0	0	0	0	129	5,180	7,010	124	0	0	0	12,443
Mean (2003 to 2012)	6	0	0	0	0	163	1,415	6,988	9,626	1,269	8	1	19,475
Mean (1933 to 2002)	-	-	-	-	-	-	477	5,721	10,931	1,416	-	-	18,546
Mean (1933 to 2012)	1	0	0	0	0	20	594	5,879	10,768	1,398	1	0	18,662

Table 3A-5. Hog Park Reservoir Natural Inflow [ac-ft]													
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	690	686	742	746	717	727	760	6,168	6,807	1,241	779	617	20,680
2004	816	28	376	303	262	508	1,708	5,874	2,366	345	166	488	13,238
2005	663	684	597	593	569	599	1,746	10,191	7,951	1,640	206	116	25,556
2006	343	510	673	506	380	488	2,340	11,418	5,196	577	205	477	23,113
2007	880	743	608	522	527	879	2,395	7,173	2,644	354	285	322	17,332
2008	471	285	519	460	508	178	427	5,911	10,686	2,465	235	309	22,453
2009	179	348	432	426	327	520	1,143	9,981	10,311	1,786	160	165	25,779
2010	618	530	88	435	302	418	1,455	6,012	11,883	2,132	264	54	24,191
2011	478	494	603	441	451	528	1,027	5,193	16,881	7,458	422	331	34,308
2012	344	453	360	321	361	788	3,962	6,983	1,200	230	-	67	15,068
Mean (2003 to 2012)	**548**	**476**	**500**	**475**	**440**	**563**	**1,696**	**7,490**	**7,592**	**1,823**	**272**	**295**	**22,172**
Mean (1933 to 2002)	**252**	**259**	**251**	**253**	**223**	**324**	**1,517**	**6,599**	**6,811**	**1,086**	**235**	**204**	**18,013**
Mean (1933 to 2012)	**289**	**286**	**282**	**281**	**250**	**354**	**1,539**	**6,710**	**6,909**	**1,178**	**240**	**215**	**18,533**

Table 3A-6. Rob Roy Reservoir Natural Inflow [ac-ft]													
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	596	593	636	639	617	624	649	5,172	5,800	1,017	664	541	17,549
2004	692	714	758	745	723	729	945	5,207	3,213	362	683	800	15,571
2005	1,747	679	552	523	505	528	586	6,030	6,189	1,110	478	420	19,347
2006	561	593	629	604	579	594	1,162	8,457	6,126	501	632	619	21,057
2007	868	768	673	673	619	621	971	7,336	4,310	322	668	525	18,354
2008	790	749	570	593	553	570	561	2,769	9,454	2,422	863	538	20,431
2009	453	494	482	482	444	463	729	6,282	15,177	1,506	793	445	27,751
2010	169	141	143	124	114	114	287	712	25,503	3,281	212	96	30,898
2011	155	213	182	175	161	168	392	916	35,176	7,550	654	281	46,024
2012	697	824	697	643	651	697	2,629	6,492	1,477	189	512	480	15,987
Mean (2003 to 2012)	**673**	**577**	**532**	**520**	**497**	**511**	**891**	**4,937**	**11,243**	**1,826**	**616**	**475**	**23,297**
Mean (1933 to 2002)	**486**	**338**	**369**	**304**	**283**	**378**	**788**	**7,516**	**11,434**	**1,465**	**705**	**361**	**24,428**
Mean (1933 to 2012)	**509**	**368**	**389**	**331**	**310**	**395**	**801**	**7,194**	**11,410**	**1,510**	**694**	**375**	**24,287**

Table 3A-7. Horse Creek Natural Flows [ac-ft]													
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	73	73	78	78	76	76	79	633	710	124	81	66	2,148
2004	85	87	93	91	89	89	116	637	393	44	84	98	1,906
2005	214	83	68	64	62	65	72	738	758	136	59	51	2,368
2006	69	73	77	74	71	73	142	1,035	750	61	77	76	2,577
2007	106	94	82	82	76	76	119	898	528	39	82	64	2,247
2008	97	92	70	73	68	70	69	339	1,157	296	106	66	2,501
2009	55	61	59	59	54	57	89	769	1,858	184	97	55	3,397
2010	21	17	18	15	14	14	35	87	3,122	402	26	12	3,782
2011	19	26	22	21	20	21	48	112	4,306	924	80	34	5,634
2012	85	101	85	79	80	85	322	795	181	23	63	59	1,957
Mean (2003 to 2012)	**82**	**71**	**65**	**64**	**61**	**63**	**109**	**604**	**1,376**	**224**	**75**	**58**	**2,852**
Mean (1933 to 2002)	**59**	**41**	**45**	**37**	**35**	**46**	**96**	**920**	**1,400**	**179**	**86**	**44**	**2,990**
Mean (1933 to 2012)	**62**	**45**	**48**	**40**	**38**	**48**	**98**	**881**	**1,397**	**185**	**85**	**46**	**2,973**

Table 3A-8. Other Douglas Creek Diversions Natural Flows [ac-ft]													
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	38	38	40	40	39	40	41	328	368	64	42	34	1,113
2004	44	45	48	47	46	46	60	330	204	23	43	51	987
2005	111	43	35	33	32	33	37	382	392	70	30	27	1,227
2006	36	38	40	38	37	38	74	536	388	32	40	39	1,335
2007	55	49	43	43	39	39	62	465	273	20	42	33	1,164
2008	50	47	36	38	35	36	36	176	600	154	55	34	1,296
2009	29	31	31	31	28	29	46	398	962	96	50	28	1,760
2010	11	9	9	8	7	7	18	45	1,617	208	13	6	1,959
2011	10	13	12	11	10	11	25	58	2,231	479	41	18	2,918
2012	44	52	44	41	41	44	167	412	94	12	32	30	1,014
Mean (2003 to 2012)	**43**	**37**	**34**	**33**	**31**	**32**	**57**	**313**	**713**	**116**	**39**	**30**	**1,477**
Mean (1933 to 2002)	**31**	**21**	**23**	**19**	**18**	**24**	**50**	**477**	**725**	**93**	**45**	**23**	**1,549**
Mean (1933 to 2012)	**32**	**23**	**24**	**21**	**20**	**25**	**51**	**457**	**723**	**96**	**44**	**24**	**1,540**

Table 3A-9. Lake Owen Natural Inflow [ac-ft]												
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	9	9	10	10	9	9	10	77	87	15	10	8	263
2004	10	-	154	79	76	65	106	52	58	-	11	15	625
2005	16	11	33	8	8	8	67	204	140	-	-	-	495
2006	1	16	20	11	7	40	109	35	37	35	17	31	360
2007	-	-	42	44	47	70	118	101	210	29	38	22	721
2008	-	-	40	24	35	9	93	198	43	6	2	24	473
2009	-	4	29	41	16	22	127	171	150	30	2	56	649
2010	180	-	2	-	-	-	96	137	-	43	77	292	827
2011	6	12	-	-	-	-	103	269	-	92	55	23	558
2012	19	69	74	81	65	93	98	36	297	-	-	11	843
Mean (2003 to 2012)	**24**	**12**	**40**	**30**	**26**	**32**	**92**	**128**	**102**	**25**	**21**	**48**	**581**
Mean (1933 to 2002)	**7**	**5**	**6**	**5**	**4**	**6**	**12**	**113**	**171**	**22**	**11**	**5**	**366**
Mean (1933 to 2012)	**9**	**6**	**10**	**8**	**7**	**9**	**22**	**115**	**162**	**22**	**12**	**10**	**393**

Table 3A-10. Middle Crow Creek Natural Flows [ac-ft]													
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	69	89	77	64	61	107	739	1,142	634	222	92	77	3,373
2004	93	99	81	69	65	126	459	476	188	58	20	109	1,846
2005	134	132	120	107	97	149	556	650	290	92	42	22	2,390
2006	109	122	108	96	92	140	272	226	67	12	-	4	1,247
2007	17	50	29	16	19	76	388	371	133	53	62	104	1,318
2008	264	255	239	230	201	237	546	630	278	86	108	115	3,189
2009	150	162	150	133	119	171	528	596	257	76	32	17	2,391
2010	128	130	113	99	94	145	741	1,151	641	232	109	52	3,635
2011	131	141	125	113	104	156	483	516	210	69	17	9	2,074
2012	96	100	82	67	70	132	364	338	117	34	-	-	1,399
Mean (2003 to 2012)	**119**	**128**	**112**	**99**	**92**	**144**	**508**	**610**	**282**	**93**	**48**	**51**	**2,286**
Mean (1933 to 2002)	**129**	**135**	**120**	**106**	**98**	**146**	**625**	**1,058**	**632**	**230**	**92**	**84**	**3,447**
Mean (1933 to 2012)	**128**	**134**	**119**	**105**	**97**	**146**	**610**	**1,002**	**588**	**213**	**87**	**80**	**3,302**

Table 3A-11. South Crow Creek Natural Flows [ac-ft]													
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	18	23	20	17	16	28	299	288	150	57	24	20	958
2004	39	42	34	29	28	53	182	120	37	25	8	46	645
2005	47	46	42	37	34	52	223	164	63	32	15	8	764
2006	53	59	52	47	44	68	105	57	7	6	-	2	499
2007	10	29	17	9	11	44	153	94	24	31	36	60	517
2008	74	71	67	64	56	66	219	159	60	24	30	32	923
2009	52	56	52	46	41	59	211	151	55	26	11	6	764
2010	31	31	27	24	23	35	300	290	152	56	26	13	1,006
2011	50	54	48	43	40	60	192	130	43	26	7	4	696
2012	48	50	41	34	35	66	143	86	20	17	-	-	538
Mean (2003 to 2012)	**42**	**46**	**40**	**35**	**33**	**53**	**203**	**154**	**61**	**30**	**16**	**19**	**731**
Mean (1933 to 2002)	**43**	**36**	**30**	**26**	**25**	**49**	**253**	**274**	**152**	**48**	**22**	**22**	**979**
Mean (1933 to 2012)	**43**	**37**	**31**	**27**	**26**	**50**	**247**	**259**	**141**	**46**	**21**	**22**	**948**

Table 3A-12. Upper North Crow Reservoir Natural Inflows [ac-ft]													
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	39	51	43	36	34	60	418	646	359	126	52	43	1,907
2004	53	56	46	39	37	72	259	269	106	33	11	62	1,044
2005	76	74	68	60	55	84	314	367	164	52	24	13	1,352
2006	61	69	61	55	52	79	154	128	38	7	-	2	705
2007	9	28	16	9	11	43	220	210	75	30	35	59	745
2008	149	144	135	130	114	134	309	356	157	49	61	65	1,803
2009	85	92	85	75	67	96	299	337	145	43	18	10	1,352
2010	72	73	64	56	53	82	419	651	363	131	62	30	2,056
2011	74	80	71	64	59	88	273	292	119	39	10	5	1,173
2012	54	56	46	38	39	74	206	191	66	19	-	-	791
Mean (2003 to 2012)	**67**	**72**	**64**	**56**	**52**	**81**	**287**	**345**	**159**	**53**	**27**	**29**	**1,293**
Mean (1933 to 2002)	**65**	**70**	**62**	**54**	**50**	**78**	**333**	**543**	**318**	**115**	**47**	**44**	**1,773**
Mean (1933 to 2012)	**65**	**70**	**62**	**54**	**50**	**78**	**327**	**518**	**298**	**107**	**45**	**42**	**1,713**

Table 3A-13. Upper North Crow Reservoir Natural Inflows [ac-ft]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
2003	6	8	7	6	5	9	65	101	56	20	8	7	299
2004	8	9	7	6	6	11	41	42	17	5	2	10	164
2005	12	12	11	9	9	13	49	58	26	8	4	2	212
2006	10	11	10	9	8	12	24	20	6	1	-	0	110
2007	1	4	3	1	2	7	34	33	12	5	5	9	117
2008	23	23	21	20	18	21	48	56	25	8	10	10	282
2009	13	14	13	12	11	15	47	53	23	7	3	2	212
2010	11	11	10	9	8	13	66	102	57	21	10	5	322
2011	12	12	11	10	9	14	43	46	19	6	2	1	184
2012	9	9	7	6	6	12	32	30	10	3	-	-	124
Mean (2003 to 2012)	**11**	**11**	**10**	**9**	**8**	**13**	**45**	**54**	**25**	**8**	**4**	**5**	**203**
Mean (1933 to 2002)	**10**	**11**	**10**	**8**	**8**	**12**	**52**	**85**	**50**	**18**	**7**	**7**	**278**
Mean (1933 to 2012)	**10**	**11**	**10**	**8**	**8**	**12**	**51**	**81**	**47**	**17**	**7**	**7**	**269**

Table 3A-14. Brush Creek Natural Flows [ac-ft]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
2003	11	14	12	10	9	16	114	176	98	34	14	12	520
2004	14	15	13	11	10	20	71	73	29	9	3	17	285
2005	21	20	18	16	15	23	86	100	45	14	7	3	369
2006	17	19	17	15	14	22	42	35	10	2	-	1	192
2007	3	8	4	2	3	12	60	57	21	8	9	16	203
2008	41	39	37	35	31	37	84	97	43	13	17	18	492
2009	23	25	23	21	18	26	81	92	40	12	5	3	369
2010	20	20	17	15	14	22	114	177	99	36	17	8	561
2011	20	22	19	17	16	24	74	80	32	11	3	1	320
2012	15	15	13	10	11	20	56	52	18	5	-	-	216
Mean (2003 to 2012)	**18**	**20**	**17**	**15**	**14**	**22**	**78**	**94**	**43**	**14**	**7**	**8**	**353**
Mean (1933 to 2002)	**18**	**19**	**17**	**15**	**14**	**21**	**91**	**148**	**87**	**31**	**13**	**12**	**484**
Mean (1933 to 2012)	**18**	**19**	**17**	**15**	**14**	**21**	**89**	**141**	**82**	**29**	**12**	**11**	**468**

Table 3A-15. Granite Springs Reservoir Natural Inflows [ac-ft]													
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	81	105	90	76	71	125	866	1,339	744	260	108	90	3,955
2004	109	21	241	287	141	284	496	184	244	212	184	190	2,594
2005	322	254	212	125	114	174	394	819	3,766	495	367	152	7,194
2006	180	129	187	147	149	490	413	532	179	93	184	179	2,861
2007	239	133	144	60	211	482	509	459	179	184	184	163	2,947
2008	186	165	220	144	183	250	477	1,045	1,451	358	362	296	5,137
2009	216	233	240	239	263	338	1,233	1,382	598	148	319	260	5,471
2010	410	507	132	240	311	351	1,672	2,074	1,653	937	500	209	8,995
2011	367	308	355	301	277	422	878	1,760	1,030	564	227	190	6,680
2012	260	270	251	219	278	497	440	184	179	200	184	175	3,137
Mean (2003 to 2012)	**237**	**212**	**207**	**184**	**200**	**341**	**738**	**978**	**1,002**	**345**	**262**	**190**	**4,897**
Mean (1933 to 2002)	**137**	**148**	**130**	**118**	**108**	**179**	**660**	**1,112**	**658**	**241**	**102**	**89**	**3,677**
Mean (1933 to 2012)	**149**	**156**	**140**	**126**	**119**	**199**	**670**	**1,095**	**701**	**254**	**122**	**102**	**3,830**

Table 3A-16. Crystal Lake Natural Inflows [ac-ft]													
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	11	14	12	10	10	17	119	183	102	36	15	12	541
2004	15	2	15	23	6	24	45	-	9	4	-	3	145
2005	27	23	17	17	16	24	31	88	492	44	27	-	806
2006	6	-	5	-	2	42	32	48	-	-	-	-	134
2007	13	-	5	-	17	51	53	38	-	-	-	-	176
2008	4	6	16	8	14	13	41	118	175	24	24	18	460
2009	12	15	19	21	26	35	148	164	60	-	21	12	533
2010	39	55	18	12	20	26	205	259	202	105	53	5	998
2011	27	27	34	29	27	43	98	217	118	56	13	3	690
2012	15	21	19	16	24	47	39	-	-	2	-	-	183
Mean (2003 to 2012)	**17**	**16**	**16**	**14**	**16**	**32**	**81**	**111**	**116**	**27**	**15**	**5**	**467**
Mean (1933 to 2002)	**19**	**20**	**18**	**16**	**15**	**25**	**90**	**152**	**90**	**33**	**14**	**12**	**503**
Mean (1933 to 2012)	**19**	**20**	**18**	**16**	**15**	**26**	**89**	**147**	**93**	**32**	**14**	**11**	**498**

Table 3A-17. South Crow Creek Natural Flows [ac-ft]												
Year	**Oct**	**Nov**	**Dec**	**Jan**	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Annual**
2003	18	23	19	16	15	27	294	283	147	56	23	20	942
2004	39	41	34	29	27	52	179	118	37	24	8	45	635
2005	46	45	41	37	33	51	219	161	62	32	15	8	752
2006	52	58	51	46	44	67	103	56	7	6	-	2	491
2007	10	28	17	9	11	43	150	92	23	30	35	59	509
2008	73	70	66	63	55	65	215	156	59	24	30	32	908
2009	51	55	51	45	40	58	208	148	54	26	11	6	752
2010	30	31	27	23	22	34	295	285	149	55	26	12	989
2011	49	53	47	43	39	59	189	128	42	26	6	3	685
2012	47	49	40	33	34	65	141	84	19	17	-	-	529
Mean (2003 to 2012)	**41**	**45**	**39**	**34**	**32**	**52**	**199**	**151**	**60**	**30**	**15**	**19**	**719**
Mean (1933 to 2002)	**42**	**36**	**30**	**26**	**25**	**49**	**251**	**272**	**151**	**48**	**21**	**22**	**971**
Mean (1933 to 2012)	**42**	**37**	**31**	**27**	**26**	**49**	**245**	**257**	**140**	**46**	**20**	**22**	**940**

3.2. Comparison of 2000 to 2008 to other droughts

Droughts can be classified by hydrologic impacts, such as meteorological drought (snowpack), agricultural drought (soil moisture), and hydrologic drought (stream flows and reservoir levels). A long-term reconstruction of agricultural drought for the Lower Platte basin in Wyoming[5] is shown in Figure 3A-2. Drought events can further be described as the number of consecutive years of below normal hydrologic conditions and the peak severity of the drought. The latest drought of 2000 to 2008 is one of the longest droughts and greatest peak severity in the reconstructed period of record. A comparison of this drought to previous droughts is presented to determine the drought of record. The droughts examined are:

- 1950 to 1956 (the “1950s drought”)
- 1958 to 1967 (the “1960s drought”)
- 2000 to 2008 (the “2000s drought”)

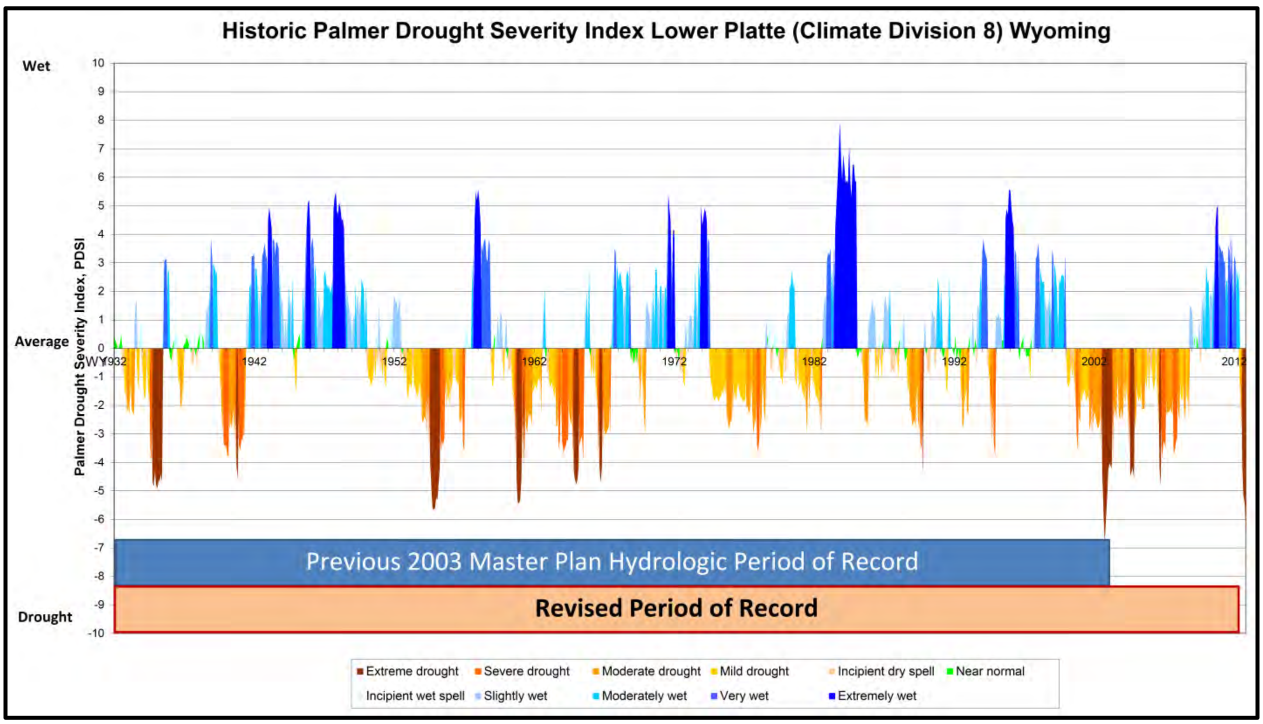

Figure 3A-2. Historic Palmer Drought Severity Index

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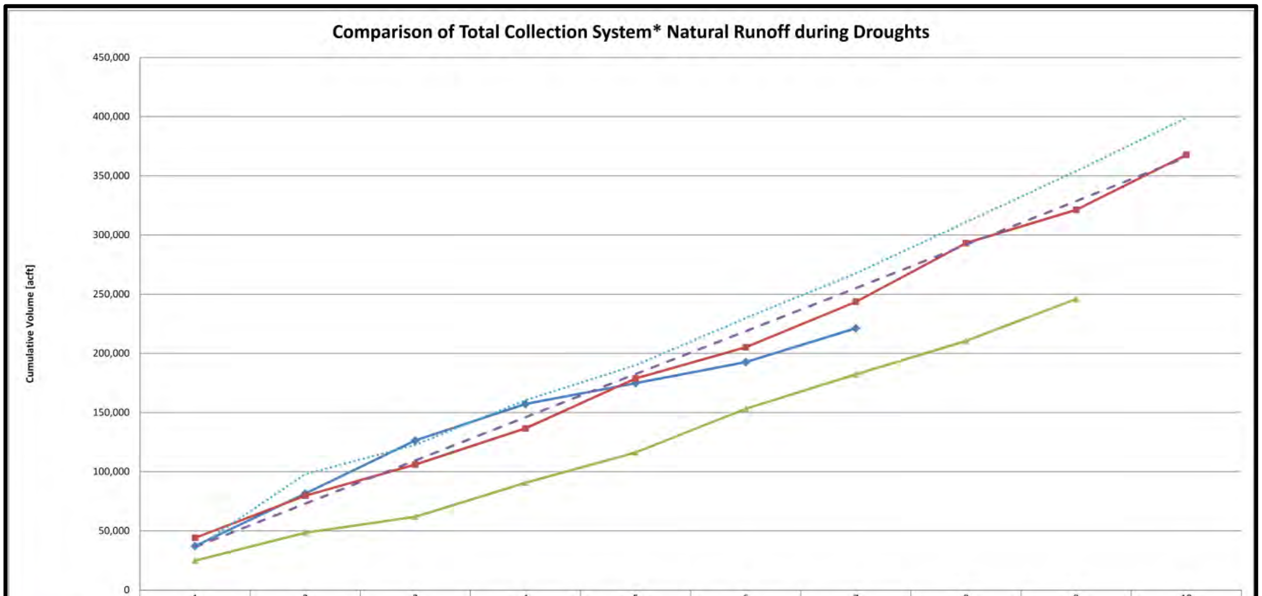
[5] Source: National Climatic Data Center (National Oceanic and Atmospheric Administration). Historic Divisional Drought Statistics. Available on-line at: http://www7.ncdc.noaa.gov/CDO/CDODivisionalSelect.jsp#. Accessed August 2012.

The cumulative runoff over the course of each drought event was compared. Additionally the 10-year moving average and water years 1990 to 2000 were used in the comparison. Figure 3A-3 shows a comparison of the total collection system runoff. Table 3A-18 shows the difference for each drought and runoff location at the end of the respective drought compared to the 10-year moving average.



	1	2	3	4	5	6	7	8	9	10
1950s Drought 1950 to 1956	37,260	81,557	126,495	157,311	174,912	192,695	221,257			
1960s Drought 1958 to 1967	44,244	79,782	106,092	136,705	178,805	205,199	243,637	293,202	321,344	367,761
2000s Drought 2000 to 2008	24,989	48,463	61,979	90,766	116,282	153,225	182,304	210,562	245,946	
10 Year Average	36,405	73,026	109,514	146,066	182,565	218,842	255,205	291,851	328,686	365,681
1990 to 2000	35,190	97,939	122,834	160,479	190,031	229,916	267,699	310,904	354,077	399,148

Figure 3A-3. Comparison of Total Collection System Natural Runoff during Droughts

Table 3A-18. Comparison of Droughts Based on Cumulative Runoff Volumes

Inflow Location	Change in Cumulative Volume at end of drought from average [%]			
	1950s Drought 1950 to 1956	1960s Drought 1958 to 1967	2000s Drought 2000 to 2008	1990 to 2000
Total System	-13%	1%	-25%	9%
Hog Park	-3%	-6%	-13%	6%
Rob Roy	-1%	-7%	-26%	11%
Lake Owen	-3%	-10%	5%	8%
Middle Crow Creek	-52%	31%	-46%	0%
South Crow Creek	-36%	19%	-35%	-2%
Granite and Crystal	-55%	22%	-5%	6%

Note: Middle Crow Creek is computed as a proxy to North Crow Creek. Using area ratios, it is assumed that inflows into the North Crow Creek reservoirs and dams are equivalent to 81% of the Middle Crow Creek runoff value.

The 2000s drought was consistently below average conditions in most of the collection area. The lowest runoff year was 2002 where a system runoff total of 13,500 acre-feet was estimated, roughly 50% of the average annual runoff during the drought. Total cumulative runoff was approximately 25% below normal by the end of the drought. The 1950s drought comparatively ran 13% below normal, while the 1960s drought was about average. A crucial difference between the drought events was the geographic area that was affected:

- Hog Park Reservoir Tunnel: The estimated tunnel flows during the 1950s and 1960s drought were approximately average. The tunnel flows during the 2000s drought were below average, mainly as reduced tunnel flows during July. In some cases, the tunnel was shutoff due to water right restrictions and in others cases due to complete fill of Hog Park Reservoir.
- Hog Park Reservoir Natural Runoff: The estimated runoff during the 1950s drought was about 3% below average, the 1960s drought was 6% below average, and the 2000s drought was approximately 13% below average.
- Rob Roy Reservoir Natural Runoff: The 1950s drought estimated runoff ranged from average to below average. The 1960s drought was about 7% below average. The 2000s drought was 26% below average runoff.
- Middle and South Crow: Both the 1950s and 2000s drought were below average runoff (40% to 50% below average). The 1960s drought was above average runoff in this area.
- Granite and Crystal Reservoirs Natural Runoff: Runoff during the 1950s drought was over 50% below average. The 2000s drought ranged from

slightly below average. The 1960s drought at times was below average to above average.

Based on regional climate data, the 2000s drought at its peak was as severe as the 1950s drought. The duration of the 2000s drought was longer than the 1950s drought and as long as the 1960s drought. During both the 1950s and 1960s drought, certain portions of BOPU's raw water collection drainage area tended to perform better than others. The Rob Roy and Hog Park areas were slightly below average runoff during the 1950s drought. Generally, the Middle Crow/South Crow/Granite-Crystal drainage areas were average for the 1960s drought. In the 2000s drought, all raw water collection drainage areas were below average.

4. Snow Pack Sensitivity

An analysis of reduced snowpack was conducted to examine the sensitivity of the existing raw water collection system to a sustained 25% reduction in historic snowpack. The background of this snowpack sensitivity is presented in Appendix 3A Item 4.1. While multiple approaches to estimating the reduced natural runoff are available, the selected approach is presented in Appendix 3A Item 4.2. The natural runoff for various locations which may result from a 25% reduction in historic snowpack is presented in Section Appendix 3A Item 4.3.

4.1. Background

The Brooklyn Lake SnoTel site is one of the longest running snow measurement stations in the collection system area, operated by the NRCS from 1936 to present. Snow Water Equivalent (SWE) is generally provided for January to June each year, although in early years in the period of record the SWE is estimated. The maximum SWE for each year ranges from 10 inches to nearly 50 inches, with a long term average of 30 inches (Figure 3A-4). The 10-year moving average has had a downward trend, influenced in part by below average SWE during the drought of 2000. There is a high degree of variability in year-to-year snowpack. Additionally, minimum and average temperatures have an increasing trend (Figure 3A-5). These increasing temperatures will promote earlier snow melt and potentially lessen April 1st SWE.

The US Bureau of Reclamation (2011) examined potential snowpack and runoff changes based in part on trends and global circulation models of future climate variability. Several basins were part of the study, including the mainstem Missouri and the South Platte. Table 3A-19 provides projected hydrology for years 2020, 2050, and 2070 using a range of climate variables for the South Platte River near Sterling, CO. The mean change of temperature under the climate scenarios rises an additional 5 degrees Fahrenheit. While precipitation increases by 2%, April 1st SWE and annual runoff decreases. The change in annual runoff is an 8% decrease in 2020 to a 17% decrease in 2070.

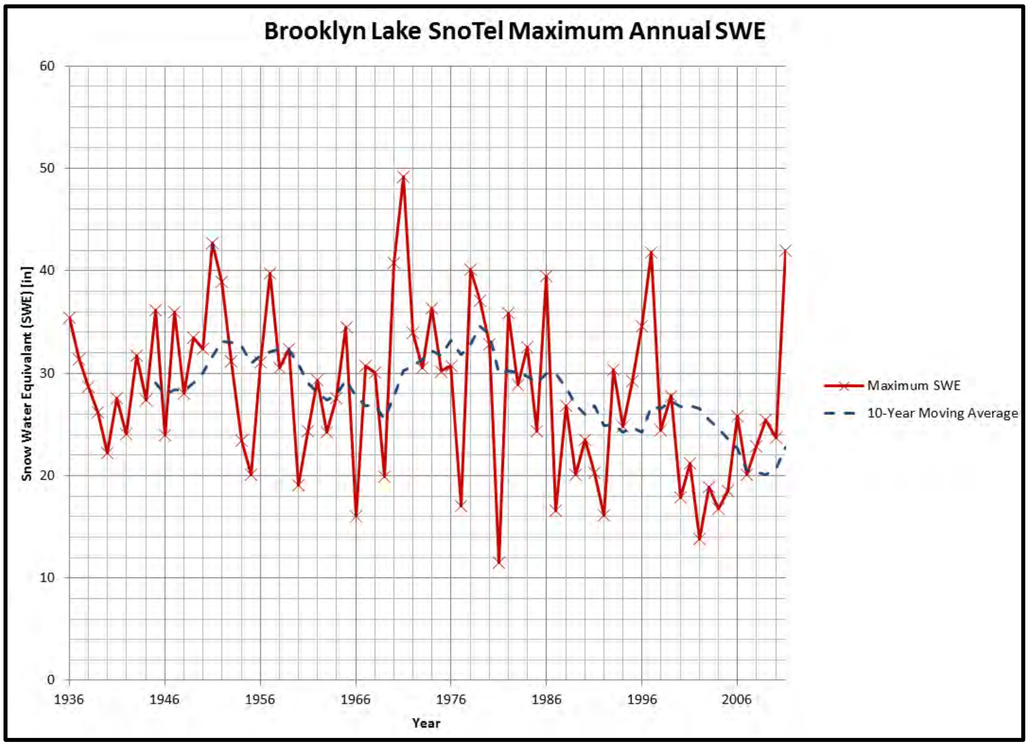


Figure 3A-4. Historic Brooklyn Lake SnoTel SWE Measurements

Source: Natural Resources Conservation Service. Brooklyn Lake SnoTel Site. Available online at: http://www.wcc.nrcs.usda.gov/nwcc/site?sitenum=367&state=wy. Accessed August 2013.

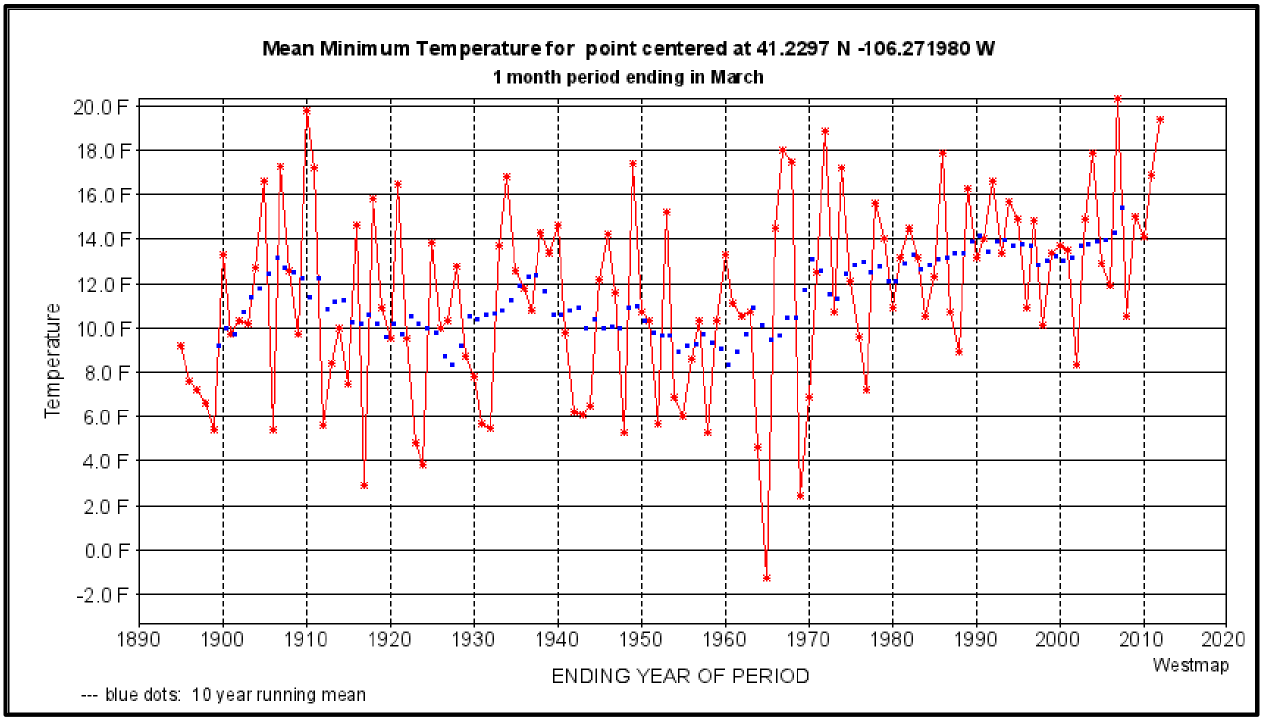


Figure 3A-5. Historic Average Minimum March Temperatures, Rob Roy Drainage Area

Source: Desert Research Institute, Western Climate Mapping Initiative (WestMap), Available on-line at: http://www.cefa.dri.edu/Westmap/Westmap_home.php

Table 3A-19. USBR Climate Change Ensemble Predictions **South Platte River near Sterling, Colorado**			
Hydroclimate Metric (change from 1990s)	Projected Year 2020	Projected Year 2050	Projected Year 2070
Mean Annual Temperature (°F)	1.8	3.6	5.0
Mean Annual Precipitation (%)	0.0	0.6	2.1
Mean April 1st Snow Water Equivalent (%)	-59.9	-72.1	-74.7
Mean Annual Runoff (%)	-8.5	-13.9	-17.5
Mean December–March Runoff (%)	-7.8	-12.2	-11.4
Mean April–July Runoff (%)	-7.2	-10.8	-9.9
Mean Annual Maximum Week Runoff (%)	1.8	-3.4	-2.3
Mean Annual Minimum Week Runoff (%)	-16.3	-23.5	-29.3

4.2. Methodology

A set of synthetic runoffs were developed to provide an assessment of existing collection system performance assuming that the snowpack is 75% of the historic water content. The approach used is a statistical adjustment of the calculated runoff from inflow locations. The annual runoff of each inflow location for the reduced snowpack scenario is 75% of the historic estimated runoff.

The adjustment is performed based on the hydrograph type. Figure 3A-6 shows a typical snowmelt dominated hydrograph. Base flow is fairly constant through most of the year, with a peak driven by snowmelt. The base flow is derived from mountain front recharge which is assumed to not be greatly affected with the magnitude of snowpack change. In addition to winter and fall flows, a portion of the snowmelt runoff is also ascribed to baseflow. This portion is calculated using the US Bureau of Reclamation's Base Flow Index (BFI) method. The BFI value is the average annual ratio of baseflow volume to total runoff. In adjusting the runoff, baseflow is not changed. The spring and summer runoff is adjusted such that the annual runoff change provides the 25% reduction.

The second type of hydrograph is typical of lower elevations, with an example hydrograph shown in Figure 3A-7. Flows peak both with snowmelt and rainfall precipitation. The precipitation reduction scenario is performed by uniformly reducing all flows by 25%. Table 3A-20 shows the precipitation reduction approach and associated baseflow index values (if applicable). The upper elevation runoff locations (Hog Park, Rob Roy, and Lake Owen) are snowmelt dominated hydrographs. The baseflow index is typically 0.55 in most areas and 0.61 in the Hog Park drainage area, meaning that 55% and 61% of the respective annual runoff volumes is from baseflow. The remaining locations are uniformly reduced by 25%. Hog Park Tunnel flows are assumed to be 75% of the historic potential tunnel flows.

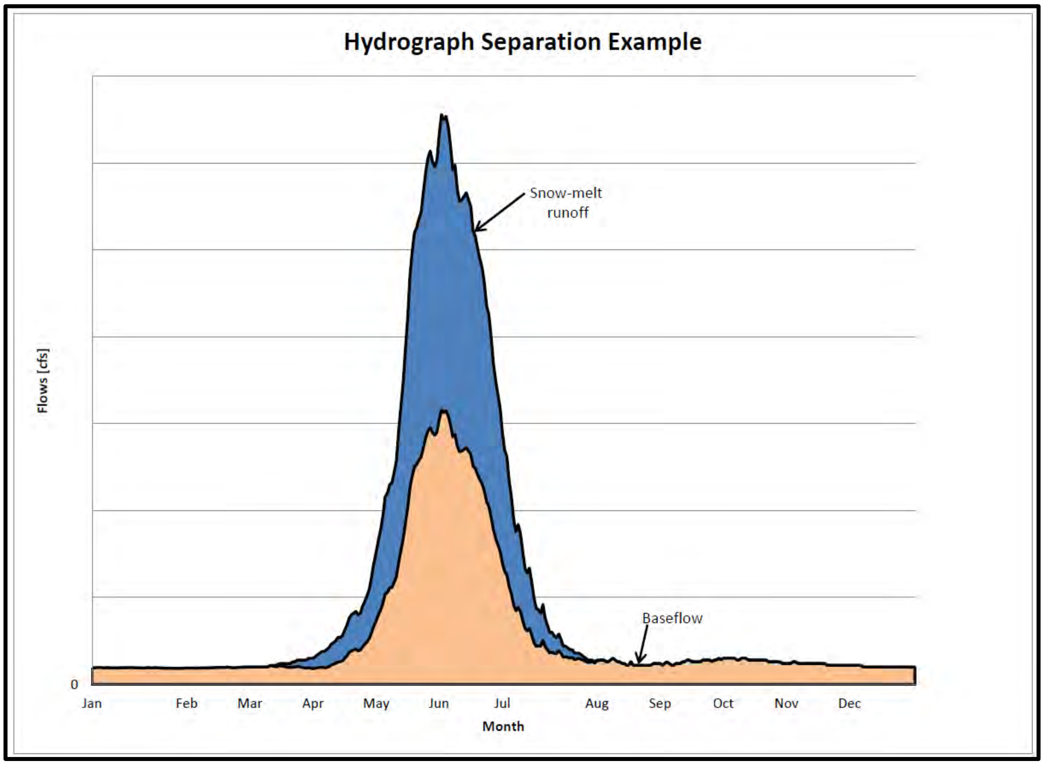


Figure 3A-6. Hydrograph Type I

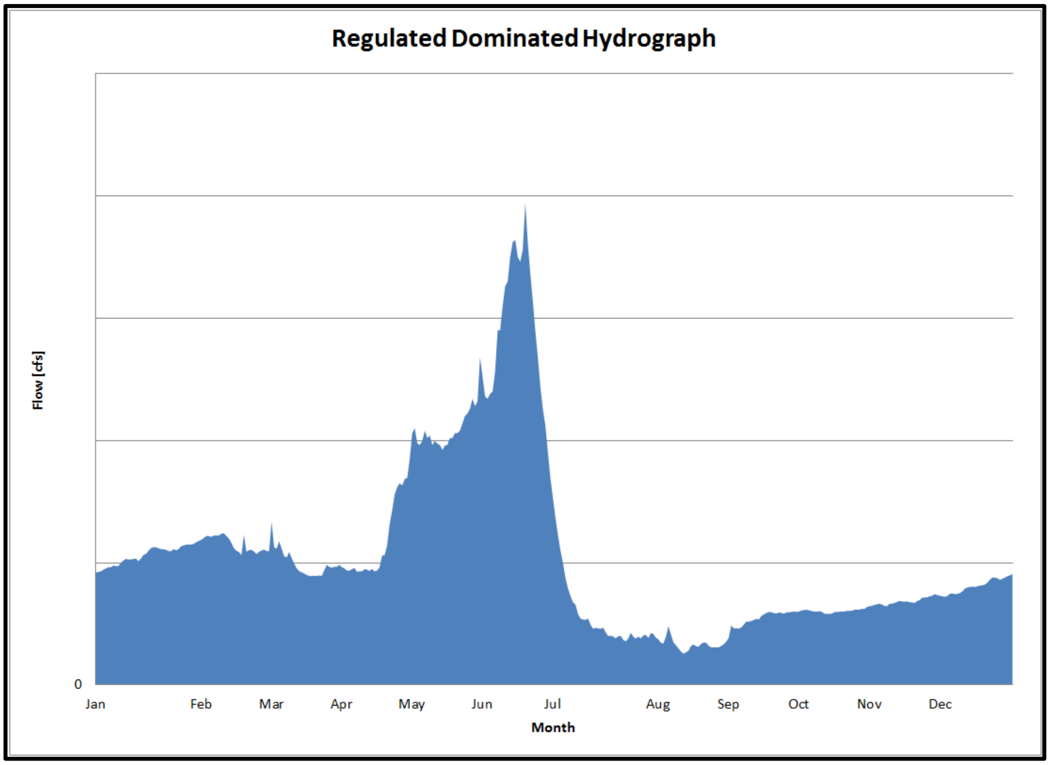


Figure 3A-7. Hydrograph Type II

Table 3A-20. Precipitation Reduction Methods

Inflow Location	Precipitation Sensitivity Approach	Baseflow Index
Hog Park Reservoir Natural Runoff (HPN)	Hydrograph Type I	0.61
Rob Roy Reservoir Natural Runoff (RRN)	Hydrograph Type I	0.55
Horse Creek and other collection area (HCD, PLD)	Hydrograph Type I	0.55
Lake Owen Natural Runoff (LON)	Hydrograph Type I	0.55
Granite and Crystal Reservoirs Natural Runoff (GRN, CRN)	Hydrograph Type II	n/a
Middle Crow Creek Reservoirs Natural Runoff and collection area (NCN, ONN, BCD)	Hydrograph Type II	n/a
South Crow Creek Reservoir Natural Runoff (SCN)	Hydrograph Type II	n/a

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4.3. Reduced Snowpack Datasets

Table 3A-21. Hog Park Reservoir Natural Inflows with 75% of Historic Snow Pack													
[acre-feet]													
Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	194	189	195	195	192	270	447	3,529	5,141	60	67	65	10,544
1934	204	210	205	228	310	654	2,044	2,262	935	59	89	192	7,391
1935	209	181	156	154	151	287	656	4,372	7,420	287	84	70	14,027
1936	156	193	203	201	187	418	1,902	5,119	2,494	111	34	93	11,111
1937	202	262	220	174	157	299	805	5,527	5,804	694	169	105	14,417
1938	245	266	307	255	298	453	1,924	5,682	6,439	311	118	192	16,489
1939	347	340	375	397	399	778	1,747	3,853	2,378	63	84	223	10,985
1940	264	191	207	224	275	423	1,706	4,558	2,768	89	40	157	10,903
1941	287	261	214	200	232	437	798	5,636	4,492	208	231	199	13,195
1942	607	399	294	296	267	357	2,139	4,698	5,658	254	57	91	15,118
1943	227	272	276	222	272	403	2,086	2,654	5,507	220	110	88	12,336
1944	248	195	201	196	246	293	524	4,526	6,298	360	63	125	13,275
1945	183	166	157	180	176	266	451	5,519	8,736	1,303	492	180	17,808
1946	355	431	367	315	321	598	3,256	2,976	3,209	134	157	182	12,302
1947	310	309	299	249	274	629	1,034	5,956	6,032	615	197	167	16,072
1948	362	336	414	366	306	441	1,617	5,565	3,143	145	81	79	12,855
1949	227	227	226	226	204	309	1,540	6,106	7,643	496	97	122	17,423
1950	357	252	245	273	259	309	1,607	5,051	7,099	471	62	162	16,148
1951	284	286	286	274	283	495	1,034	4,000	3,716	242	243	112	11,255
1952	212	152	198	241	226	274	1,738	6,259	8,045	279	177	122	17,923
1953	220	220	258	275	283	433	620	3,197	5,810	221	198	60	11,795
1954	225	282	242	265	242	362	2,112	3,518	1,757	169	91	190	9,455
1955	333	306	289	279	243	296	1,221	4,206	3,928	151	117	89	11,458
1956	201	286	382	345	266	454	2,174	5,349	3,594	115	85	57	13,307
1957	121	166	166	172	157	230	476	4,011	10,243	1,494	226	164	17,627
1958	305	300	249	241	304	383	804	7,475	4,508	119	29	136	14,852
1959	245	224	278	311	272	344	794	3,607	2,976	248	121	222	9,643
1960	454	327	225	231	225	748	2,303	3,371	3,774	175	30	57	11,920
1961	181	229	262	238	222	385	725	3,539	1,977	75	144	456	8,433
1962	334	332	256	257	404	435	2,879	5,493	4,760	368	48	120	15,686
1963	210	184	156	194	309	524	841	4,234	2,164	92	128	207	9,243
1964	166	225	178	203	202	231	635	5,047	4,684	407	118	112	12,208
1965	215	231	237	220	250	224	509	3,077	6,068	3,752	949	729	16,460
1966	654	330	268	248	160	190	1,015	5,543	1,594	470	258	160	10,890
1967	247	169	169	145	142	167	338	3,328	4,509	3,266	713	505	13,698
1968	347	228	191	171	140	167	334	2,313	5,958	2,563	931	460	13,804
1969	255	152	136	127	61	96	1,031	6,908	2,965	1,100	310	190	13,330
1970	300	238	236	209	189	242	456	4,857	6,427	3,272	571	461	17,459
1971	89	152	136	127	53	96	1,008	4,658	8,428	921	157	95	15,920
1972	672	478	411	345	290	456	709	4,196	3,474	1,435	615	535	13,616
1973	106	152	136	127	50	96	348	3,676	4,602	703	214	108	10,317
1974	55	152	136	127	48	96	609	8,213	7,247	36	10	57	16,786
1975	203	178	144	127	101	106	310	2,475	5,698	4,335	565	231	14,473
1976	55	152	136	127	48	96	541	4,845	4,130	57	16	57	10,260
1977	55	152	136	127	48	96	775	2,616	1,228	35	10	57	5,335
1978	55	152	136	127	48	96	859	4,338	8,977	234	33	57	15,113
1979	55	152	136	127	48	96	572	5,302	8,741	36	10	57	15,331
1980	55	152	136	127	48	96	643	5,441	7,764	36	10	57	14,564
1981	269	244	219	181	158	174	1,231	3,012	1,676	593	302	252	8,310
1982	144	152	136	127	48	96	695	4,988	8,097	2,153	307	154	17,097

Table 3A-21. Hog Park Reservoir Natural Inflows with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1983	403	288	219	212	165	183	408	2,330	8,159	4,498	751	314	17,930
1984	599	429	403	266	219	243	409	6,024	7,088	4,651	1,023	866	22,220
1985	55	152	136	127	48	96	1,345	7,442	4,697	36	10	57	14,201
1986	55	152	136	127	48	96	1,564	6,527	8,354	36	10	57	17,162
1987	55	152	136	127	48	96	1,440	5,189	893	35	10	57	8,238
1988	55	152	178	224	255	207	919	4,999	3,801	318	10	108	11,225
1989	81	182	279	292	315	420	1,476	3,996	1,910	350	162	321	9,783
1990	97	306	361	127	274	248	1,246	3,611	3,540	648	21	308	10,786
1991	177	232	296	279	240	300	484	4,971	3,979	515	227	252	11,953
1992	145	481	299	469	387	505	961	4,060	1,038	714	366	201	9,626
1993	138	428	505	748	379	557	526	5,685	7,048	1,450	616	120	18,200
1994	1,006	334	322	516	332	572	1,072	4,804	1,004	32	208	129	10,331
1995	181	329	236	327	230	279	443	2,446	7,275	1,717	323	303	14,090
1996	239	474	604	480	48	96	789	4,612	4,291	506	451	135	12,725
1997	55	529	569	565	438	526	970	5,932	7,306	894	547	805	19,136
1998	676	331	202	529	505	785	869	9,852	8,686	1,665	488	234	24,823
1999	665	720	530	593	618	623	1,019	5,079	6,752	1,009	196	497	18,302
2000	284	341	446	488	463	410	1,091	7,102	2,409	261	270	148	13,713
2001	350	152	136	292	302	243	665	5,921	1,773	36	312	57	10,240
2002	132	235	481	249	279	381	674	2,063	609	143	530	568	6,344
2003	690	686	742	746	717	727	497	4,039	4,457	813	779	617	15,510
2004	816	28	376	303	262	508	1,158	3,985	1,605	234	166	488	9,929
2005	663	684	597	593	569	599	1,228	7,167	5,592	1,154	206	116	19,167
2006	343	510	673	506	380	488	1,647	8,040	3,659	406	205	477	17,335
2007	880	743	608	522	527	879	1,569	4,700	1,733	232	285	322	12,999
2008	471	285	519	460	508	178	304	4,208	7,608	1,755	235	309	16,840
2009	179	348	432	426	327	520	826	7,211	7,450	1,290	160	165	19,334
2010	618	530	88	435	302	418	1,046	4,320	8,538	1,532	264	54	18,143
2011	478	494	603	441	451	528	739	3,735	12,143	5,365	422	331	25,731
2012	344	453	360	321	361	788	2,756	4,857	834	160	-	67	11,301
Avg	290	287	284	282	251	355	1,085	4,770	4,937	844	240	216	13,841

Table 3A-22. Rob Roy Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	273	250	223	223	225	356	585	1,416	12,988	636	310	172	17,658
1934	151	98	125	115	100	182	336	5,787	278	143	160	116	7,591
1935	124	95	239	350	253	239	417	1,948	10,245	994	842	292	16,039
1936	121	92	125	115	95	115	948	9,695	5,985	646	584	141	18,662
1937	767	283	351	247	291	293	696	7,569	5,645	1,304	855	414	18,716
1938	323	256	257	272	262	362	820	6,547	10,192	1,053	584	860	21,788
1939	747	432	328	190	161	527	566	8,693	2,472	391	390	206	15,104
1940	216	204	352	234	257	629	331	6,680	2,187	490	343	339	12,262
1941	422	244	249	196	191	383	370	8,152	3,694	584	781	458	15,724
1942	871	426	352	190	219	408	1,019	4,774	9,108	751	528	283	18,929
1943	571	303	310	202	153	379	1,072	6,846	12,853	1,260	567	114	24,630
1944	263	291	490	367	328	527	961	3,453	4,724	585	691	257	12,938
1945	291	181	180	167	172	283	863	6,087	8,903	1,945	2,229	532	21,834
1946	603	448	673	504	379	967	1,317	4,473	6,069	722	818	395	17,368
1947	495	480	502	243	203	309	424	6,711	8,954	1,476	1,572	401	21,770
1948	487	345	319	267	219	315	1,086	9,112	4,370	585	329	114	17,549
1949	344	275	167	129	111	218	334	3,975	17,427	1,418	438	212	25,049
1950	690	451	271	146	166	209	418	2,850	12,605	1,322	938	673	20,739
1951	474	450	349	192	155	226	534	6,568	13,927	1,969	1,164	285	26,294
1952	759	395	196	160	126	206	694	7,506	12,002	1,032	518	177	23,771
1953	265	193	386	426	326	621	598	1,690	9,689	946	2,008	408	17,557
1954	296	292	452	373	471	549	395	5,654	1,233	278	277	198	10,467
1955	178	136	125	17	94	80	95	5,476	2,877	603	516	210	10,406
1956	226	208	215	246	201	246	425	10,011	4,376	508	341	184	17,187
1957	235	190	172	135	111	111	96	1,883	16,107	3,007	661	293	23,001
1958	409	208	184	154	100	80	875	12,537	7,224	479	293	250	22,793
1959	176	152	129	111	94	98	138	4,557	10,498	810	378	296	17,437
1960	721	357	215	184	144	154	955	6,936	4,508	513	261	197	15,146
1961	196	215	184	154	111	142	343	5,986	4,921	506	393	786	13,937
1962	1,050	728	436	332	302	360	1,658	8,797	6,303	977	362	234	21,539
1963	274	167	125	102	109	156	183	7,752	3,227	445	373	276	13,189
1964	211	208	194	210	195	207	250	5,134	8,361	1,135	438	272	16,815
1965	227	235	280	279	246	253	186	5,099	9,708	1,043	394	244	18,194
1966	1,767	672	742	386	338	599	543	5,538	2,075	348	377	259	13,643
1967	265	185	360	266	221	488	414	2,866	9,638	1,482	1,034	682	17,902
1968	392	340	351	384	449	1,208	651	1,532	8,905	941	1,479	812	17,443
1969	412	199	258	195	138	237	388	7,580	4,967	742	506	219	15,841
1970	517	360	207	125	170	203	397	3,673	16,356	1,858	812	503	25,181
1971	1,044	490	125	68	94	170	221	1,429	24,361	2,500	417	181	31,101
1972	678	70	125	45	94	80	96	1,434	13,918	956	111	67	17,675
1973	1,531	1,044	653	835	738	943	861	6,338	5,492	1,151	1,998	869	22,454
1974	396	576	381	342	345	455	467	7,739	8,759	1,171	629	278	21,539
1975	289	280	229	204	156	238	267	1,390	9,502	2,341	1,130	245	16,272
1976	393	363	367	327	313	311	459	3,803	5,158	1,187	1,428	435	14,544
1977	551	410	388	370	323	385	640	4,547	2,605	313	324	173	11,029
1978	176	90	125	92	94	106	143	1,439	16,470	2,359	471	149	21,714
1979	176	-	125	17	94	80	98	1,452	16,503	1,735	12	2	20,294
1980	176	128	125	85	94	101	385	2,513	12,225	1,117	178	115	17,242
1981	760	660	812	531	373	609	541	4,275	2,469	305	498	451	12,284
1982	588	343	446	534	371	444	407	3,383	11,594	2,673	3,010	1,230	25,023
1983	735	442	380	338	345	561	1,092	2,561	18,062	2,825	2,803	713	30,857
1984	745	509	516	345	321	411	860	8,829	9,877	1,747	2,545	857	27,563
1985	474	410	368	337	260	321	962	7,110	5,043	690	316	239	16,530
1986	457	271	273	230	213	291	610	3,193	15,478	1,417	604	272	23,309
1987	926	656	476	448	365	410	664	5,446	788	301	405	215	11,100

Table 3A-22. Rob Roy Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1988	176	135	140	137	117	118	265	2,147	9,979	793	152	101	14,261
1989	344	318	309	314	275	395	539	3,815	2,403	361	280	632	9,985
1990	365	285	608	386	470	586	799	4,814	8,082	966	658	779	18,799
1991	779	-	243	364	345	449	396	8,807	18,582	699	663	506	31,833
1992	237	524	309	428	355	519	555	6,051	1,997	866	474	195	12,511
1993	323	310	291	456	599	628	345	6,094	8,037	1,151	684	297	19,216
1994	671	441	914	905	525	439	801	7,743	2,112	282	12	2	14,847
1995	276	137	380	347	299	428	412	1,412	11,317	2,349	102	197	17,657
1996	278	381	725	1,009	626	894	450	7,255	6,751	672	553	203	19,797
1997	506	280	1,306	682	597	454	668	7,175	8,780	749	975	1,276	23,448
1998	811	1,016	864	1,072	862	1,037	393	7,142	6,533	1,065	614	684	22,093
1999	1,056	831	892	648	613	444	821	4,894	10,855	1,640	835	510	24,038
2000	340	454	605	545	545	445	637	7,507	1,727	315	463	316	13,898
2001	184	415	699	233	503	447	408	6,857	1,571	286	398	291	12,293
2002	750	344	527	17	629	347	184	1,823	274	125	93	35	5,148
2003	596	593	636	639	617	624	424	3,377	3,787	664	664	541	13,162
2004	692	714	758	745	723	729	567	3,123	1,927	217	683	800	11,678
2005	1,747	679	552	523	505	528	382	3,934	4,038	724	478	420	14,510
2006	561	593	629	604	579	594	786	5,716	4,141	338	632	619	15,793
2007	868	768	673	673	619	621	627	4,735	2,782	208	668	525	13,766
2008	790	749	570	593	553	570	373	1,839	6,279	1,609	863	538	15,323
2009	453	494	482	482	444	463	516	4,442	10,733	1,065	793	445	20,813
2010	169	141	143	124	114	114	213	528	18,889	2,430	212	96	23,173
2011	155	213	182	175	161	168	290	677	25,985	5,578	654	281	34,518
2012	697	824	697	643	651	697	1,655	4,086	930	119	512	480	11,990
Average	509	368	389	331	310	395	558	5,055	8,155	1,075	694	375	18,215

Table 3A-23. Horse Creek Collection Area Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	33	31	27	27	27	44	72	174	1,590	78	38	21	2,162
1934	18	12	15	14	12	22	41	708	34	17	20	14	928
1935	15	12	29	43	31	29	51	238	1,254	122	103	36	1,964
1936	15	11	15	14	12	14	116	1,187	732	79	71	17	2,284
1937	94	35	43	30	36	36	85	926	691	160	105	51	2,291
1938	39	31	32	33	32	44	101	801	1,247	129	72	105	2,666
1939	91	53	40	23	20	64	69	1,064	302	48	48	25	1,848
1940	26	25	43	29	31	77	41	818	268	60	42	41	1,501
1941	52	30	31	24	23	47	45	997	452	72	96	56	1,925
1942	107	52	43	23	27	50	125	584	1,115	92	65	35	2,318
1943	70	37	38	25	19	46	131	838	1,573	154	69	14	3,014
1944	32	36	60	45	40	65	118	423	578	71	85	31	1,584
1945	36	22	22	20	21	35	106	745	1,090	238	273	65	2,673
1946	74	55	82	62	46	118	161	547	743	88	100	48	2,125
1947	61	59	61	30	25	38	52	822	1,096	180	192	49	2,665
1948	60	42	39	33	27	38	133	1,115	535	71	40	14	2,147
1949	42	34	20	16	14	27	41	486	2,133	173	54	26	3,067
1950	84	55	33	18	20	26	51	349	1,543	162	115	82	2,538
1951	58	55	43	23	19	28	65	804	1,704	241	142	35	3,218
1952	93	48	24	20	15	25	85	919	1,469	126	63	22	2,909
1953	32	24	47	52	40	76	73	207	1,186	116	246	50	2,149
1954	36	36	55	46	58	67	49	692	151	34	34	24	1,282
1955	22	17	15	2	12	10	12	670	352	74	63	26	1,274
1956	28	25	26	30	25	30	52	1,226	536	62	42	23	2,104
1957	29	23	21	17	14	14	12	230	1,971	368	81	36	2,816
1958	50	25	23	19	12	10	107	1,535	884	59	36	31	2,791
1959	22	19	16	14	12	12	17	557	1,285	99	46	36	2,135
1960	88	44	26	23	18	19	117	849	551	63	32	24	1,854
1961	24	26	23	19	14	17	42	733	602	62	48	96	1,706
1962	128	89	53	41	37	44	203	1,077	772	120	44	29	2,636
1963	33	20	15	12	13	19	22	949	395	54	46	34	1,613
1964	26	25	24	26	24	25	31	628	1,023	139	54	33	2,058
1965	28	29	34	34	30	31	22	624	1,189	128	48	30	2,227
1966	216	82	91	47	41	73	66	678	254	42	46	32	1,669
1967	32	23	44	33	27	60	51	351	1,180	181	127	83	2,192
1968	48	42	43	47	55	148	80	187	1,090	115	181	99	2,135
1969	50	24	32	24	17	29	48	928	608	91	62	27	1,940
1970	63	44	25	15	21	25	49	450	2,002	227	99	62	3,082
1971	128	60	15	8	12	21	27	175	2,982	306	51	22	3,807
1972	83	9	15	6	12	10	12	176	1,703	117	14	8	2,165
1973	187	128	80	102	90	115	105	776	673	141	245	106	2,747
1974	48	71	47	42	42	56	57	947	1,072	143	77	34	2,637
1975	35	34	28	25	19	29	33	171	1,163	287	138	30	1,991
1976	48	44	45	40	38	38	56	465	632	146	175	53	1,780
1977	67	50	47	45	39	47	78	557	319	39	40	21	1,349
1978	22	11	15	11	12	13	18	176	2,016	289	58	18	2,659
1979	22	-	15	2	12	10	12	178	2,020	212	1	-	2,485
1980	22	16	15	10	12	12	47	307	1,496	137	22	14	2,111
1981	93	81	99	65	46	75	66	524	302	38	61	55	1,505
1982	72	42	55	65	45	54	50	414	1,419	327	368	151	3,062
1983	90	54	46	41	42	69	134	314	2,211	346	343	87	3,777
1984	91	62	63	42	39	50	106	1,081	1,209	214	312	105	3,374
1985	58	50	45	41	32	39	118	871	618	84	39	29	2,024
1986	56	33	33	28	26	36	75	391	1,895	174	74	33	2,854

Table 3A-23. Horse Creek Collection Area Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1987	113	80	58	55	45	50	81	667	96	37	50	26	1,358
1988	22	16	17	17	14	14	32	262	1,222	97	19	12	1,745
1989	42	39	38	38	34	48	66	467	294	44	34	77	1,222
1990	45	35	74	47	58	72	98	589	989	118	80	95	2,301
1991	95	-	30	45	42	55	49	1,078	2,275	85	81	62	3,897
1992	29	64	38	52	43	64	68	741	244	106	58	24	1,532
1993	40	38	36	56	73	77	43	746	984	141	84	36	2,354
1994	82	54	112	111	64	54	98	948	258	34	1	-	1,817
1995	34	17	47	43	37	52	51	173	1,385	288	12	24	2,163
1996	34	47	89	123	77	109	55	888	826	82	68	25	2,423
1997	62	34	160	83	73	56	82	878	1,075	92	119	156	2,870
1998	99	124	106	131	105	127	48	875	800	131	75	84	2,704
1999	129	102	109	79	75	54	101	599	1,329	201	102	62	2,942
2000	42	56	74	67	67	55	78	918	211	38	57	39	1,703
2001	23	51	86	29	62	55	50	839	192	35	49	36	1,507
2002	92	42	65	2	77	42	23	223	34	15	11	4	630
2003	73	73	78	78	76	76	52	413	463	81	81	66	1,611
2004	85	87	93	91	89	89	69	382	236	27	84	98	1,429
2005	214	83	68	64	62	65	47	482	494	89	59	51	1,776
2006	69	73	77	74	71	73	96	700	507	41	77	76	1,933
2007	106	94	82	82	76	76	77	580	341	25	82	64	1,685
2008	97	92	70	73	68	70	46	225	769	197	106	66	1,876
2009	55	61	59	59	54	57	63	544	1,314	130	97	55	2,548
2010	21	17	18	15	14	14	26	65	2,312	297	26	12	2,836
2011	19	26	22	21	20	21	35	83	3,181	683	80	34	4,225
2012	85	101	85	79	80	85	203	500	114	15	63	59	1,468
Average	62	45	48	41	38	48	68	619	998	132	85	46	2,230

Table 3A-24. Other Douglas Creek Collection Area Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	17	16	14	14	14	23	37	90	824	41	20	11	1,121
1934	10	6	8	7	6	12	22	367	17	9	10	7	482
1935	8	6	15	22	16	15	27	123	650	63	53	19	1,017
1936	8	6	8	7	6	7	60	615	379	41	37	9	1,184
1937	49	18	22	16	18	19	44	480	358	83	54	26	1,187
1938	20	16	16	17	17	23	52	415	646	67	37	55	1,381
1939	47	27	21	12	10	33	36	551	157	25	25	13	957
1940	14	13	22	15	16	40	21	423	139	31	22	21	777
1941	27	15	16	12	12	24	23	517	234	37	50	29	997
1942	55	27	22	12	14	26	65	303	577	48	33	18	1,200
1943	36	19	20	13	10	24	68	434	815	80	36	7	1,562
1944	17	18	31	23	21	33	61	219	299	37	44	16	820
1945	18	11	11	11	11	18	55	386	564	124	141	34	1,384
1946	38	28	43	32	24	61	84	284	385	46	52	25	1,101
1947	31	30	32	15	13	20	27	425	568	93	100	25	1,379
1948	31	22	20	17	14	20	69	578	277	37	21	7	1,112
1949	22	17	11	8	7	14	21	252	1,105	90	28	13	1,588
1950	44	29	17	9	10	13	26	180	800	83	59	43	1,314
1951	30	29	22	12	10	14	34	416	883	125	74	18	1,667
1952	48	25	12	10	8	13	44	476	761	66	33	11	1,506
1953	17	12	24	27	21	39	38	107	615	60	127	26	1,113
1954	19	19	29	24	30	35	25	358	78	18	18	13	666
1955	11	9	8	1	6	5	6	347	182	38	33	13	659
1956	14	13	14	16	13	16	27	634	277	32	22	12	1,091
1957	15	12	11	9	7	7	6	120	1,021	191	42	19	1,459
1958	26	13	12	10	6	5	55	795	458	30	19	16	1,445
1959	11	10	8	7	6	6	9	289	665	51	24	19	1,106
1960	46	23	14	12	9	10	60	440	286	32	17	13	962
1961	12	14	12	10	7	9	21	380	312	32	25	50	884
1962	67	46	28	21	19	23	105	558	399	62	23	15	1,367
1963	17	11	8	6	7	10	12	492	205	28	24	17	836
1964	13	13	12	13	12	13	16	326	531	72	28	17	1,065
1965	14	15	18	18	16	16	12	324	615	66	25	15	1,154
1966	112	43	47	24	21	38	35	351	132	22	24	16	865
1967	17	12	23	17	14	31	26	182	611	94	66	43	1,136
1968	25	22	22	24	28	77	41	97	564	60	94	51	1,106
1969	26	13	16	12	9	15	25	481	315	47	32	14	1,004
1970	33	23	13	8	11	13	25	233	1,037	118	51	32	1,598
1971	66	31	8	4	6	11	14	91	1,545	158	26	11	1,971
1972	43	4	8	3	6	5	6	91	882	60	7	4	1,120
1973	97	66	41	53	47	60	55	402	349	73	127	55	1,424
1974	25	37	24	22	22	29	29	491	555	74	40	18	1,367
1975	18	18	15	13	10	15	17	88	603	148	72	16	1,034
1976	25	23	23	21	20	20	29	241	327	75	91	28	923
1977	35	26	25	23	20	24	41	288	165	20	21	11	699
1978	11	6	8	6	6	7	9	92	1,045	150	30	9	1,378
1979	11	-	8	1	6	5	6	92	1,046	110	1	-	1,287
1980	11	8	8	5	6	6	24	160	775	71	11	7	1,092
1981	48	42	52	34	24	39	34	271	156	19	32	29	781
1982	37	22	28	34	23	28	26	215	735	170	191	78	1,587
1983	47	28	24	21	22	36	69	162	1,145	179	178	45	1,956
1984	47	32	33	22	20	26	55	560	626	111	161	54	1,747
1985	30	26	23	21	17	20	61	451	320	44	20	15	1,047
1986	29	17	17	15	13	18	39	202	982	90	38	17	1,477
1987	59	42	30	28	23	26	42	345	50	19	26	14	705
1988	11	9	9	9	7	7	17	136	633	50	10	6	904
1989	22	20	20	20	17	25	34	242	153	23	18	40	634

Table 3A-24. Other Douglas Creek Collection Area Runoff with 75% of Historic Snow Pack [acre-feet]													
Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1990	23	18	39	24	30	37	50	305	512	61	42	49	1,191
1991	49	-	15	23	22	28	25	558	1,178	44	42	32	2,017
1992	15	33	20	27	23	33	35	384	126	55	30	12	794
1993	20	20	18	29	38	40	22	387	509	73	43	19	1,218
1994	43	28	58	57	33	28	51	491	134	18	1	-	941
1995	17	9	24	22	19	27	26	90	717	149	6	13	1,120
1996	18	24	46	64	40	57	29	460	428	42	35	13	1,256
1997	32	18	83	43	38	29	42	455	556	48	62	81	1,487
1998	51	64	55	68	55	66	25	453	414	67	39	43	1,400
1999	67	53	57	41	39	28	52	310	688	104	53	32	1,525
2000	22	29	38	35	35	28	41	476	109	20	29	20	881
2001	12	26	44	15	32	28	26	435	99	18	25	18	778
2002	48	22	33	1	40	22	12	116	18	8	6	2	327
2003	38	38	40	40	39	40	27	214	240	42	42	34	835
2004	44	45	48	47	46	46	36	198	122	14	43	51	741
2005	111	43	35	33	32	33	24	249	256	46	30	27	920
2006	36	38	40	38	37	38	50	362	263	21	40	39	1,001
2007	55	49	43	43	39	39	40	300	176	13	42	33	873
2008	50	47	36	38	35	36	24	117	398	102	55	34	972
2009	29	31	31	31	28	29	33	282	681	68	50	28	1,320
2010	11	9	9	8	7	7	13	33	1,198	154	13	6	1,469
2011	10	13	12	11	10	11	18	43	1,648	354	41	18	2,189
2012	44	52	44	41	41	44	105	259	59	8	32	30	760
Average	32	23	25	21	20	25	35	321	517	68	44	24	1,155

Table 3A-25. Lake Owen Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	4	4	3	3	3	5	9	21	195	9	5	3	264
1934	2	1	2	2	1	3	5	87	4	2	2	2	113
1935	2	1	4	5	4	4	6	29	154	15	13	4	242
1936	2	1	2	2	1	2	14	146	90	10	9	2	280
1937	11	4	5	4	4	4	11	114	85	20	13	6	280
1938	5	4	4	4	4	5	12	98	152	16	9	13	326
1939	11	6	5	3	2	8	8	130	37	6	6	3	225
1940	3	3	5	4	4	9	5	100	33	7	5	5	183
1941	6	4	4	3	3	6	6	122	55	9	12	7	236
1942	13	6	5	3	3	6	15	71	136	11	8	4	282
1943	9	5	5	3	2	6	16	103	192	19	9	2	371
1944	4	4	7	5	5	8	15	52	71	9	10	4	194
1945	4	3	3	3	3	4	13	91	133	29	33	8	327
1946	9	7	10	8	6	14	20	67	91	11	12	6	261
1947	7	7	8	4	3	5	6	100	134	22	24	6	327
1948	7	5	5	4	3	5	17	136	66	9	5	2	263
1949	5	4	3	2	2	3	5	60	261	21	7	3	376
1950	10	7	4	2	2	3	6	43	189	20	14	10	311
1951	7	7	5	3	2	3	8	99	209	30	17	4	393
1952	11	6	3	2	2	3	10	112	180	15	8	3	356
1953	4	3	6	6	5	9	9	25	145	14	30	6	263
1954	4	4	7	6	7	8	6	85	19	4	4	3	157
1955	3	2	2	-	1	1	1	82	43	9	8	3	155
1956	3	3	3	4	3	4	7	150	66	7	5	3	257
1957	4	3	3	2	2	2	1	28	241	45	10	4	345
1958	6	3	3	2	1	1	13	188	108	7	4	4	341
1959	3	2	2	2	1	1	2	68	157	12	6	4	261
1960	11	5	3	3	2	2	14	104	68	8	4	3	227
1961	3	3	3	2	2	2	5	90	74	8	6	12	209
1962	16	11	7	5	5	5	25	132	95	15	5	4	324
1963	4	3	2	2	2	2	3	116	49	7	6	4	199
1964	3	3	3	3	3	3	4	77	125	17	7	4	251
1965	3	4	4	4	4	4	3	76	145	16	6	4	273
1966	26	10	11	6	5	9	8	83	31	5	6	4	204
1967	4	3	5	4	3	7	6	43	144	22	15	10	267
1968	6	5	5	6	7	18	9	23	133	14	22	12	261
1969	6	3	4	3	2	4	6	113	74	11	8	3	237
1970	8	5	3	2	3	3	6	55	245	28	12	8	377
1971	16	7	2	1	1	3	4	21	365	37	6	3	467
1972	10	1	2	1	1	1	1	21	208	14	2	1	264
1973	23	16	10	13	11	14	13	95	82	18	30	13	338
1974	6	9	6	5	5	7	7	116	131	18	9	4	323
1975	4	4	3	3	2	4	4	21	143	35	17	4	244
1976	6	5	5	5	5	5	7	57	78	18	21	7	218
1977	8	6	6	6	5	6	10	68	39	5	5	3	167
1978	3	1	2	1	1	2	2	21	247	35	7	2	325
1979	3	-	2	-	1	1	1	22	248	26	-	-	304
1980	3	2	2	1	1	2	6	38	183	17	3	2	260
1981	11	10	12	8	6	9	8	64	37	5	7	7	184
1982	9	5	7	8	6	7	6	51	174	40	45	18	376
1983	11	7	6	5	5	8	16	38	270	42	42	11	462
1984	11	8	8	5	5	6	13	132	148	26	38	13	413
1985	7	6	6	5	4	5	14	106	76	10	5	4	248
1986	7	4	4	3	3	4	9	48	232	21	9	4	349
1987	14	10	7	7	5	6	10	82	12	5	6	3	167

Table 3A-25. Lake Owen Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1988	3	2	2	2	2	2	4	32	149	12	2	2	214
1989	5	5	5	5	4	6	8	57	36	5	4	9	150
1990	5	4	9	6	7	9	12	72	121	15	10	12	282
1991	12	-	4	5	5	7	6	132	278	10	10	8	477
1992	4	8	5	6	5	8	8	91	30	13	7	3	188
1993	5	5	4	7	9	9	5	92	121	17	10	4	287
1994	10	7	14	14	8	7	12	116	32	4	-	-	224
1995	4	2	6	5	4	6	7	21	170	36	2	3	265
1996	4	6	11	15	9	13	7	109	101	10	8	3	296
1997	8	4	20	10	9	7	10	108	132	11	15	19	353
1998	12	15	13	16	13	16	6	107	98	16	9	10	332
1999	16	12	13	10	9	7	12	74	162	24	13	8	361
2000	5	7	9	8	8	7	10	113	26	5	7	5	209
2001	3	6	10	3	8	7	6	103	23	4	6	4	184
2002	11	5	8	-	9	5	3	28	4	2	1	1	77
2003	9	9	10	10	9	9	6	51	57	10	10	8	197
2004	10	-	154	79	76	65	29	14	16	-	11	15	469
2005	16	11	33	8	8	8	47	142	98	-	-	-	371
2006	1	16	20	11	7	40	64	20	22	21	17	31	270
2007	-	-	42	44	47	70	72	62	128	18	38	22	540
2008	-	-	40	24	35	9	60	129	28	4	2	24	355
2009	-	4	29	41	16	22	84	113	99	20	2	56	487
2010	180	-	2	-	-	-	24	34	-	11	77	292	620
2011	6	12	-	-	-	-	72	188	-	64	55	23	419
2012	19	69	74	81	65	93	50	19	152	-	-	11	632
Average	9	6	10	8	7	9	14	79	115	16	12	11	295

Table 3A-26. Granite Springs Reservoir Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	113	117	105	95	90	150	824	1,102	396	53	27	68	3,140
1934	47	50	62	45	37	100	211	131	21	-	-	-	703
1935	5	16	14	19	17	29	101	1,471	1,214	190	47	35	3,157
1936	48	68	66	47	35	86	581	251	87	2	11	-	1,282
1937	10	31	29	19	21	86	327	314	421	73	9	-	1,337
1938	26	34	19	14	21	142	927	1,316	441	89	54	235	3,317
1939	150	101	38	24	17	71	728	768	150	15	-	-	2,062
1940	10	33	24	14	14	24	221	219	51	17	-	6	632
1941	29	10	10	10	13	29	615	1,102	575	322	182	62	2,957
1942	62	50	33	19	21	71	1,091	2,953	1,033	344	107	67	5,850
1943	398	331	166	47	26	43	573	1,481	824	204	38	16	4,145
1944	47	71	52	29	18	29	616	1,592	530	190	17	11	3,201
1945	51	52	38	24	21	29	435	957	691	220	152	34	2,702
1946	162	123	113	103	95	119	555	716	406	122	14	77	2,605
1947	173	188	181	170	155	179	482	1,520	1,009	297	113	75	4,540
1948	47	41	29	19	14	47	674	570	122	28	5	-	1,595
1949	24	28	24	14	17	47	739	1,337	1,868	383	83	56	4,619
1950	130	114	71	43	35	71	281	482	323	164	26	59	1,796
1951	48	56	38	29	17	33	311	462	224	77	39	2	1,334
1952	43	41	33	29	26	38	941	1,180	314	62	20	6	2,733
1953	23	30	33	38	35	71	260	397	217	71	83	6	1,263
1954	14	23	24	24	21	29	235	117	11	1	-	-	498
1955	-	2	5	10	13	38	222	176	119	6	-	-	590
1956	-	2	10	10	5	29	225	402	45	-	-	-	726
1957	-	2	10	14	26	38	592	2,683	934	302	96	54	4,749
1958	127	100	71	47	64	95	685	1,125	331	239	53	21	2,957
1959	41	68	38	33	30	55	598	1,115	418	120	27	17	2,559
1960	60	46	38	33	26	198	276	201	53	4	-	-	935
1961	-	35	29	24	17	68	341	1,178	813	206	83	113	2,906
1962	126	124	74	49	134	155	800	767	527	147	19	22	2,942
1963	86	63	50	28	77	196	582	561	191	53	22	35	1,944
1964	68	131	142	108	107	110	410	1,096	1,021	739	362	110	4,403
1965	161	176	206	230	197	244	406	1,235	2,599	1,007	371	302	7,133
1966	274	224	230	116	96	188	332	256	194	151	143	152	2,354
1967	140	395	490	484	409	403	523	1,004	1,369	830	210	239	6,494
1968	305	363	326	394	374	307	689	1,706	832	290	270	207	6,061
1969	241	204	191	228	203	303	911	656	908	749	146	183	4,922
1970	213	208	202	194	171	182	653	1,416	948	350	111	102	4,750
1971	179	170	163	149	136	164	608	1,069	641	233	77	119	3,707
1972	130	136	128	117	104	140	241	211	67	14	33	44	1,363
1973	102	90	78	68	61	98	473	614	296	113	20	86	2,097
1974	77	52	33	21	20	64	494	666	332	116	17	18	1,910
1975	111	112	104	93	85	119	433	521	236	84	64	45	2,005
1976	87	101	86	75	71	112	397	449	193	65	25	23	1,682
1977	38	58	48	38	36	83	299	289	104	41	39	6	1,079
1978	7	35	24	14	17	68	231	198	62	19	20	5	698
1979	66	85	71	60	56	90	612	1,092	661	238	131	82	3,242
1980	103	78	59	41	41	81	647	1,347	885	323	94	77	3,775
1981	123	130	122	112	99	132	506	698	355	131	69	52	2,528
1982	107	122	109	100	89	129	381	420	176	56	68	121	1,876
1983	149	150	140	132	116	137	638	2,115	1,657	626	218	117	6,194
1984	122	90	80	68	66	103	574	917	518	191	121	92	2,940
1985	164	169	159	147	130	161	275	254	87	34	-	71	1,651
1986	128	125	112	103	92	131	411	477	209	68	21	17	1,892
1987	170	169	161	151	137	167	598	1,019	600	218	97	84	3,568

Table 3A-26. Granite Springs Reservoir Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1988	92	92	77	67	65	105	422	500	223	72	28	40	1,782
1989	33	48	34	23	29	77	323	325	122	32	15	99	1,160
1990	106	122	111	101	94	113	438	533	244	85	62	74	2,080
1991	140	126	119	109	97	65	382	1,073	1,165	438	182	137	4,029
1992	171	161	152	137	122	423	356	357	137	42	52	12	2,121
1993	26	32	17	14	10	405	728	762	547	193	100	132	2,965
1994	158	143	137	128	116	157	433	545	209	71	67	37	2,199
1995	84	95	86	77	73	71	26	1,187	1,119	415	128	122	3,480
1996	119	182	133	254	232	53	371	864	362	143	193	41	2,947
1997	52	117	75	121	45	272	542	647	356	221	380	195	3,021
1998	269	263	255	245	216	350	872	698	243	81	32	18	3,542
1999	212	231	205	187	166	233	746	1,151	320	98	34	89	3,669
2000	102	132	115	97	92	175	343	384	50	18	9	127	1,643
2001	74	87	77	68	62	84	395	680	78	27	25	20	1,676
2002	229	319	278	227	220	750	492	320	65	21	22	99	3,040
2003	61	79	67	57	53	94	650	1,004	558	195	81	68	2,966
2004	82	16	181	215	106	213	372	138	183	159	138	142	1,946
2005	241	190	159	94	85	131	296	614	2,825	371	276	114	5,396
2006	135	97	140	111	112	368	310	399	134	69	138	134	2,146
2007	179	100	108	45	158	361	382	344	134	138	138	122	2,210
2008	139	124	165	108	137	187	358	783	1,089	268	271	222	3,853
2009	162	175	180	179	198	254	925	1,037	449	111	239	195	4,103
2010	307	380	99	180	233	263	1,254	1,555	1,239	703	375	157	6,746
2011	275	231	266	226	208	317	659	1,320	772	423	170	143	5,010
2012	195	202	188	165	209	373	330	138	134	150	138	131	2,353
Average	113	117	105	95	90	150	502	821	526	190	92	76	2,877

Table 3A-27. Crystal Lake Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	14	15	13	12	11	19	113	151	54	8	4	9	422
1934	7	7	8	6	5	14	29	18	3	-	-	-	96
1935	1	2	2	2	2	4	14	201	167	26	6	5	432
1936	7	9	9	7	5	12	80	35	12	-	2	-	176
1937	2	5	4	2	3	12	45	43	58	10	2	-	184
1938	4	5	2	2	3	20	127	180	60	12	8	32	454
1939	20	14	5	3	2	10	100	105	20	2	-	-	281
1940	2	5	3	2	2	3	30	30	7	2	-	1	86
1941	4	2	2	2	2	4	84	151	79	44	25	8	404
1942	8	7	5	2	3	10	149	404	142	47	14	9	800
1943	55	45	23	7	4	6	79	203	113	28	5	2	568
1944	7	10	7	4	2	4	84	218	73	26	2	2	437
1945	7	7	5	3	3	4	59	131	95	30	21	5	369
1946	22	17	16	14	13	17	76	98	56	17	2	11	357
1947	24	26	25	23	21	25	66	208	138	41	15	11	621
1948	7	6	4	2	2	7	92	78	17	4	1	-	218
1949	3	4	3	2	2	7	101	183	256	53	11	8	632
1950	18	16	10	6	5	10	38	66	44	23	4	8	247
1951	7	8	5	4	2	5	43	63	31	11	5	-	182
1952	6	6	5	4	4	5	129	161	43	8	3	1	374
1953	3	5	5	5	5	10	35	54	30	10	11	1	173
1954	2	3	3	3	3	4	32	16	2	-	-	-	67
1955	-	-	1	2	2	5	31	24	17	1	-	-	81
1956	-	-	2	2	1	4	31	55	6	-	-	-	99
1957	-	-	2	2	4	5	81	368	128	41	13	8	650
1958	17	14	10	7	9	13	94	154	45	33	8	3	405
1959	6	10	5	5	4	8	82	153	57	17	4	2	351
1960	8	6	5	5	4	27	38	28	8	1	-	-	128
1961	-	5	4	3	2	9	47	161	111	29	11	15	396
1962	17	17	11	7	18	21	110	105	72	20	2	3	403
1963	12	8	7	4	11	27	80	77	26	8	3	5	266
1964	9	18	20	15	15	15	56	150	140	101	50	15	603
1965	22	24	28	32	27	34	56	169	356	138	51	41	976
1966	38	31	32	16	14	26	45	35	26	21	20	21	323
1967	19	54	67	66	56	56	71	137	188	113	29	33	888
1968	42	50	45	54	51	42	95	233	114	40	37	29	830
1969	33	28	26	32	28	41	125	90	125	103	20	25	674
1970	29	29	28	26	23	25	89	194	130	48	15	14	650
1971	25	23	23	20	19	23	83	146	88	32	11	17	509
1972	18	19	17	16	14	20	33	29	9	2	5	6	186
1973	14	12	11	9	8	14	65	84	41	16	3	12	287
1974	11	7	5	3	3	9	68	92	46	16	2	2	262
1975	15	15	14	13	11	17	59	71	32	11	9	6	274
1976	12	14	12	11	10	15	54	62	26	9	3	3	230
1977	5	8	7	5	5	11	41	40	14	6	5	1	149
1978	1	5	3	2	2	9	32	27	8	2	3	1	94
1979	9	12	10	8	8	12	84	149	91	32	18	11	444
1980	14	11	8	6	5	11	89	185	121	44	13	11	517
1981	17	18	17	15	14	18	69	95	49	18	10	7	345
1982	14	17	15	14	12	18	52	58	24	8	9	17	256
1983	20	20	20	18	16	19	87	290	227	86	30	16	847
1984	17	12	11	9	9	14	79	125	71	26	17	13	403
1985	23	23	22	20	18	22	38	35	12	5	-	10	226
1986	17	17	15	14	13	18	56	65	29	9	3	2	259
1987	23	23	22	20	19	23	82	140	83	30	14	11	488

Table 3A-27. Crystal Lake Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1988	13	13	11	9	9	14	58	68	31	10	4	5	244
1989	5	7	5	3	4	11	44	44	17	5	2	14	158
1990	14	17	15	14	13	16	60	73	33	11	8	10	283
1991	19	17	17	15	14	9	53	147	160	60	25	19	553
1992	23	22	21	19	17	58	49	49	19	6	7	2	290
1993	4	5	2	2	2	56	100	104	75	26	14	18	406
1994	22	20	19	17	16	22	59	74	29	10	9	5	301
1995	11	13	12	11	10	10	4	163	153	57	17	17	476
1996	17	25	18	35	32	8	51	119	50	20	26	6	404
1997	7	16	11	17	6	38	74	89	49	30	52	27	413
1998	37	36	35	34	29	48	119	95	33	11	5	2	485
1999	29	32	28	26	23	32	102	158	44	14	5	12	501
2000	14	18	16	14	13	24	47	53	7	2	2	17	226
2001	11	12	11	9	8	11	54	93	11	4	4	3	230
2002	32	44	38	31	30	103	68	44	9	3	3	14	416
2003	8	11	9	8	7	13	89	137	76	27	11	9	406
2004	11	2	11	17	4	18	34	-	7	3	-	3	109
2005	20	17	13	13	12	18	23	66	369	33	20	-	604
2006	4	-	4	-	1	32	24	36	-	-	-	-	101
2007	9	-	4	-	13	38	40	28	-	-	-	-	132
2008	3	5	12	6	10	10	31	89	131	18	18	13	345
2009	9	11	14	16	20	26	111	123	45	-	16	9	400
2010	29	41	14	9	15	19	154	194	151	79	40	4	749
2011	20	20	25	22	20	32	73	162	89	42	10	2	518
2012	11	16	14	12	18	35	29	-	-	2	-	-	137
Average	14	15	13	12	11	19	67	110	70	24	11	8	374

Table 3A-28. North Crow Reservoir Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	49	53	46	40	38	59	455	608	218	29	15	38	1,647
1934	26	28	34	25	20	55	116	72	11	-	-	-	387
1935	3	8	8	11	10	16	56	811	669	104	26	19	1,739
1936	26	37	37	26	20	47	320	139	48	1	6	-	707
1937	5	17	16	11	12	47	180	173	232	41	5	-	739
1938	14	19	11	8	12	78	512	725	243	50	29	130	1,829
1939	83	56	21	13	10	39	401	424	83	8	-	-	1,136
1940	5	18	13	8	8	13	122	121	28	9	-	4	347
1941	16	5	5	5	7	16	339	608	317	177	100	35	1,629
1942	34	28	18	11	12	39	602	1,628	569	190	59	37	3,224
1943	219	182	92	26	14	23	316	817	455	113	21	9	2,286
1944	26	39	29	16	10	16	339	878	293	105	10	6	1,765
1945	28	29	21	13	12	16	240	527	381	122	84	19	1,490
1946	89	68	62	57	53	66	306	395	224	68	8	42	1,436
1947	95	104	100	94	85	99	266	838	557	164	62	41	2,503
1948	26	23	16	11	8	26	372	314	67	15	3	-	880
1949	14	15	13	8	10	26	407	737	1,030	211	46	32	2,547
1950	71	63	39	23	19	39	155	266	178	90	14	32	989
1951	26	31	21	16	10	18	171	255	123	43	21	1	735
1952	23	23	18	16	15	21	519	650	173	35	11	3	1,507
1953	13	17	18	21	19	39	143	218	119	39	46	4	695
1954	8	13	13	13	12	16	129	65	6	1	-	-	274
1955	-	1	2	5	7	21	122	97	66	3	-	-	324
1956	-	1	5	5	2	16	124	221	25	-	-	-	399
1957	-	2	5	8	14	21	326	1,479	515	167	53	30	2,618
1958	70	56	39	26	35	53	378	620	182	132	29	12	1,632
1959	23	38	21	18	17	30	330	615	230	66	15	9	1,411
1960	33	26	21	18	15	110	152	111	29	2	-	-	517
1961	-	20	16	13	10	37	188	650	449	114	46	62	1,602
1962	69	68	41	27	74	86	441	423	290	81	11	12	1,623
1963	47	35	28	15	43	108	321	309	106	29	12	20	1,072
1964	38	73	78	59	59	61	226	604	563	407	199	61	2,426
1965	89	98	113	127	109	134	224	680	1,433	555	205	166	3,931
1966	151	124	127	64	53	104	183	141	107	83	80	84	1,300
1967	77	218	270	266	226	222	288	554	755	458	116	132	3,580
1968	168	200	180	217	206	170	380	941	459	160	149	114	3,342
1969	133	113	106	126	112	167	503	362	500	413	80	101	2,715
1970	39	38	37	35	32	34	120	260	174	65	20	19	872
1971	44	42	40	37	34	41	149	263	158	57	19	29	911
1972	47	50	47	43	38	51	88	77	24	5	12	16	495
1973	24	22	19	16	14	23	113	146	71	27	5	20	498
1974	7	5	3	2	2	13	56	56	28	10	2	2	183
1975	28	28	26	23	21	29	107	129	59	21	16	11	497
1976	23	26	22	20	18	29	103	116	50	17	7	6	435
1977	32	48	40	32	30	68	247	238	86	35	32	5	890
1978	14	76	51	29	35	145	492	422	131	40	42	10	1,485
1979	48	62	51	44	41	65	442	788	477	172	95	59	2,342
1980	43	33	25	17	17	34	270	563	370	134	39	32	1,577
1981	38	40	38	35	31	41	157	215	110	41	21	16	780
1982	61	69	62	57	50	74	217	239	100	32	38	69	1,067
1983	22	22	20	20	17	20	92	306	240	91	32	17	898
1984	53	38	34	29	29	44	245	391	221	82	52	39	1,256
1985	108	111	105	97	86	107	182	168	58	23	11	47	1,100
1986	71	69	62	57	51	72	227	263	115	38	12	10	1,044
1987	73	72	69	65	59	71	257	437	257	94	41	36	1,530

Table 3A-28. North Crow Reservoir Natural Runoff with 75% of Historic Snow Pack													
[acre-feet]													
Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1988	86	86	73	62	61	99	397	470	210	68	26	38	1,675
1989	50	74	52	35	44	118	493	495	187	48	23	151	1,768
1990	59	67	61	56	52	62	242	293	134	47	34	41	1,145
1991	65	59	55	50	45	60	284	746	542	203	85	63	2,255
1992	77	73	69	62	55	71	171	169	62	19	23	5	857
1993	13	15	8	2	5	33	308	473	260	92	47	63	1,317
1994	92	83	80	74	68	86	231	272	121	41	39	21	1,207
1995	29	32	29	26	25	38	203	524	377	140	43	41	1,505
1996	32	32	31	28	25	33	94	104	44	15	13	21	470
1997	59	62	57	51	46	68	184	188	73	31	59	104	980
1998	127	124	121	116	102	112	221	259	115	38	15	8	1,356
1999	29	32	28	26	23	32	95	105	44	14	5	12	442
2000	32	42	36	31	29	56	86	63	16	6	3	41	440
2001	58	68	60	53	47	74	184	173	61	21	20	16	833
2002	26	37	32	26	26	50	57	37	8	2	2	11	314
2003	29	38	32	27	26	45	313	484	269	94	39	33	1,430
2004	40	42	35	29	28	54	194	202	80	25	8	46	783
2005	57	56	51	45	41	63	236	276	123	39	18	9	1,014
2006	46	52	46	41	39	59	115	96	28	5	-	1	529
2007	7	21	12	7	8	32	165	157	56	23	26	44	559
2008	112	108	102	98	85	100	231	267	118	36	46	49	1,353
2009	64	69	64	56	50	72	224	253	109	32	13	7	1,014
2010	54	55	48	42	40	62	314	488	272	98	46	22	1,542
2011	56	60	53	48	44	66	205	219	89	29	7	4	880
2012	41	42	35	28	29	56	154	144	50	14	-	-	593
Average	49	53	46	40	38	59	245	389	224	80	34	32	1,289

Table 3A-29. Upper North Crow Reservoir Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	8	8	7	6	6	9	71	95	35	5	2	6	258
1934	4	5	5	4	3	8	18	11	2	-	-	-	59
1935	1	2	2	2	2	2	9	127	105	17	4	3	273
1936	5	6	6	4	3	8	50	22	8	-	1	-	111
1937	1	3	2	2	2	8	29	27	36	6	1	-	115
1938	2	3	2	2	2	12	80	113	38	8	5	20	286
1939	13	9	3	2	2	6	63	66	13	2	-	-	178
1940	1	3	2	2	2	2	19	19	5	2	-	1	56
1941	2	1	1	1	1	2	53	95	50	28	16	5	254
1942	5	5	3	2	2	6	95	255	89	30	9	6	506
1943	35	29	14	4	2	4	50	128	71	17	3	2	358
1944	4	6	5	2	2	2	53	137	46	17	2	1	275
1945	5	5	3	2	2	2	38	83	60	19	14	3	233
1946	14	11	10	9	8	11	48	62	35	11	2	7	226
1947	15	17	16	15	14	16	41	131	87	26	10	7	393
1948	4	4	2	2	2	4	59	50	11	2	1	-	138
1949	2	2	2	2	2	4	64	116	161	33	7	5	399
1950	11	10	6	4	3	6	24	41	28	14	2	5	155
1951	5	5	3	2	2	3	27	40	20	7	3	-	115
1952	4	4	3	2	2	3	81	102	27	5	2	1	236
1953	2	2	3	3	3	6	23	35	19	6	8	1	110
1954	2	2	2	2	2	2	20	11	1	-	-	-	44
1955	-	-	1	1	1	3	20	15	11	1	-	-	51
1956	-	-	1	1	1	2	20	35	4	-	-	-	62
1957	-	-	1	2	2	3	51	232	81	26	8	5	410
1958	11	9	6	4	5	8	59	98	29	21	5	2	256
1959	4	6	3	3	2	5	52	96	36	11	2	2	221
1960	5	4	3	3	2	17	24	17	5	-	-	-	80
1961	-	3	2	2	2	6	29	102	71	18	8	10	252
1962	11	11	7	5	11	14	69	66	46	13	2	2	254
1963	8	5	5	2	7	17	50	49	17	5	2	3	168
1964	6	11	12	9	9	10	35	95	89	64	32	10	380
1965	14	15	18	20	17	21	35	107	224	87	32	26	617
1966	24	20	20	10	8	17	29	22	17	13	13	14	203
1967	12	35	42	42	35	35	45	87	119	72	18	21	562
1968	26	32	29	34	32	26	59	147	72	25	23	18	523
1969	21	18	17	20	17	26	79	57	79	65	13	16	426
1970	6	6	6	5	5	5	19	41	27	10	3	3	136
1971	7	7	6	6	5	6	23	41	25	9	3	5	143
1972	8	8	8	7	6	8	14	12	4	1	2	2	77
1973	4	3	3	2	2	4	18	23	11	5	1	3	78
1974	1	1	1	-	1	2	9	9	5	2	-	-	29
1975	5	5	4	4	3	5	17	20	9	3	2	2	77
1976	4	4	4	3	3	5	16	18	8	2	1	1	67
1977	5	8	6	5	5	11	39	38	14	5	5	1	140
1978	2	12	8	5	5	23	77	66	20	6	7	2	233
1979	8	10	8	7	6	11	69	124	75	27	15	9	368
1980	7	5	4	3	3	5	42	88	58	21	6	5	247
1981	6	6	6	5	5	7	25	34	17	6	3	2	122
1982	10	11	10	9	8	11	34	38	16	5	6	11	167
1983	3	4	3	3	3	3	14	48	38	14	5	3	141
1984	8	6	5	5	5	7	38	62	35	13	8	6	197
1985	17	17	17	15	14	17	29	26	9	4	2	8	173
1986	11	11	10	9	8	11	35	41	18	6	2	2	164
1987	11	11	11	10	9	11	41	68	41	14	7	6	239

Table 3A-29. Upper North Crow Reservoir Natural Runoff with 75% of Historic Snow Pack													
[acre-feet]													
Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1988	14	14	11	10	10	16	62	74	33	11	4	6	263
1989	8	11	8	5	7	19	77	77	29	8	4	23	277
1990	9	11	10	9	8	10	38	46	21	8	5	6	179
1991	11	9	8	8	7	10	44	117	85	32	14	10	353
1992	12	11	11	10	8	11	27	26	10	3	4	1	134
1993	2	2	2	-	1	5	48	74	41	14	8	10	206
1994	14	13	13	11	11	14	36	43	19	7	6	3	188
1995	5	5	5	4	4	6	32	82	59	22	7	7	236
1996	5	5	5	5	4	5	15	17	7	2	2	3	74
1997	9	10	9	8	8	11	29	29	11	5	9	17	153
1998	20	20	19	18	16	17	35	41	18	6	2	2	212
1999	5	5	5	4	4	5	15	17	7	2	1	2	71
2000	5	7	6	5	5	9	14	10	2	1	1	6	69
2001	9	11	9	8	8	11	29	27	10	3	3	2	129
2002	5	6	5	4	4	8	9	6	2	1	1	2	51
2003	5	6	5	4	4	7	49	76	42	15	6	5	224
2004	6	7	5	5	4	8	30	32	13	4	1	7	123
2005	9	9	8	7	6	10	37	43	19	6	3	1	159
2006	7	8	7	6	6	9	18	15	4	1	-	0	83
2007	1	3	2	1	1	5	26	25	9	4	4	7	88
2008	18	17	16	15	13	16	36	42	18	6	7	8	212
2009	10	11	10	9	8	11	35	40	17	5	2	1	159
2010	8	9	7	7	6	10	49	76	43	15	7	3	241
2011	9	9	8	8	7	10	32	34	14	5	1	1	138
2012	6	7	5	4	5	9	24	22	8	2	-	-	93
Average	8	8	7	6	6	9	38	61	35	13	5	5	202

Table 3A-30. Brush Creek Collection Area Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	13	14	13	11	10	16	124	166	59	8	4	11	449
1934	7	8	9	7	5	15	32	20	3	-	-	-	104
1935	1	2	2	3	2	5	15	221	182	29	7	5	474
1936	8	10	10	7	5	13	87	38	13	-	2	-	191
1937	2	5	5	3	3	13	49	47	63	11	2	-	201
1938	4	5	3	2	3	21	140	198	66	14	8	35	499
1939	23	15	6	4	2	11	110	116	23	2	-	-	310
1940	2	5	4	2	2	4	33	33	8	2	-	1	95
1941	5	2	2	2	2	5	92	166	86	48	27	9	444
1942	9	8	5	3	3	11	164	444	155	52	16	10	879
1943	60	50	25	7	4	7	86	223	124	31	6	2	623
1944	7	11	8	5	3	5	92	239	80	29	3	2	481
1945	8	8	6	4	3	5	65	144	104	33	23	5	407
1946	24	19	17	16	14	18	83	107	61	19	2	11	392
1947	26	29	27	26	23	27	72	229	152	44	17	11	682
1948	7	6	5	3	2	7	101	86	18	5	1	-	239
1949	4	5	4	2	2	7	111	201	281	58	13	8	695
1950	20	17	11	7	5	11	42	72	49	25	4	9	270
1951	8	8	6	5	2	5	47	70	34	11	6	-	201
1952	7	6	5	5	4	6	142	177	47	10	3	1	412
1953	4	5	5	6	5	11	39	59	32	11	13	1	190
1954	2	4	4	4	3	5	35	18	2	-	-	-	76
1955	-	-	1	2	2	6	34	26	18	1	-	-	89
1956	-	-	2	2	1	5	34	61	7	-	-	-	110
1957	-	-	2	2	4	6	89	404	140	45	14	8	714
1958	19	15	11	7	10	14	103	170	50	36	8	3	444
1959	6	11	6	5	5	8	90	168	63	18	4	2	386
1960	9	7	6	5	4	30	41	30	8	1	-	-	141
1961	-	5	5	4	2	10	51	177	122	31	13	17	437
1962	19	19	11	8	20	23	120	116	79	22	3	3	442
1963	13	10	8	5	12	29	88	84	29	8	3	5	293
1964	11	20	21	17	17	17	62	165	154	111	54	17	662
1965	24	26	31	35	30	37	61	185	391	152	56	45	1,071
1966	41	34	35	17	14	29	50	38	29	23	22	23	354
1967	21	59	74	73	62	61	79	151	206	125	32	36	976
1968	46	55	49	59	56	47	104	257	125	44	41	32	912
1969	36	31	29	35	31	46	137	99	137	113	22	28	741
1970	11	11	10	10	8	9	33	71	47	17	5	5	237
1971	12	11	11	10	9	11	41	71	43	16	5	8	248
1972	13	14	13	11	11	14	24	21	7	2	3	5	136
1973	7	6	5	5	4	6	31	40	20	8	2	5	137
1974	2	2	1	1	1	4	15	15	8	3	1	1	51
1975	8	8	7	6	6	8	29	35	16	6	5	3	136
1976	6	7	6	5	5	8	28	32	14	5	2	2	118
1977	9	13	11	9	8	19	68	65	23	9	9	2	244
1978	4	20	14	8	10	40	134	116	36	11	11	3	407
1979	13	17	14	12	11	18	121	215	131	47	26	17	639
1980	12	9	7	5	5	9	74	153	101	37	11	9	429
1981	11	11	11	10	8	11	43	59	30	11	6	5	215
1982	17	19	17	16	14	20	59	65	27	9	11	19	292
1983	6	6	5	5	5	5	26	83	65	25	8	5	244

Table 3A-30. Brush Creek Collection Area Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1984	14	11	9	8	8	12	67	107	60	23	14	11	342
1985	29	30	29	26	23	29	50	46	16	6	3	13	299
1986	20	19	17	16	14	20	62	72	32	11	3	2	285
1987	20	20	19	17	16	20	70	119	71	26	11	10	416
1988	23	23	20	17	17	27	108	128	57	19	8	11	457
1989	14	20	14	10	12	32	134	135	51	14	6	41	483
1990	16	18	17	15	14	17	66	80	37	13	9	11	313
1991	18	16	15	14	12	17	77	203	148	56	23	17	615
1992	21	20	19	17	15	20	47	46	17	5	7	2	234
1993	4	5	2	1	2	9	84	129	71	25	13	17	361
1994	25	23	22	20	19	23	63	74	33	11	11	6	329
1995	8	9	8	7	7	11	56	143	103	38	12	11	410
1996	9	9	8	8	7	9	26	29	12	4	4	6	129
1997	16	17	16	14	13	19	50	52	20	8	17	29	269
1998	35	34	33	32	28	31	60	71	32	11	4	2	370
1999	8	8	8	7	6	9	26	29	12	4	2	3	119
2000	9	11	10	8	8	15	23	17	5	2	1	11	120
2001	16	18	17	14	13	20	50	47	17	6	5	5	227
2002	8	10	9	8	7	14	16	10	2	1	1	3	86
2003	8	10	9	7	7	12	85	132	73	26	11	9	390
2004	11	11	9	8	8	15	53	55	22	7	2	13	214
2005	16	15	14	12	11	17	64	75	34	11	5	3	276
2006	13	14	12	11	11	16	31	26	8	1	-	0	144
2007	2	6	3	2	2	9	45	43	15	6	7	12	152
2008	31	29	28	27	23	27	63	73	32	10	13	13	369
2009	17	19	17	15	14	20	61	69	30	9	4	2	276
2010	15	15	13	11	11	17	86	133	74	27	13	6	420
2011	15	16	14	13	12	18	56	60	24	8	2	1	240
2012	11	12	9	8	8	15	42	39	14	4	-	-	162
Average	13	14	13	11	10	16	67	106	61	22	9	9	351

Table 3A-31. South Crow Reservoir Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1933	32	28	23	20	19	37	329	269	75	5	-	29	866
1934	11	21	20	18	25	75	89	44	1	-	-	-	302
1935	-	-	-	4	3	8	44	506	391	82	26	36	1,099
1936	47	20	20	14	11	26	228	63	20	-	-	-	449
1937	-	3	11	7	8	35	125	91	151	31	1	-	461
1938	7	13	11	8	13	96	371	211	62	5	-	40	837
1939	38	32	11	7	5	22	321	164	14	-	-	-	613
1940	-	-	3	39	36	67	89	42	7	-	-	-	283
1941	29	-	-	-	-	2	257	329	140	98	98	47	998
1942	85	-	-	-	-	2	496	944	329	140	80	67	2,141
1943	278	-	-	-	-	2	331	730	409	110	56	35	1,949
1944	52	50	-	-	-	2	242	440	112	85	7	4	993
1945	37	14	11	6	6	8	169	201	151	65	47	19	733
1946	45	30	27	25	23	29	218	175	88	34	4	20	716
1947	49	11	11	10	9	11	188	370	236	86	31	20	1,031
1948	44	26	17	11	8	30	225	125	30	2	-	-	518
1949	-	35	30	18	22	63	179	140	411	74	35	34	1,041
1950	50	46	28	16	13	28	107	128	75	45	13	13	560
1951	21	23	18	14	8	14	140	135	84	15	23	2	495
1952	33	32	28	23	17	23	407	275	47	4	-	-	890
1953	1	15	18	23	21	50	125	109	50	39	21	-	473
1954	8	22	23	23	21	28	72	43	5	-	-	-	245
1955	-	-	5	5	8	18	117	51	47	3	-	-	253
1956	-	-	2	5	5	18	152	138	20	-	-	-	338
1957	-	-	3	9	13	23	284	574	155	66	21	20	1,167
1958	39	35	28	23	25	41	331	267	68	80	47	21	1,004
1959	28	35	28	23	21	39	378	568	189	58	22	17	1,405
1960	48	35	28	23	17	127	136	86	28	1	-	-	529
1961	-	18	18	14	13	41	143	391	201	63	26	37	964
1962	46	40	34	29	72	71	277	146	107	32	5	-	857
1963	34	43	27	5	13	88	289	142	44	-	2	-	686
1964	4	17	5	4	2	6	137	152	76	3	-	-	404
1965	-	-	-	-	-	2	60	95	838	251	83	82	1,409
1966	95	82	75	59	34	84	110	53	18	1	-	-	610
1967	-	12	9	-	1	56	165	173	171	123	9	12	731
1968	32	65	30	30	34	61	156	352	151	16	9	5	938
1969	38	29	28	33	23	67	149	116	55	8	-	-	543
1970	29	29	28	26	23	25	258	345	221	50	15	14	1,062
1971	33	32	30	28	25	31	239	260	145	44	14	22	902
1972	49	51	48	44	38	53	91	52	13	5	11	15	468
1973	32	29	24	21	19	31	184	150	62	36	6	27	620
1974	29	20	12	8	8	24	193	163	71	45	6	6	583
1975	35	36	33	29	27	38	167	128	49	26	20	14	602
1976	32	37	32	27	26	42	153	110	39	23	8	8	537
1977	20	30	24	20	18	43	114	71	20	21	20	3	402
1978	5	27	18	10	12	53	87	49	11	14	14	3	302
1979	13	17	14	11	11	17	241	266	149	48	26	16	827
1980	14	11	8	5	5	11	255	329	205	47	13	11	912
1981	33	35	32	30	26	35	197	170	76	35	18	14	701
1982	35	40	35	32	29	43	146	103	35	17	21	40	575
1983	8	8	7	7	6	7	251	515	406	34	11	6	1,264
1984	28	20	18	15	14	23	225	224	115	44	28	20	774
1985	56	57	53	50	44	54	104	62	17	11	5	23	534
1986	43	41	37	34	30	44	159	116	43	22	7	5	580
1987	33	33	31	29	26	32	235	248	134	43	18	16	878

Table 3A-31. South Crow Reservoir Natural Runoff with 75% of Historic Snow Pack

[acre-feet]

Year	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Annual
1988	33	33	28	23	23	38	164	122	46	26	10	14	558
1989	16	24	17	11	14	39	124	80	24	15	7	51	420
1990	33	38	35	32	29	35	170	130	50	26	19	23	617
1991	14	12	11	11	9	13	261	424	302	46	18	14	1,133
1992	51	47	45	41	35	47	119	75	22	11	14	3	509
1993	6	8	4	1	2	17	218	209	104	50	25	33	675
1994	50	45	43	40	36	47	162	121	45	22	20	11	640
1995	8	9	8	7	7	11	262	416	293	41	12	11	1,083
1996	43	43	41	36	32	44	147	104	36	19	17	27	587
1997	35	37	34	30	27	41	128	84	26	18	35	64	558
1998	65	63	61	59	51	56	162	120	45	18	7	4	710
1999	40	44	38	35	31	44	147	103	35	18	6	16	557
2000	27	36	31	26	25	48	56	27	5	5	2	35	320
2001	37	43	38	33	29	47	109	66	18	13	11	9	452
2002	26	36	32	26	25	50	38	17	2	2	2	11	265
2003	13	17	15	12	12	20	221	212	110	42	18	15	706
2004	29	31	25	22	20	39	134	89	28	18	6	34	476
2005	35	34	31	28	25	39	164	121	46	24	11	6	564
2006	39	43	38	34	33	50	77	42	5	4	-	1	368
2007	7	21	12	7	8	33	113	69	17	23	26	44	382
2008	55	53	49	48	41	49	161	117	44	18	22	24	681
2009	38	41	38	34	30	43	156	111	40	19	8	4	564
2010	23	23	20	17	17	26	221	214	112	41	19	9	742
2011	37	40	35	32	29	44	142	96	32	19	5	3	514
2012	35	37	30	25	26	49	105	63	14	12	-	-	397
Average	32	28	23	20	19	37	184	193	105	34	16	16	707

Appendix 3-B
Well Hydrographs and Drought Monitoring Plan

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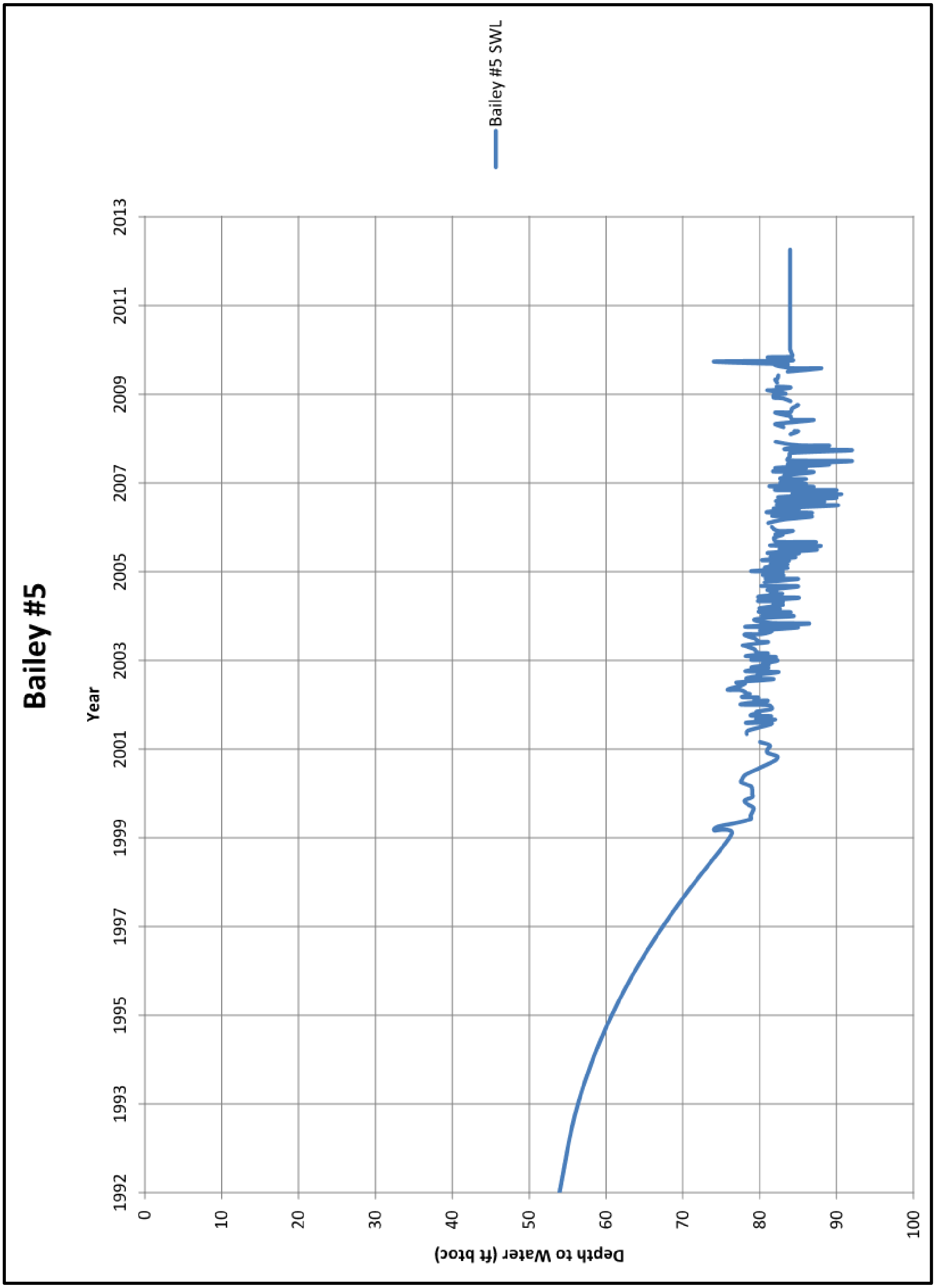
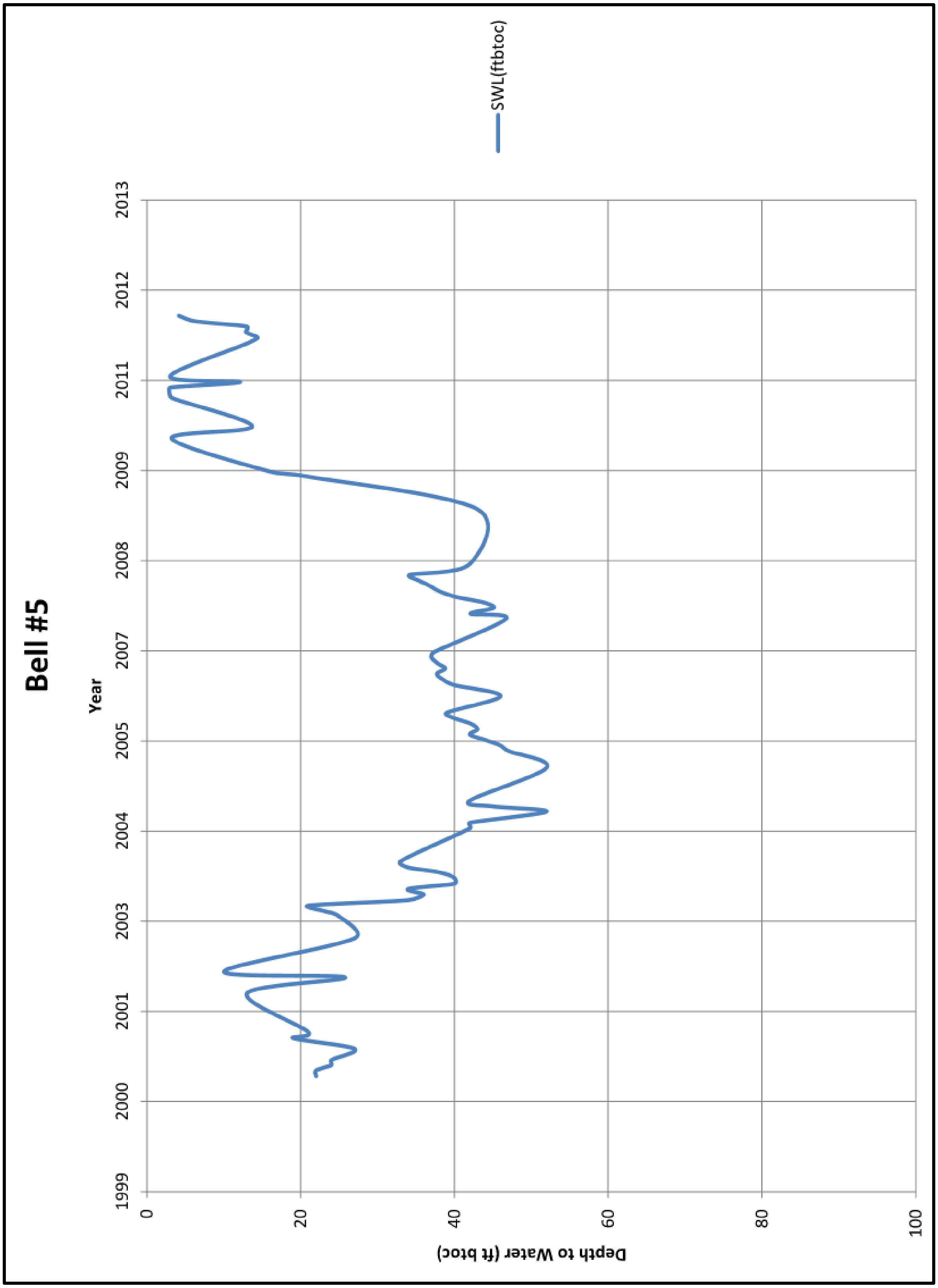
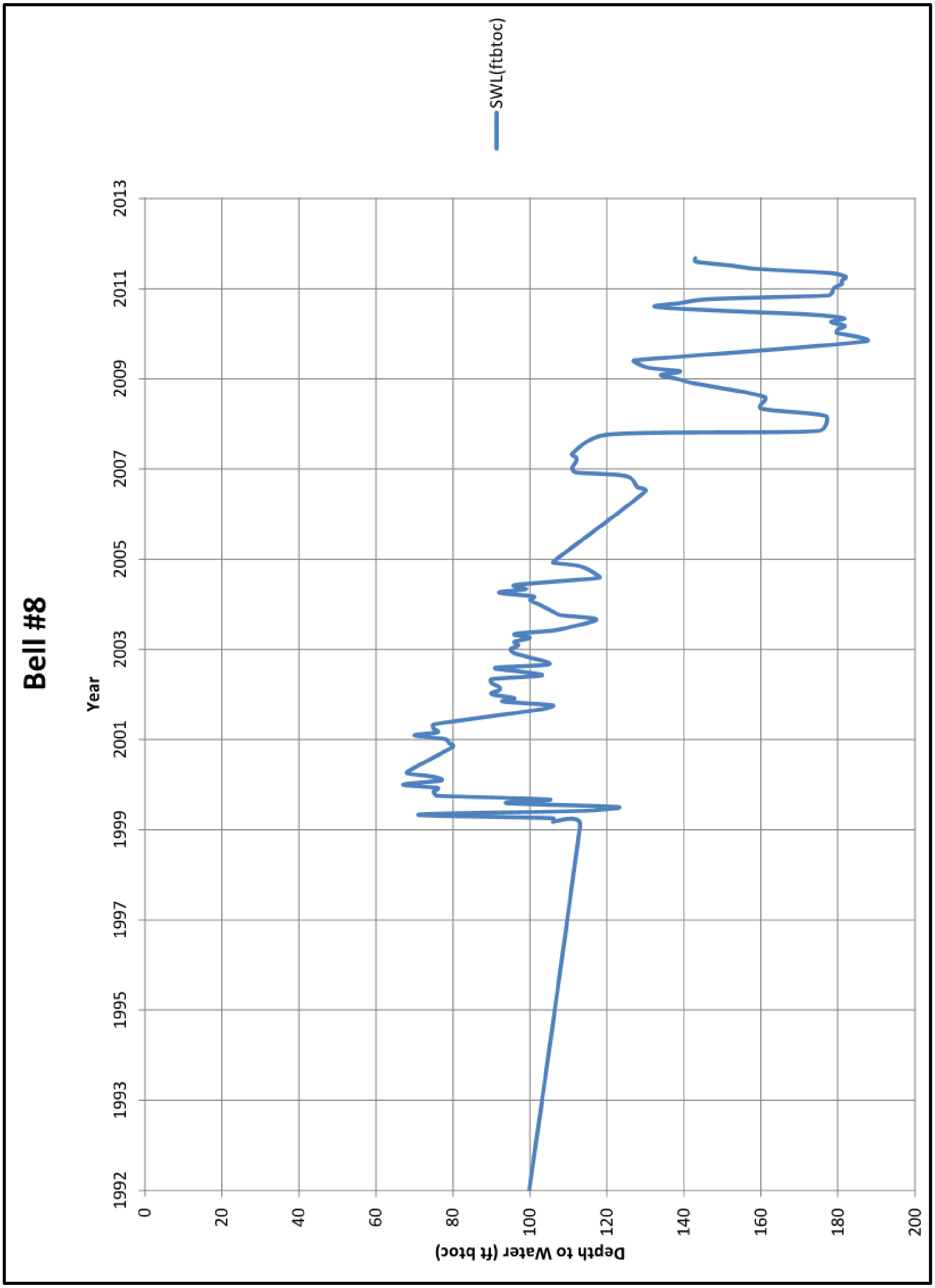

Bailey #5
Year
Depth to Water (ft btoc)
Bailey #5 SWL
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
0
10
20
30
40
50
60
70
80
90
100


Bell #5
SWL(ftbtoc)
Year
Depth to Water (ft btoc)
1999
2000
2001
2003
2004
2005
2007
2008
2009
2011
2012
2013
0
20
40
60
80
100


Bell #8
SWL(ftbtoc)
Year
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
Depth to Water (ft btoc)
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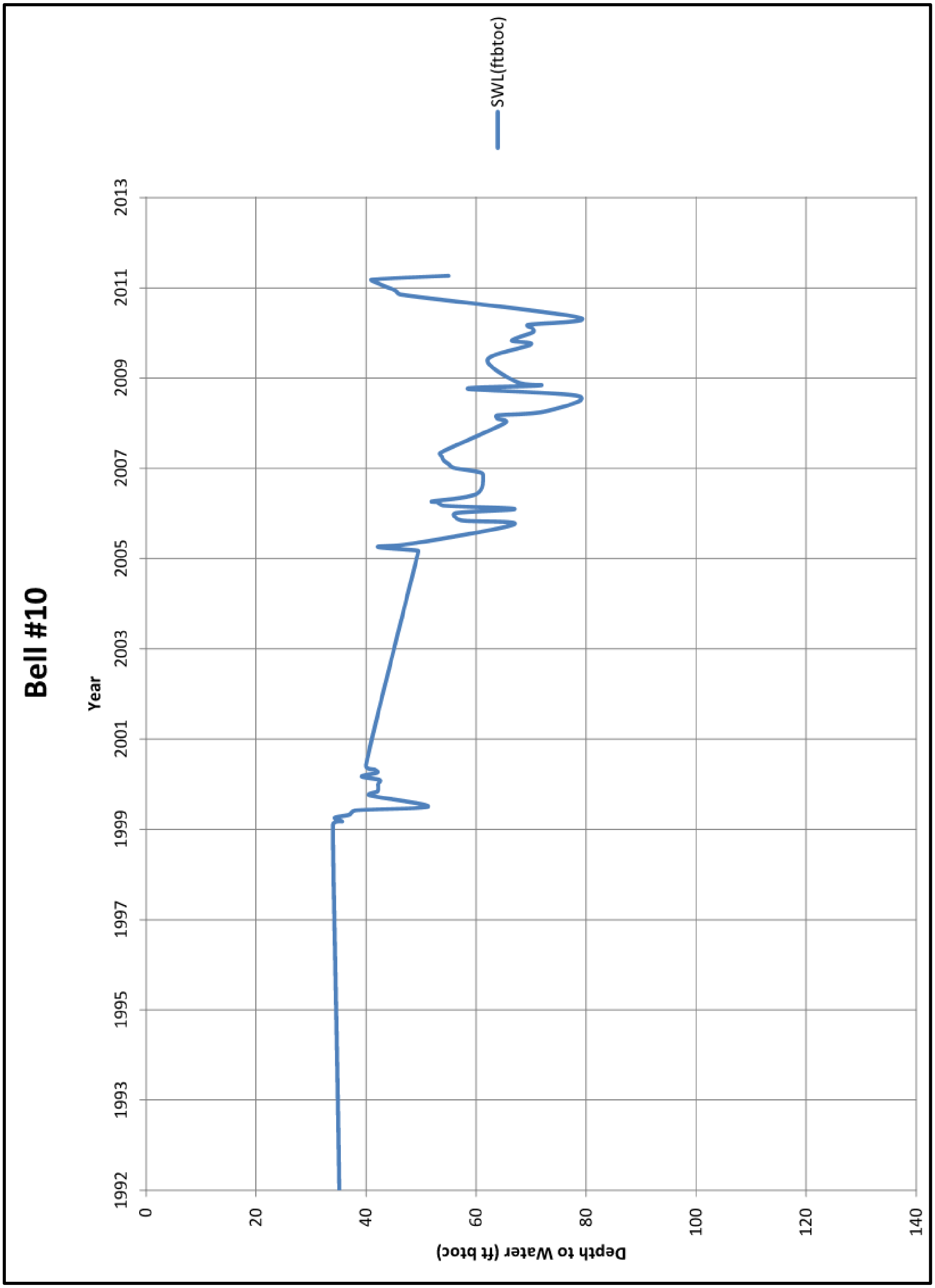
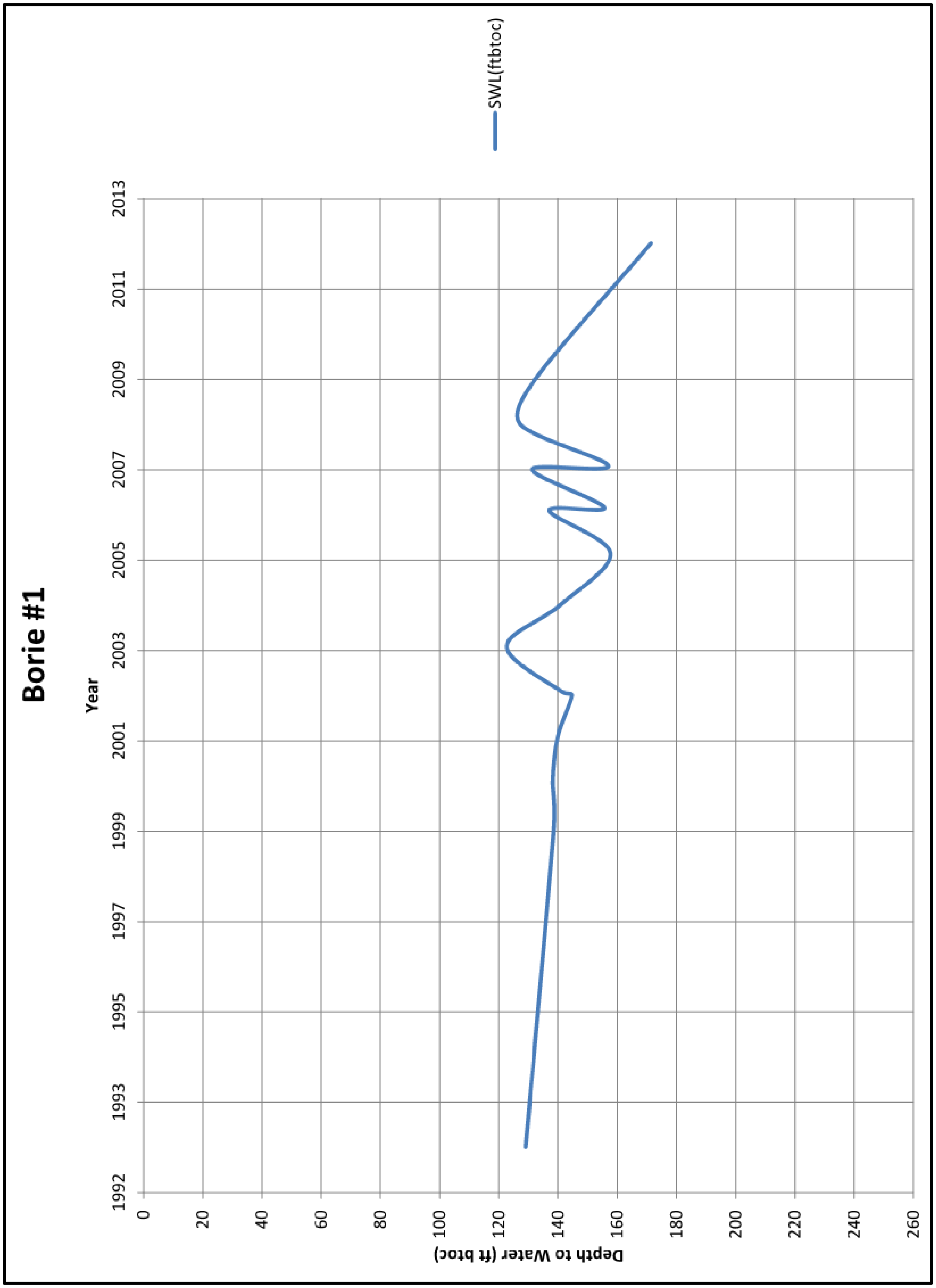
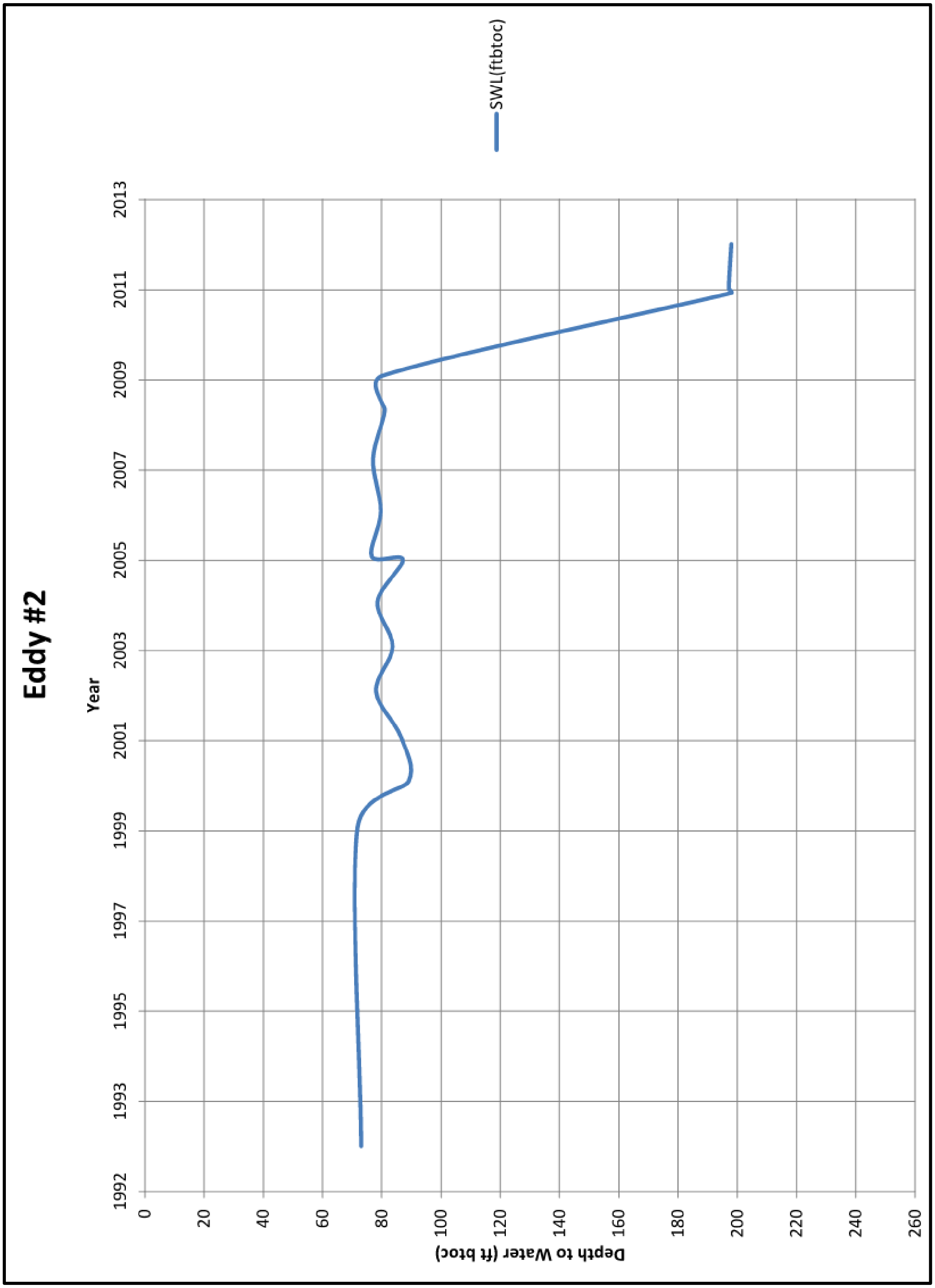
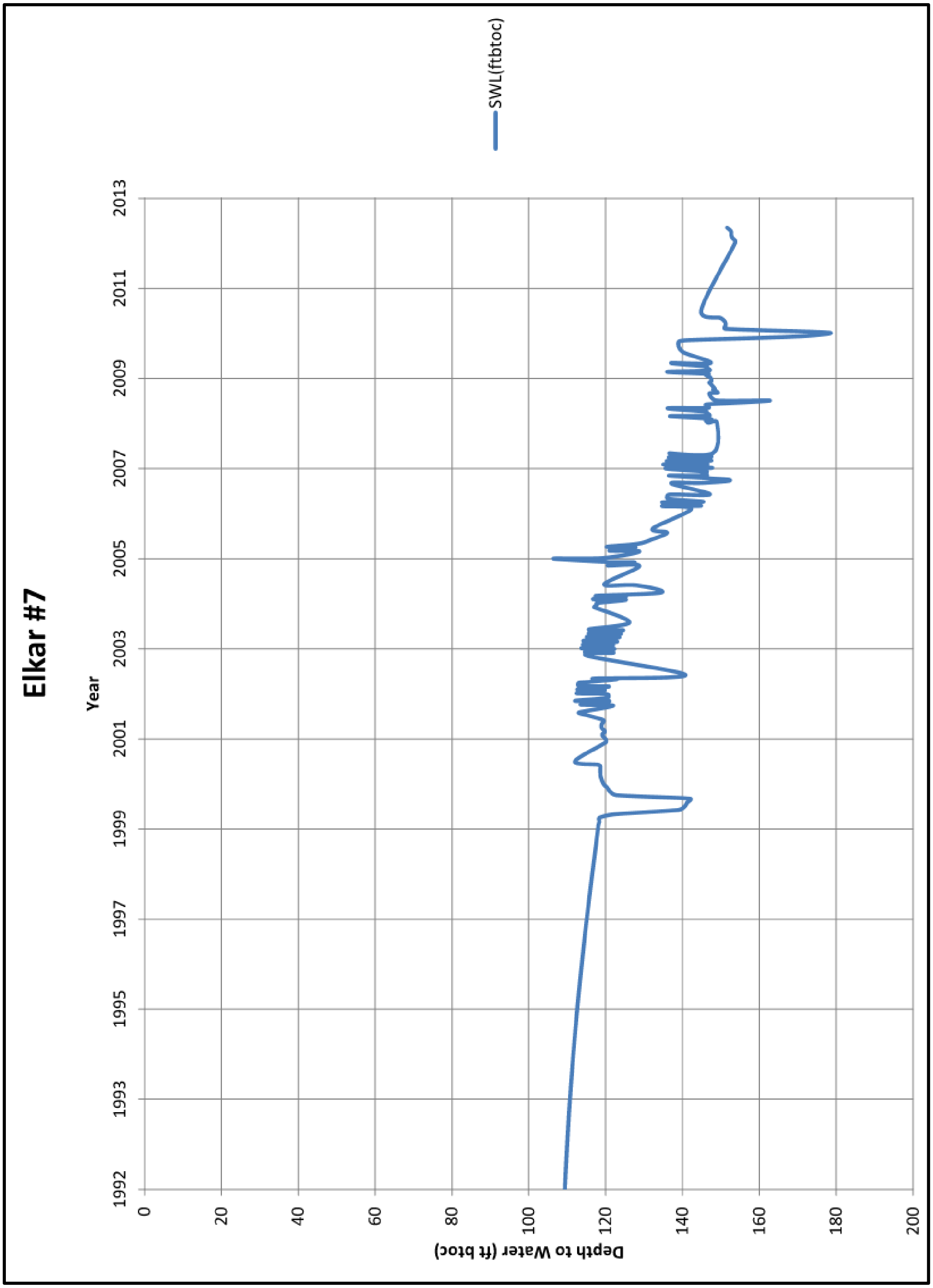
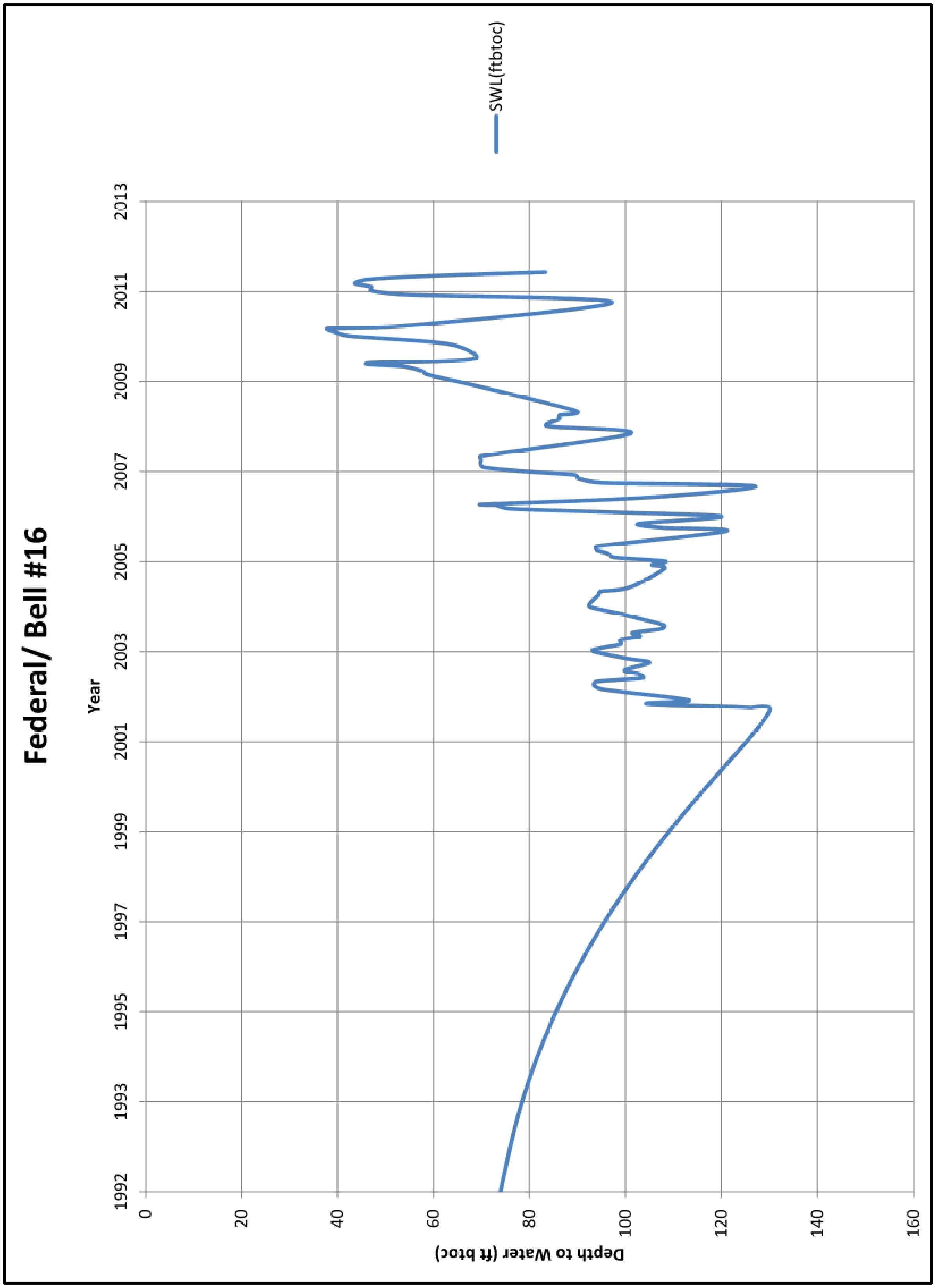
20
40
60
80
100
120
140
160
180
200


Bell #10
SWL(ftbtoc)
Year
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
Depth to Water (ft btoc)
0
20
40
60
80
100
120
140


Borie #1
SWL(ftbtoc)
Year
Depth to Water (ft btoc)
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
0
20
40
60
80
100
120
140
160
180
200
220
240
260


Eddy #2
SWL(ftbtoc)
Year
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
Depth to Water (ft btoc)
0
20
40
60
80
100
120
140
160
180
200
220
240
260


Elkar #7
Year
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
0
20
40
60
80
100
120
140
160
180
200
Depth to Water (ft btoc)
SWL(ftbtoc)


Federal/ Bell #16
SWL(ftbtoc)
Year
Depth to Water (ft btoc)
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
0
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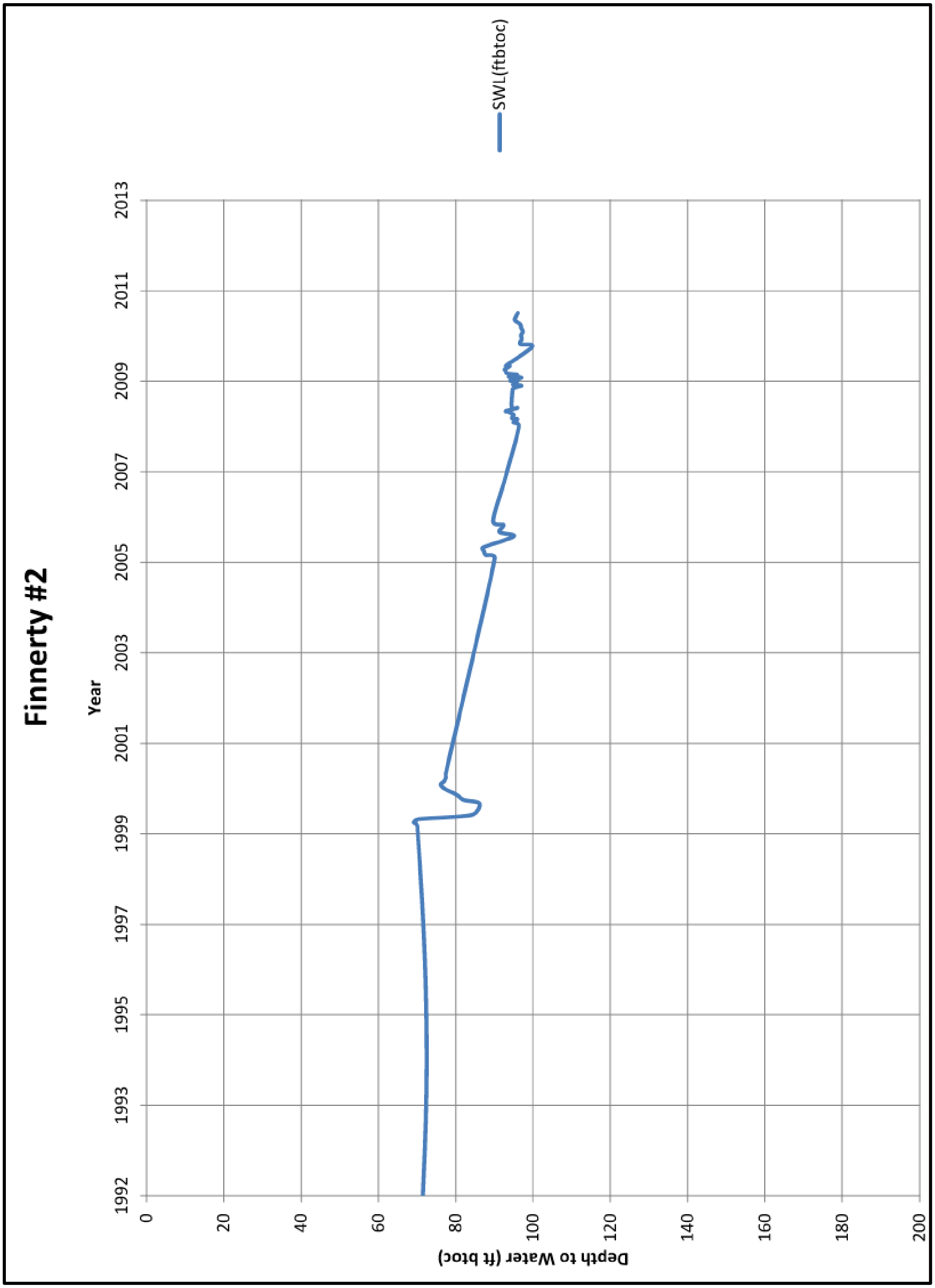
60
80
100
120
140
160


Finnerty #2
Year
Depth to Water (ft btoc)
SWL(ftbtoc)
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
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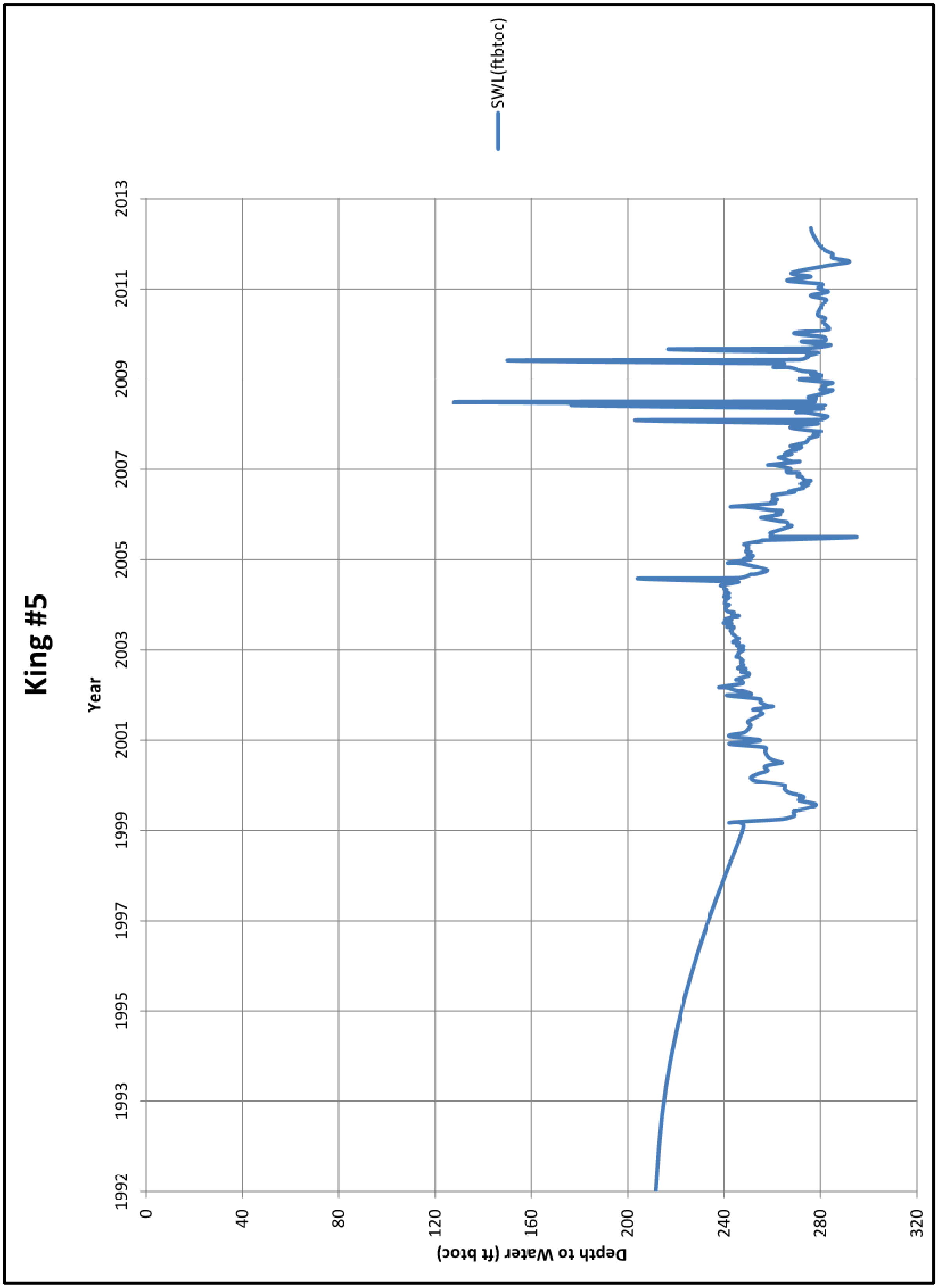
40
60
80
100
120
140
160
180
200


King #5
Year
Depth to Water (ft btoc)
SWL(ftbtoc)
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
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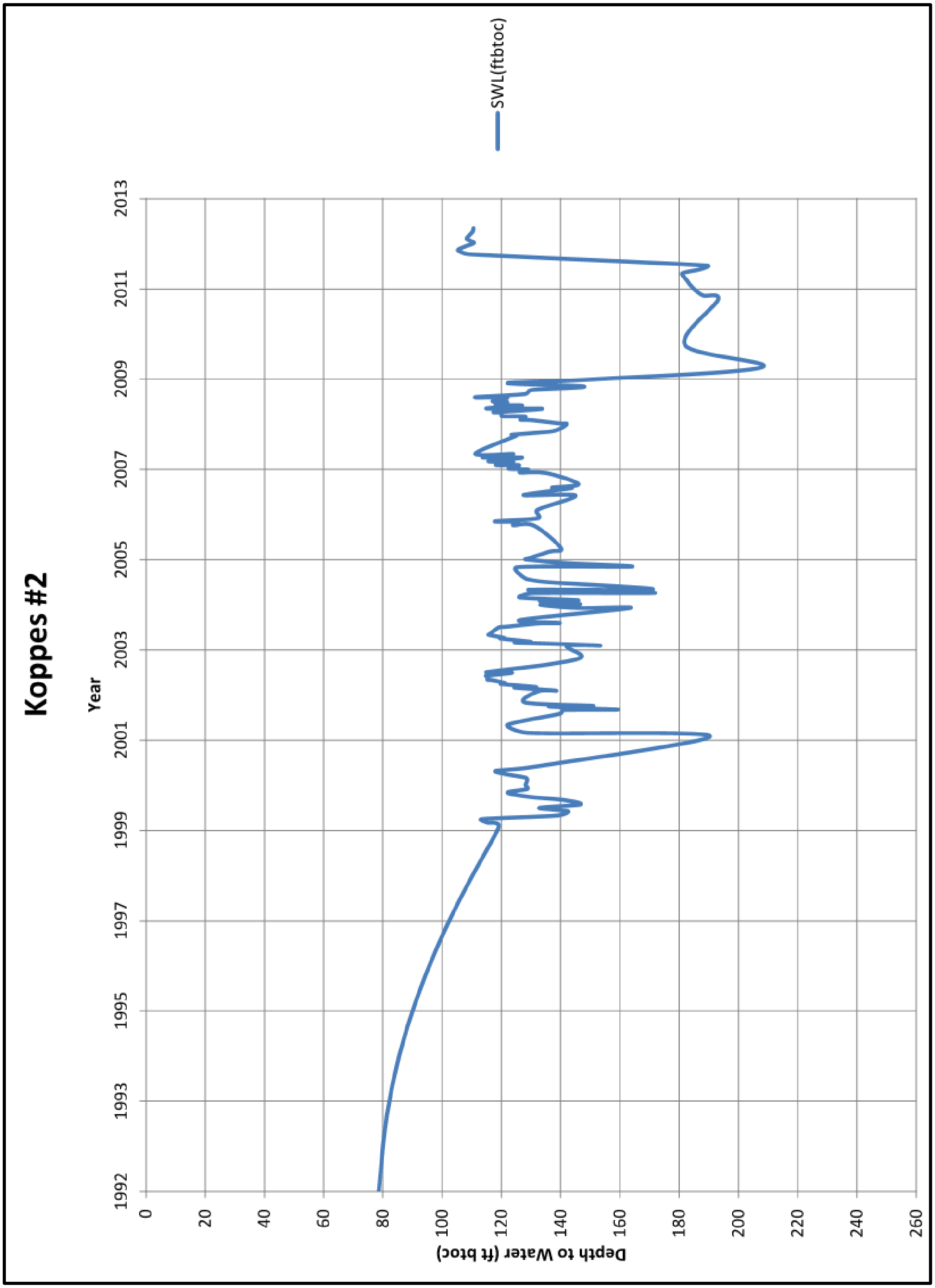
160
200
240
280
320


Koppes #2
Year
Depth to Water (ft btoc)
SWL(ftbtoc)
1992
1993
1995
1997
1999
2001
2003
2005
2007
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2011
2013
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40
60
80
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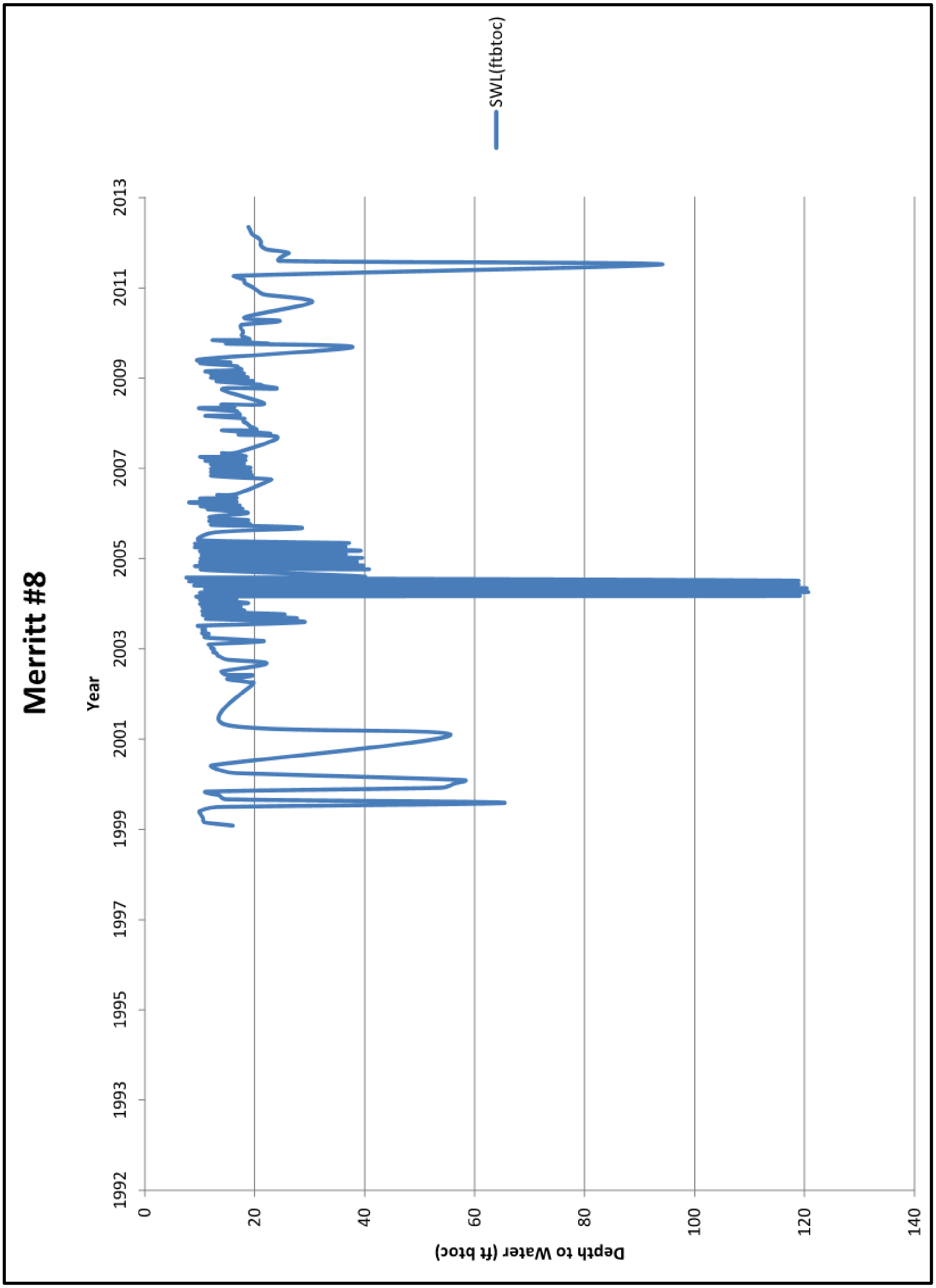
180
200
220
240
260


Merritt #8
Year
Depth to Water (ft btoc)
SWL(ftbtoc)
1992
1993
1995
1997
1999
2001
2003
2005
2007
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2011
2013
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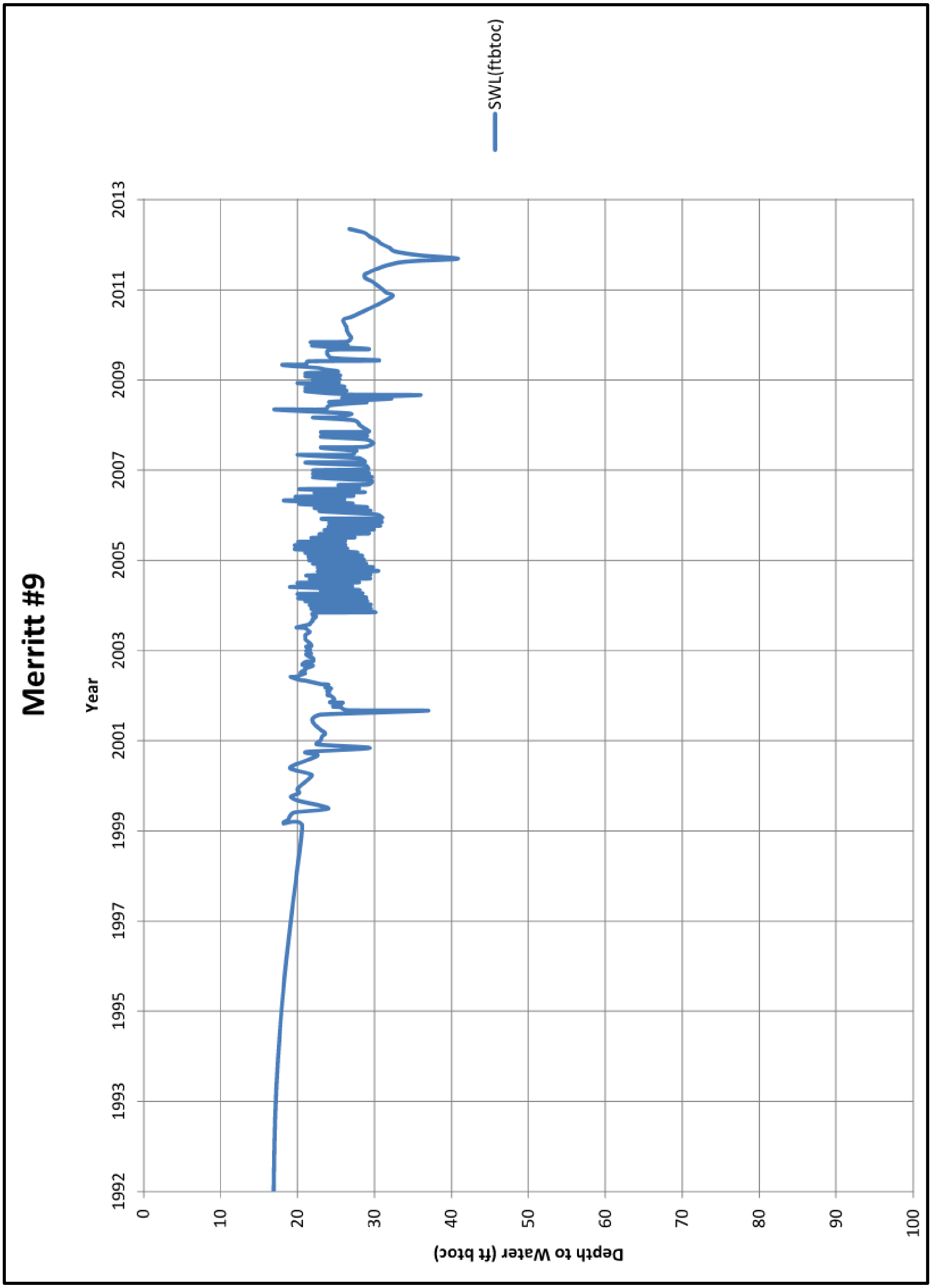
60
80
100
120
140


Merritt #9
Year
Depth to Water (ft btoc)
SWL(ftbtoc)
1992
1993
1995
1997
1999
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2003
2005
2007
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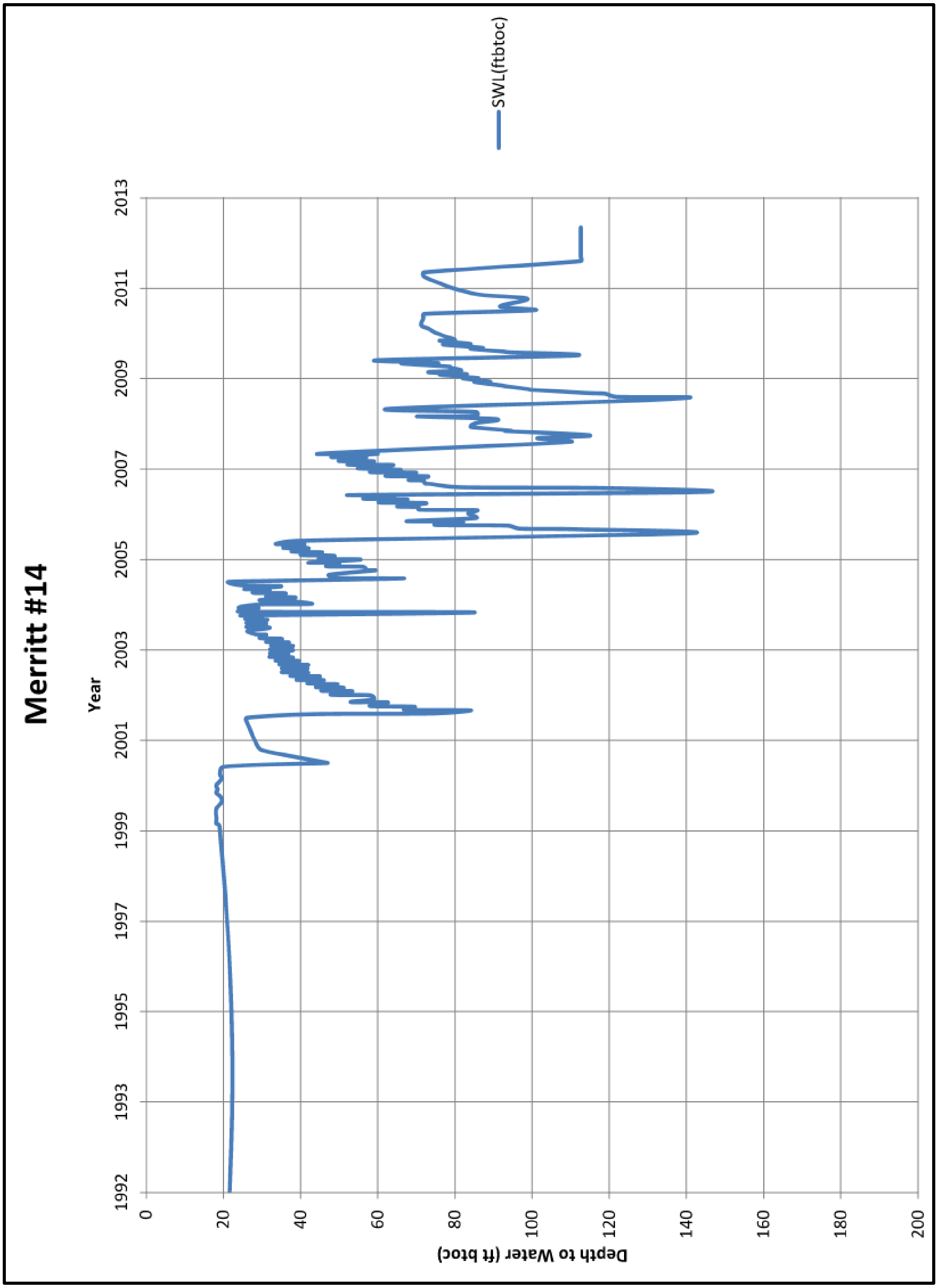
70
80
90
100


Merritt #14
SWL(ftbtoc)
Year
1992
1993
1995
1997
1999
2001
2003
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2007
2009
2011
2013
Depth to Water (ft btoc)
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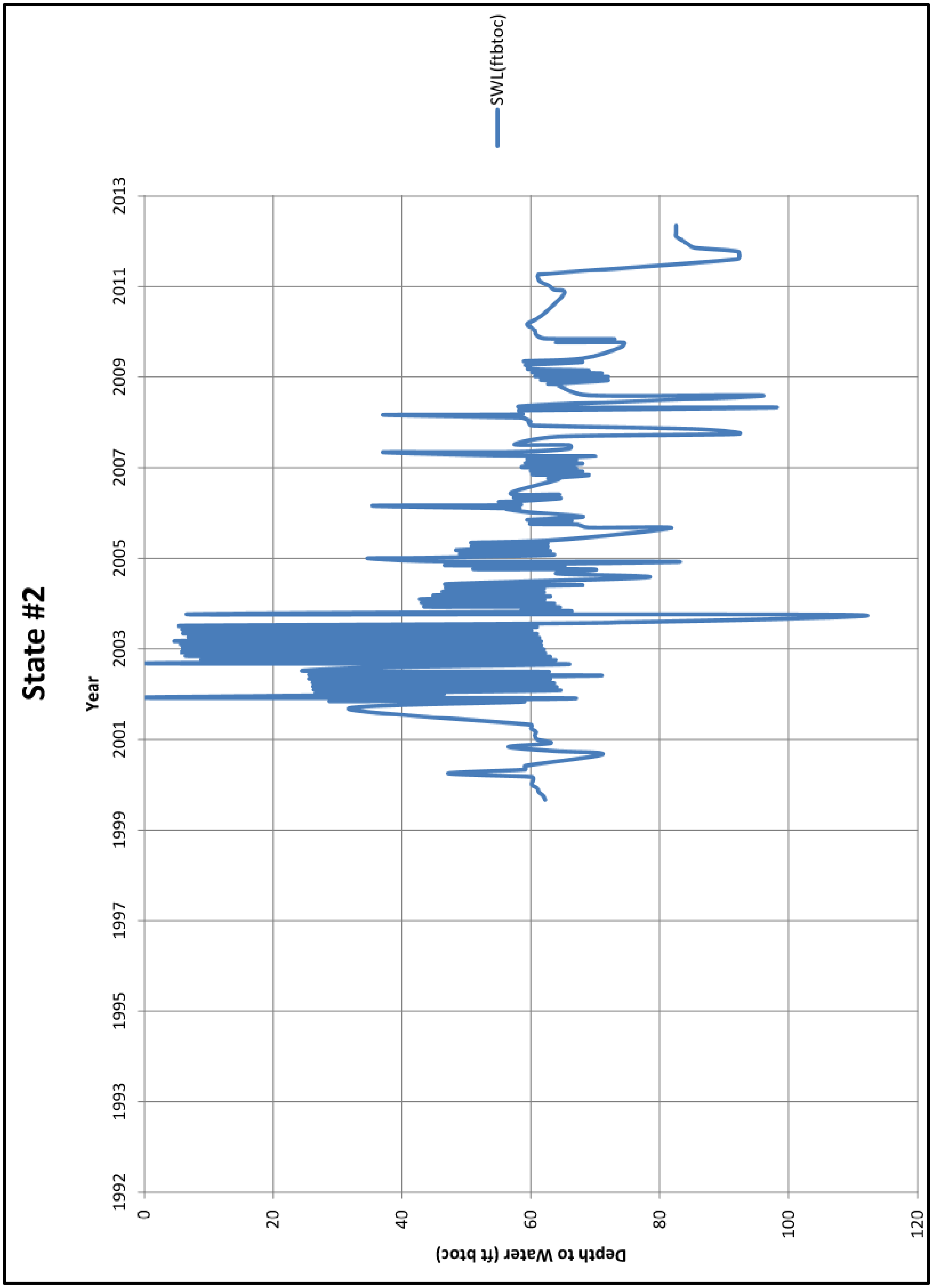
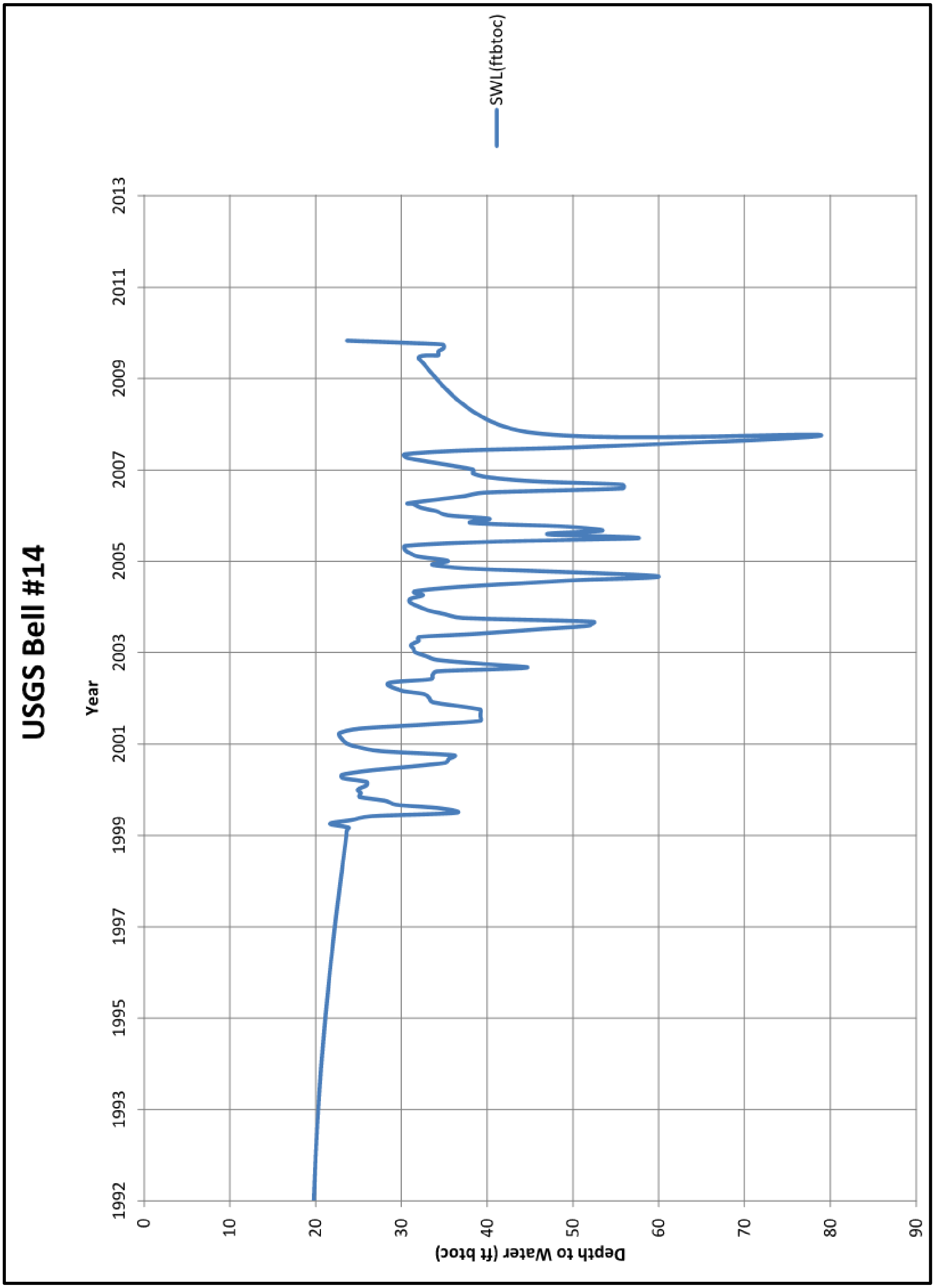
40
60
80
100
120
140
160
180
200


State #2
SWL(ftbtoc)
Year
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
Depth to Water (ft btoc)
0
20
40
60
80
100
120


USGS Bell #14
Year
Depth to Water (ft btoc)
SWL(ftbtoc)
1992
1993
1995
1997
1999
2001
2003
2005
2007
2009
2011
2013
0
10
20
30
40
50
60
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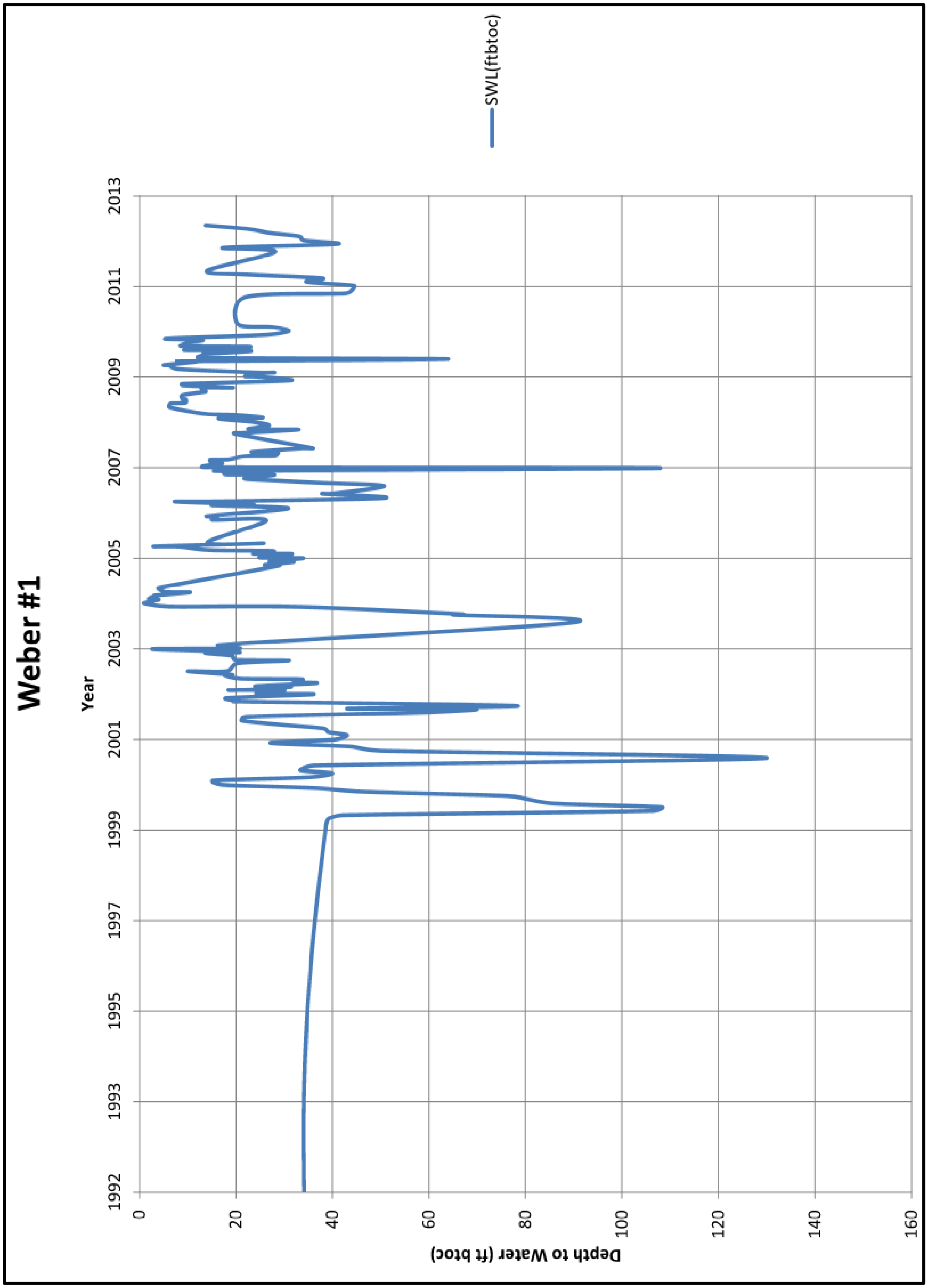
80
90


Weber #1
SWL(ftbtoc)
Year
Depth to Water (ft btoc)
1992
1993
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160

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Drought Monitoring Plan

Reprinted from

2002 Water/Wastewater Master Plans, rev. April 2004

3B.1 Guidelines for Plan Development

A Drought Monitoring Plan is necessary to assess the need to implement corrective actions in the event water demands begin to outpace available supplies. It is also necessary for BOPU to maintain a comfortable reservoir supply to ensure sufficient water supply in the event of continuing drought conditions. This Drought Management Plan is based on comparing anticipated demand with anticipated supply in the coming year and maintaining adequate future year reserves.

The Drought Monitoring Plan must be based on practical yet reliable methods. The plan must be practical so that BOPU personnel, with adequate training, are able to implement the various elements of the plan. The plan must also be compatible with the existing drought monitoring capabilities of BOPU while providing recommendations for additional monitoring. Regardless of any current drought monitoring limitations, the methods used in this plan must be reliable and commonly accepted as evidenced through use in drought plans for other, similar communities.

3B.2 Drought Definition

Drought is defined for Drought Monitoring Plan as single, or multiple consecutive, water years with below average streamflow. For the purpose of drought response planning, the droughts of interest would only include those droughts that, because of severity, directly impact and stress the raw water availability for BOPU.

Drought is not a sudden, discrete event like a flood but rather is a cumulative effect. Droughts progress in stages. Within a given drought contingency plan, drought can be classified based on stages where distinct mitigation measures would be taken according to the level of severity. For example, a Mild Drought classification might warrant voluntary conservation measures whereas an Extreme Drought classification might prompt restrictions on certain water uses and mandatory rationing. These drought classifications are usually triggered by one or more index-based on available climatological and water resources data. Descriptions of the various indices selected for BOPU are presented in following sections of this report.

For the purposes of this report, an advancing drought will be defined by the following drought classifications: (1) No Drought; (2) Mild Drought, (3) Moderate Drought; (4) Severe Drought, and (5) Extreme Drought. Each of these stages will be triggered by selected drought indices as will be discussed in later sections of this report. Each stage will also prompt certain mitigation action detailed in the Drought Management and Mitigation Plan (Appendix 3A). There are also receding stages of a drought that, in effect, mirror the advancing stages of the drought, which emphasize the need to update drought classifications regularly to keep pace with changing conditions and projections regularly.

3B.3 Review of Existing Drought Plans

Many Front Range communities have developed drought contingency plans to assist water management. The basics of these plans are summarized in Table 3B-1. Additionally, the State of Wyoming established a drought plan in October 2002. The Wyoming Drought Plan includes an overview of available data sources, recommended drought indices, and drought-triggering mechanisms. The Wyoming Drought Plan also reviews drought assessment and response. The assessment and response is the responsibility of the Wyoming Drought Task Force, which is divided into the sub-groups listed below:

- Agriculture
- Drinking Water, Health, and Energy
- Wildfire Protection
- Tourism and Economic Impact
- Communication

The actions of assessment and response vary based on the sub-group and are not specifically outlined in the Wyoming Drought Plan.

Table 3B-1

Summary of Other Front Range Drought Plans

City	Drought Trigger	Drought Response	Restrictions	Water Conservation Target
Boulder	Determined by City staff between April and May based on projected supply and reservoir storage; updated as needed	None	None	None
		Moderate	Voluntary	10 percent
		Serious	Mandatory - Moderate	20 percent
		Severe	Mandatory - Severe	30 percent
		Extreme	Mandatory - Ban	50 percent
Longmont	Reservoir Storage below Target and Combination of Storage and Projected Supply Exceed Demand by 115 percent	None	None	None
	115 to 100 percent	Level I	Voluntary	0 to 15 percent
	100 to 80 percent	Level II	Mandatory	15 to 35 percent
	<80 percent	Level III	Mandatory	As needed
Thornton	Case by case analysis of reservoir storage, projected streamflows, and availability of alternative supplies	None	None	None
		Drought Watch	Voluntary	10 percent
		Drought Warning	Mandatory	30 percent
		Drought Emergency	Mandatory	45 percent
Denver	Maximum Reservoir Storage, >80 percent	None	None	None
	< 80 percent	Mild Drought	Voluntary	10 percent
	<60 percent	Moderate Drought	Mandatory	30 percent
	<40 percent	Severe Drought	Mandatory	50 percent

Review of available documents indicates that most of the available drought contingency plans consist of two major components: drought monitoring and drought mitigation. Typically, one or more indices are used to trigger the formal announcement of a drought. Commonly used indices include reservoir storage as well as projected surface and groundwater supplies.

Most drought contingency plans use three stages to define drought status:

(1) Drought Watch or Drought Alert; (2) Drought Warning; and (3) Drought Emergency. Some plans have introduced an additional preliminary stage named Drought Advisory to represent the condition which approaches or experiences incipient drought.

3B.4 Surface Water Monitoring Plan

BOPU storage typically peaks in May or June. This peak level represents a very reliable drought indicator. After peak storage levels have been attained, the total amount of water available to BOPU for the next year is known. If this supply is insufficient to meet projected demands, then very serious drought situations can be virtually assured to arise. For this reason, Total Reservoir Storage represents the primary drought trigger to be used for evaluating BOPU's water supply conditions and triggering appropriate drought response actions. In this case, Total Reservoir Storage refers to the amount of water in BOPU system reservoirs which can be delivered to meet demands (Rob Roy, Granite, Crystal, and North Crow Reservoirs).

Prior to the actual occurrence of the peak level, streamflow estimates can be made with high levels of accuracy based on snow pack and other available indicators. Additionally, SnoTel stations exist in the Douglas Creek and Little Snake basins which can provide continuous records of existing snow pack through the spring season.

Based upon streamflow forecast, the projected yields of each watershed can be used for calculation of raw water availability. The most appropriate time for evaluating drought response triggers and planning drought responses is late April to early May. During this timeframe, final spring snow pack measurements are available and provide a relatively high degree of confidence regarding the amount of runoff that can be expected and the amount of water available to BOPU. Earlier estimates can be considered but could be very unreliable.

It should be noted that any drought response triggers should be used only as a guideline. BOPU should carefully evaluate these triggers and other factors unique to the particular drought, to determine the drought level response.

There are a number of tools available for BOPU to use in making this evaluation.

- SWSS Model. The SWSS model contains a "Current Year" scenario tool that is available to allow BOPU staff to evaluate system performance over the course of the upcoming 12-month period. The scenario tool is designed for operation in January through April and uses current reservoir storage, snow pack, estimated groundwater availability, and other prediction variables to model water availability and usage through a 1- to 3-year period.
- Water Budget Analysis. BOPU staff estimates of projected demand, groundwater production, and streamflow runoff can be combined with current reservoir storage values to derive a water budget approach to predict the occurrence of drought conditions.

3B.5 Groundwater Monitoring Plan

Another important component of BOPU's Drought Monitoring Plan is an evaluation of the availability of groundwater. Groundwater is an important element of BOPU's supply system, typically providing 25 percent of the total demand and providing important water quality benefits to the system.

3B.5.1 Objective

The primary objective of the groundwater monitoring plan is to provide forward-looking groundwater production estimates. In conjunction with surface water estimates, projected water supply and demand deficits/surpluses can be anticipated and appropriate drought mitigation measures implemented. Although groundwater provides approximately 25 percent of the total annual water supply for BOPU and is a valuable resource to meet peak demands during the summer months, the condition of surface water supplies will play a larger relative role in determining if and what drought mitigation measures are needed.

A secondary objective of the groundwater monitoring plan is to develop a systematic method(s) for the assessment of well field and aquifer conditions. This assessment process, through time, will provide BOPU groundwater management staff with a better understanding of how various factors affect well field production that, in turn, will allow more dependable predictions of groundwater production.

3B.5.2 Assessment Methods

An assessment of the groundwater supply and projected production estimates involve the periodic review of four types of data: (1) hydrologic data, (2) well production rates, (3) well field infrastructure condition, and (4) the accuracy of past production estimates. A brief overview of each data type is provided below. The review of these data requires insight from BOPU staff with specific knowledge of each production well and the operation of the well fields.

3B.5.2.1 Hydrologic Data

Hydrologic data review focuses on determining the present and/or projected condition of the Tertiary Aquifer from which the well fields derive water. A barometer of aquifer conditions are water level trends in production wells and dedicated monitoring wells. Historic water level trends in production wells will be developed via the manual and/or SCADA collection, storage, and manipulation of static (nonpumping) and pumping water levels. The SCADA system at each production well and the archiving of past data will be critical to the efficient review of water level data. Water level trends at three observation wells located in the vicinity of the well fields (monitored by the USGS) should be included in the data review.

Aquifer recharge plays an important role in determining the present and future condition of the aquifer; however, recharge mechanisms, timing, and amount are not well understood and are difficult to quantify. Maintaining databases of monthly, annual, moving annual average and the cumulative deficit/surplus of below/above average annual precipitation at the Hecla Station will provide additional insight into future aquifer conditions.

3B.5.2.2 Well Production Rates

An increase/decrease in production (i.e., sustained gpm) from each well over time provides a practical indication of well field production into the future. A snapshot of production rates for each well should be documented at the end of the high demand summer season (October) and again after the wells have recovered over the winter (May). Continuous well production data can be collected and stored automatically by the SCADA system. The production data must be analyzed in consideration of factors such as interference from nearby wells, well operation history, and the condition of pumping equipment.

3B.5.2.3 Well Field Infrastructure Condition

Projections of well field production will depend on the specific wells (i.e., high/low yield) and number of wells anticipated to be operational through the year, and any construction projects that could disrupt pipeline and/or well operation.

3B.5.2.4 Past Production Estimates

A review of the accuracy of past well field production estimates will provide insight into what parameters (as identified above) are relatively important/unimportant in the prediction of well field production. The over/under estimation of past well field production estimates, in conjunction with an update of aquifer conditions, will provide more accurate forward-looking well field production estimates.

3B.5.3 Groundwater Production Assessment Procedure

The Drought Monitoring Plan involves a schedule of water supply assessments during the calendar year. This section describes a recommended annual sequence of groundwater supply assessments.

3B.5.3.1 Initial Estimates of Groundwater Production (December)

BOPU wants to know in December the anticipated production from the groundwater supply for the coming year. BOPU groundwater staff will employ the general methods described in Section 3B.5.2.2 to generate an estimate for the total annual production from the well fields. For example, based on recent well production rates, total groundwater production for the year ending, and the present condition of the well fields, BOPU staff estimated a sustainable production of 3,870 ac-ft for the year 2003. As shown on Figure 3B-1, this total amount was distributed to each month in accordance with seasonal and system requirements.

In conjunction with projected monthly production estimates from the surface water supply and estimates of water demand, the relative magnitude of possible water supply surplus/deficit can be determined. This initial determination of drought conditions allows BOPU time to notify the public and to prepare for the implementation of various mitigation measures.

3B.5.3.2 Spring Assessment of Groundwater Supply (May)

In early May, production estimates from groundwater for the remaining months of the year (May through December) will be updated, and drought levels adjusted accordingly. Monthly groundwater production estimates will be updated as needed throughout the summer and fall.

Precipitation since the last assessment in December (i.e., January through April) may suggest potential increases/decreases in aquifer recharge. However, the lag time between precipitation events and aquifer response is not known precisely and favorable recharge events may not be apparent or predictable in the coming summer months. This is not the case, however, for surface water supplies that respond quickly to precipitation.

3B.5.3.3 Groundwater System Summary (October/November)

It is important that sometime during the year, BOPU groundwater staff reflect upon the objectives, methods, and value of the groundwater assessment process as specified in the drought monitoring plan. A brief document should be prepared that summarizes the ability/inability to adequately predict groundwater production on a month-by-month basis. What assessment parameters are important/ unimportant? What additional data are needed? How accurate were the predictions? How to improve assessment methods and procedure? Should more/less groundwater have been produced during the year? What is the current condition of the groundwater supply with respect to well production rates, water levels, etc.?

3B.5.3.4 Initial Estimates of Groundwater Production (December)

To start the next year's cycle of drought monitoring, BOPU staff would use the groundwater system summary document and the assessment methods described previously to provide an initial estimate of the next year's total and month-by-month groundwater production.

3B.5.4 Well Field Assessment Tool

The following is an approach to quickly assess the general condition of the well fields and may be a tool in the groundwater production assessment procedure or for general real-time well field monitoring. Ten high-yield wells were chosen and distributed throughout each well field (i.e., Borie – 2, Happy Jack – 4, Bell – 3, Federal - 1) in proportion to the relative contribution of each well field to total groundwater production. Production from the wells listed in Table 3B-2 typically represents 60 percent to 65 percent of the total annual groundwater production. Various ranges of feet of water above the pump intake (i.e., percentage of ADD) are used to determine the general condition of pumping water levels in each well. LOCs (e.g., low, moderate, high, and extreme) are based on pumping water levels and give well field operators a quick indication of conditions at critical wells. Table 3B-2 can be expanded with more wells and indicators as deemed appropriate.

Table 3B-2

Determination of LOC for Well Field Drought Assessment

Well	Pump Set	2002 Static DTW	ADD	DTW at 40 Feet above Pump	LOC: Low 30% of ADD DTW	LOC: Moderate 50% of ADD DTW	LOC: High 70% of ADD DTW	LOC: Extreme >70% of ADD DTW	2002 Pumping DTW	2002 LOC
	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	(ft)	
Koppes #2	210	129	81	170	153	170	186	>186	196	Extreme
Eddy #2	260	75	185	220	131	168	205	>205	190	High
Elkar #5	300	160	140	260	202	230	258	>258	178	Low
Koppes #1	220	98	122	180	135	159	183	>183	173	High
Bell #11	120	21	99	80	51	71	90	>90	49	Low
Bell #6	190	?	?	150	?	?	?			?
Bell #25	240	105	135	200	146	173	200	>200	183	High
Elkar #7	200	111	89	160	138	156	173	>173	144	Moderate
Borie #1	220	140	80	180	164	180	196	>196	?	?
Merritt #8	153	15	138	113	56	84	112	>112	77	Moderate

Appendix 3-C
Granite Springs Reservoir Site Visit

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A site visit was conducted by HDR on May 6th, 2013. The HDR personnel conducting the inspection were Elena Sossenkina, P.E. and David Isley. Bill Ray, with the City of Cheyenne Board of Public Utilities (BOPU), accompanied HDR on the site visit. The weather was mostly sunny and about 60° F. Observations of the project features and photos taken during the site visit are provided below.

Spillway Release Guide Dike

Some erosion has occurred on the west slope of the guide dike (see Photo 3). There is no erosion protection or sod cover on the west slope. Remedial action should be considered to protect the slope of the guide dike should the fuse plug breach. There is little to no information regarding the foundation conditions of the guide dike.

Fuse Plugs

The two fuse plugs looked to be in good condition. The lines and grades appeared to be accurate but were not confirmed with a survey. Riprap on the upstream slope was in good condition. The concrete slabs downstream of the fuse plugs did not have any apparent spalling or deterioration. There was minor seepage coming from underneath the fuse plug material and through some of the concrete joints (see Photos 5 through 8). There is no erosion protection downstream of the fuse plugs and the foundation conditions are unknown (see Photo 8).

Spillway

The reservoir level was approximately 1 foot below the crest of the main spillway. The spillway looked to be in good condition. No cracking or major spalling was evident. There was minor seepage coming from the right spillway wall (see Photo 10). The grouted riprap stilling basin appeared to be in good condition (see Photo 11).

There is some concern with the design of the spillway system. According to the drawings, the wall to the right of the spillway is at elevation 7213.5 (see Photo 13) and the elevations of the top of the fuse plugs are 7218.5 and 7219.5. This implies that water will flow as much as 5 feet over the top of the wall to the right of the spillway before the first fuse plug is breached. There is no identifiable erosion protection downstream of the wall and the foundation conditions of the spillway and surrounding areas are unknown. This area should be further evaluated by a qualified dam safety engineer.

Dam Abutments

Overall the abutments appear to be in good condition. There was some minor seepage near the top of the right abutment (see Photos 22 and 23). There is some shrub and tree growth along the abutments. It is good dam safety practice not to allow any woody vegetation within 50 feet of the dam toe or abutments.

Dam Crest and Upstream Face

The water level was a little over two feet below the crest of the dam so most of the upstream face was not visible (see Photo 26). Most of the upstream face of the dam is not visible without a boat because of the parapet wall. The crest and parapet wall (downstream of the parapet wall) were in good condition with no visible problems.

Downstream Face of Dam

The downstream face of the dam was in good condition with no visible seepage areas or other problems.

Downstream Toe of Dam

The downstream toe area was in good condition with no visible seepage areas or other problems.

Monitoring

The only instrumentation which exists at the dam site are 3 survey monuments on the crest of the dam. There was no available historic data from the survey monuments.

O&M Recommendations

- Remove woody vegetation downstream of the abutments from within 50 feet of the abutment contacts.
- Consider adding erosion protection to the spillway release guide dike downstream slope.

Other Recommendations

- The design of the spillway system should be evaluated by a qualified dam safety engineer.

Photo 1: From left abutment of spillway fuse plugs looking west. (May 6, 2013)

Photo 2: From left-most fuse plug, looking northeast at fuse plug left abutment (spillway release guide dike). (May 6, 2013)

Photo 3: From left-most fuse plug, looking southeast at spillway release guide dike. Notice the house on the other side of the dike. (May 6, 2013)

Photo 4: From right-most fuse plug, looking northeast across fuse plugs. (May 6, 2013)

Photo 5: Dowstream side of west fuse plug. Seepage coming from underneath fuse plug material. (May 6, 2013)

Photo 6: Downstream side of west fuse plug. Seepage coming from underneath fuse plug material. (May 6, 2013)

Photo 7: Downstream side of east fuse plug. Minor seepage coming through concrete base construction joints.

Photo 8: Downstream side of east fuse plug. Minor seepage coming from underneath fuse plug material and through concrete base construction joints. (May 6, 2013)

Photo 9: Granite Springs main spillway looking downstream. (May 6, 2013)

Photo 10: Granite Springs main spillway looking upstream. Minor seepage along west wall. (May 6, 2013)

Photo 11: Downstream of main spillway. Notice spillway subdrain system outfall pipe, flowing approximately 20 gpm. Riprap is grouted in. (May 6, 2013)

Photo 12: Downstream of grouted riprap below spillway. (May 6, 2013)

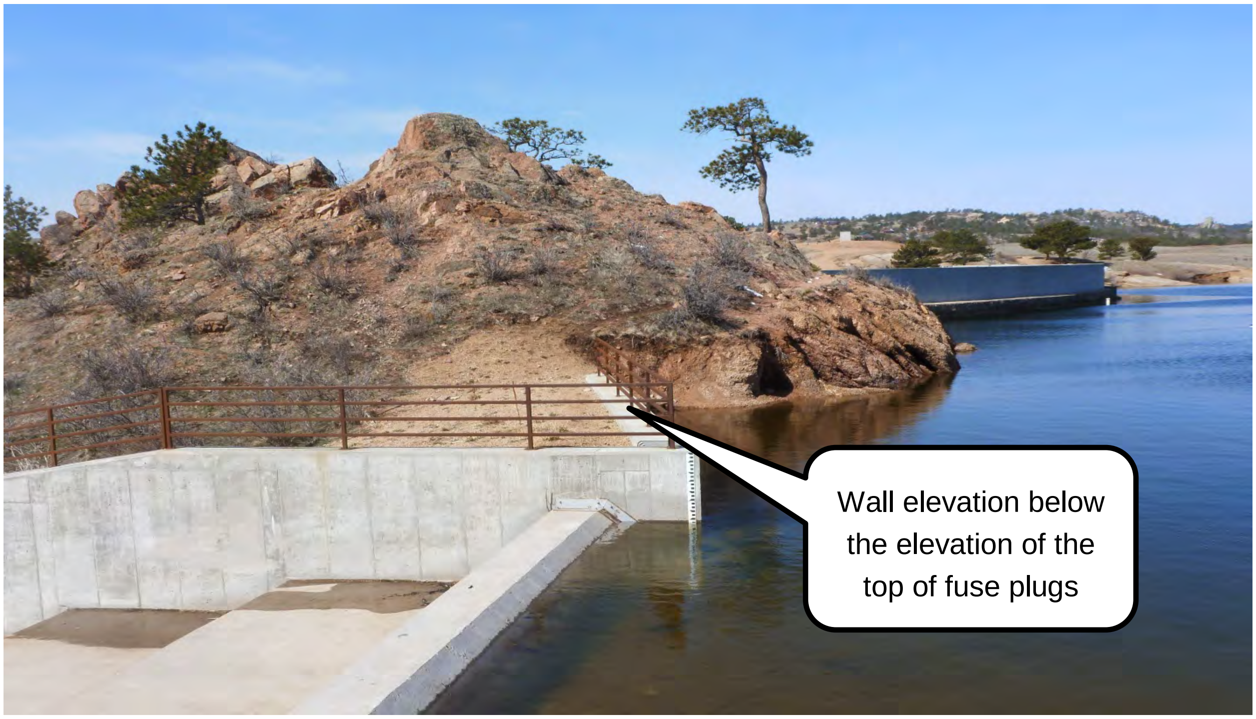

Photo 13: Right abutment of main spillway. Notice that the wall to the right of the spillway is below the elevation of the top of the fuse plugs. (May 6, 2013)

Photo 14: Looking upstream between the left abutment of the dam and right abutment of the main spillway. Notice the near vertical orientation of fractures in the rock.

Photo 15: From left abutment, looking west at upstream face of dam.

Photo 16: Dam crest, typical. Some deterioration. (May 6, 2013)

Photo 17: Left abutment of dam. No visible seepage. (May 6, 2013)

Photo 18: Left abutment of dam. (May 6, 2013)

Photo 19: Left abutment of dam. Some tree and shrub growth along abutment. (May 6, 2013)

Photo 20: Right abutment of dam. Some shrub growth along abutment. (May 6, 2013)

Photo 21: Right abutment of dam. Outlet works building. Some shrub growth along the abutment. (May 6, 2013)

Photo 22: Minor seepage along right abutment. Starts at wall interface. (May 6, 2013)

Photo 23: Minor seepage along right abutment. Starts at wall interface. (May 6, 2013)

Photo 24: Downstream toe of dam. Outlet works building. Some trees and shrubs. (May 6, 2013)

Photo 25: Survey monument marker on dam crest. Three markers total. (May 6, 2013)

Photo 26: Upstream face of dam, typical. Notice high water mark approximately 1 foot above current pool elevation. (May 6, 2013)

Photo 27: Downstream face of dam. (May 6, 2013)

Photo 28: Downstream face of dam. (May 6, 2013)

Photo 29: Upstream of right abutment of dam looking downstream to the south. (May 6, 2013)

Photo 30: Upstream of right abutment of dam looking downstream and to the southeast. (May 6, 2013)

Photo 31: Outlet works outfall pipes. (May 6, 2013)

Photo 32: Downstream of outlet works. Water falls directly onto exposed granite. (May 6, 2013)

Photo 33: Inside of outlet works building. (May 6, 2013)

Photo 34: Standing water on floor of outlet works building. (May 6, 2013)

Photo 35: Security gate. Typical both sides. (May 6, 2013)

Photo 36: Downstream weir. (May 6, 2013)

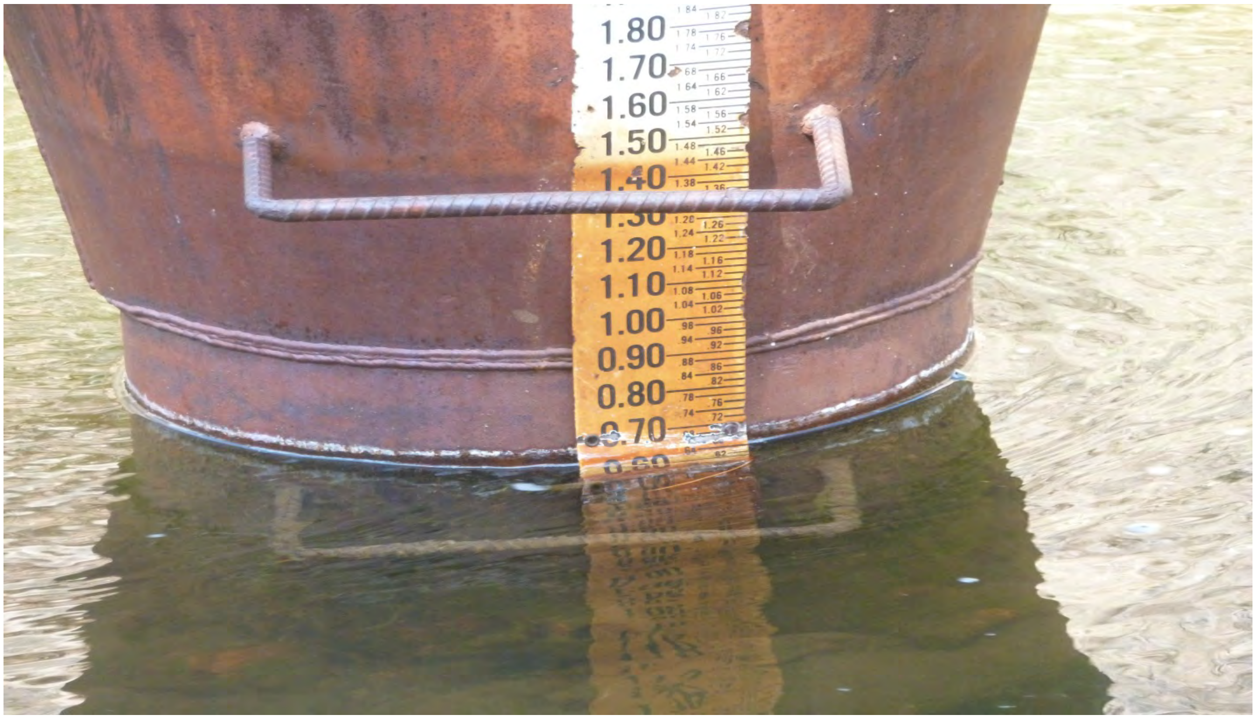

Photo 37: Weir measurement. (May 6, 2013)

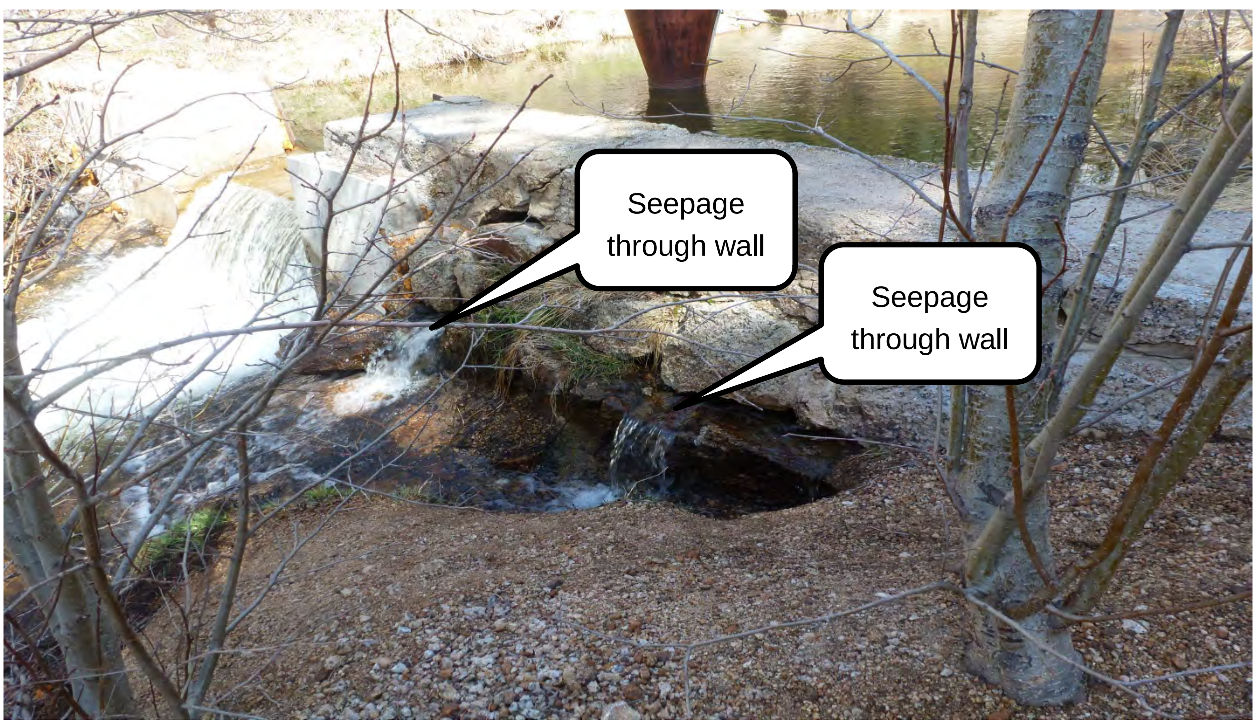

Photo 38: Downstream of weir. Notice water flowing through the wall. Weir measurements will not be accurate. (May 6, 2013)

Photo 39: Culverts running under a land bridge near where Middle Crow Creek meets Granite Springs Reservoir. (May 6, 2013)

Photo 40: Land bridge near where Middle Crow Creek meets Granite Springs Reservoir. (May 6, 2013)

Photo 41: Campground off the shore of Granite Springs Reservoir. (May 6, 2013)

Photo 42: Outfall of pipeline from Lake Owen to Granite Springs Reservoir. (May 6, 2013)

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Appendix 3-D
Crystal Lake Reservoir Site Visit

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A site visit was conducted by HDR on May 6th, 2013. The HDR personnel conducting the inspection were Elena Sossenkina, P.E. and David Isley. Bill Ray, with the City of Cheyenne Board of Public Utilities (BOPU), accompanied HDR on the site visit. The weather was mostly sunny and about 60° F. Below are some observations of the project features and photos taken during the site visit.

Saddle Dike

Saddle dikes exist on both sides of the spillway fuse plugs and each segment is approximately 90 feet in length. The southern and northern saddle dikes have trees growing on the upstream slopes. The presence of trees on the dike increases the uncertainty associated with the dikes integrity and performance. There is some sloughing occurring on the southern dike's upstream slope and the sloughing continues on the upstream slope of the existing ground surface (see Photos 1, 2 and 3). The downstream slope of the southern dike appears to be about 1H:1V (Horizontal: Vertical) or less. There was no seepage evident downstream of the saddle dikes. There is little to no information regarding the design of the saddle dikes or their foundation conditions.

Spillway and Fuse Plugs

The reservoir level was approximately 1 foot below the crest of the main spillway. The spillway slab looked to be in good condition. No cracking or major spalling was evident. The main spillway right sheetpile wall was bulging inward slightly (see Photo 8). There was little to no erosion downstream of the spillway slab. There was a large wet area downstream of the main spillway; however, it is possible that this was from recent snow melt (see Photo 11). There is reference to a 150 to 200-foot wide seepage area downstream of the saddle dike toe (before the existing spillway configuration) in the December 1983 Woodward-Clyde report.

The three fuse plugs looked to be in good condition. There was no evident seepage coming through or underneath the fuse plugs. The lines and grades appeared to be accurate but were not confirmed with a survey. Trees and shrubs were growing on the upstream side of the fuse plugs (see Photo 14). Vegetation on the upstream or downstream slopes may prevent the fuse plugs from functioning as designed and should be removed.

There is little to no information regarding the supporting engineering analyses or the foundation conditions of the spillway and fuse plugs.

Dam Abutments

Minor seepage was observed in both the right and left abutment contacts. The observed seepage was near the top of the dam. There was some snow pack near the middle of the right abutment contact. A void or an irregular rock surface was observed near the top of the right abutment contact (see Photo 23). The orientation of rock joints in the left abutment is adverse for dam stability. The joints are dipping downstream at approximate 45° as shown on Photos 16 and 17. Additional geological investigations are recommended to assess the integrity and stability of the abutments. A stress analysis was completed in April 1983 as part of the Engineering Evaluation by Woodward-Clyde. The stress analysis concluded that for any water level above the top of the dam it is imperative that compressive stresses be carried across radial cracks. A stress analysis should be completed taking the 1985 modifications into account and using updated software.

Dam Crest and Upstream Face

The water level was a little over two feet below the crest of the dam so most of the upstream face was not visible. There is also not a good way to inspect the upstream face of the dam without a boat. It is not visible from the crest of the dam due to the parapet wall. The crest and parapet wall (downstream of the parapet wall) were in good condition with no visible cracks or spalling.

Downstream Face of Dam

The downstream face of the dam was in good condition with no visible cracks or spalling. There are some visible calcium deposits developing below the concrete joints (see Photos 28 and 29), implying that there is some seepage through the dam. Internal seepage pipes were installed along some of the existing cracks during the 1985 Harza contract. The pipes discharge along the downstream face. During the site visit there was minor flow through the pipe which outfalls about mid way up the dam along the left abutment (see Photo 25). According to BOPU personnel, the pipe just above the outlet works building is also usually flowing but it was filled with ice during the site visit (see Photo 24). None of the other pipes were flowing during the site visit but there were some calcium deposits below the pipe outfalls indicating that there is some water exiting out of the pipes. According to BOPU personnel, the pipes are cleaned once a year through risers on the crest of the dam. It is recommended that BOPU continue with the yearly cleanings to prevent development of calcium deposits which could restrict flow through the pipes.

Downstream Toe

The downstream toe area was in good condition with no visible seepage areas or other problems.

Monitoring

The only instrumentation which exists at the dam site are 3 survey monuments on the crest of the dam. There was no available historic data from the survey monuments.

O&M Recommendations

- Remove woody vegetation from the upstream side of the fuse plugs and saddle dikes.

- Regrade the downstream slope and crest of the southern saddle dike with a 15-foot wide crest and a 2H;1V downstream slope.
- Regrade and place riprap on the upstream slope of the southern saddle dike and the adjacent existing ground surface where sloughing has occurred.
- Continue cleaning the internal seepage pipes in the dam on a yearly basis.

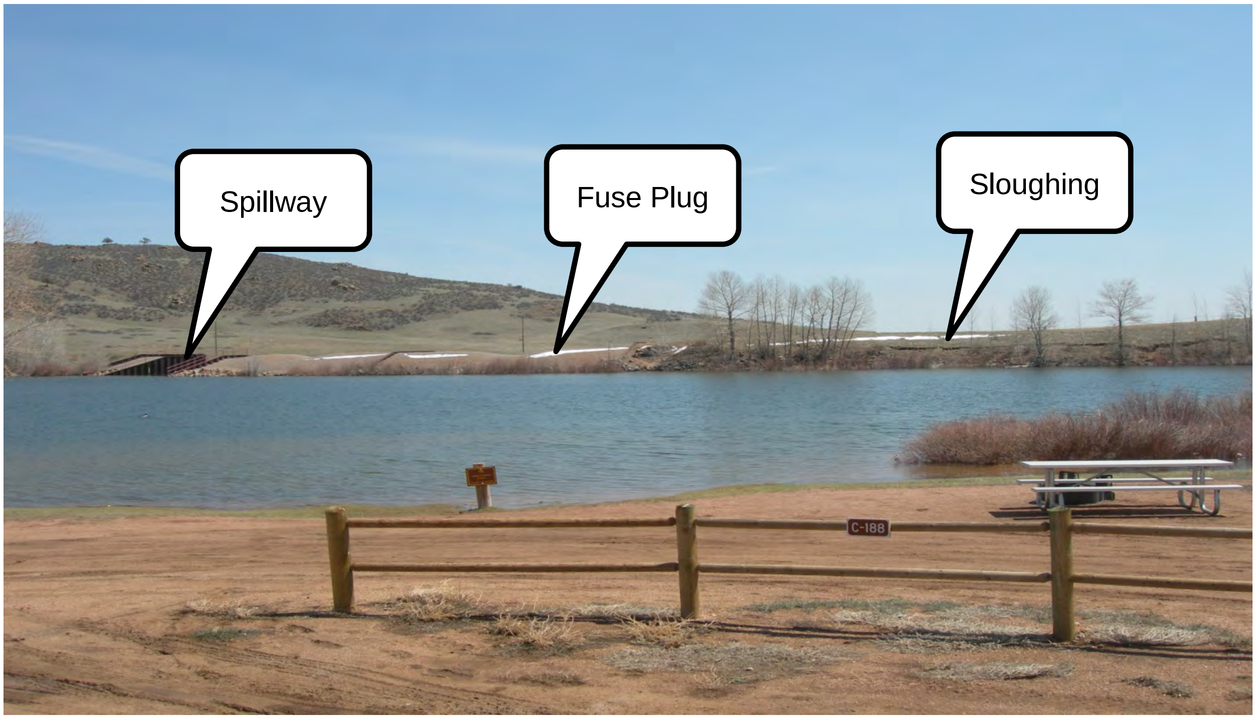

Photo 1: Crystal Lake spillway and fuse plugs. Notice sloughing of the existing ground to the right of the fuse plugs. See Photo 2. (May 6, 2013)

Photo 2: Sloughing of existing ground to the right of the fuse plugs. (May 6, 2013)

Photo 3: Sloughing of existing ground to the right (south) of the fuse plugs. See Photo 2 for reference. (May 6, 2013)

Photo 4: Downstream slope of right abutment of southern-most fuse plug. Slope approximately 1H:1V or steeper.

Photo 5: Downstream slope of right abutment of southern-most fuse plug. Slope approximately 1H:1V or steeper. (May 6, 2013)

Photo 6: Downstream side of the three fuse plugs and main spillway. (May 6, 2013)

Photo 7: Water level relative to main spillway. (May 6, 2013)

Photo 8: Right abutment wall of spillway is bulging slightly inward. (May 6, 2013)

Photo 9: Downstream of main spillway. (May 6, 2013)

Photo 10: Main spillway looking upstream. (May 6, 2013)

Photo 11: Large wet area downstream of spillway. Could potentially be runoff from recent snowfall. (May 6, 2013)

Photo 12: Embankment to the north of the northern-most fuse plug. Slightly above the elevation of the fuse plugs for 100-200 yards. (May 6, 2013)

Photo 13: Looking south from northern-most fuse plug. Elevation of existing ground beyond southern-most fuse plug is only slightly above elevation of fuse plugs for several hundred yards. (May 6, 2013)

Photo 14: Trees on upstream side of northern-most fuse plug. (May 6, 2013)

Photo 15: Crystal Lake Dam. (May 6, 2013)

Photo 16: Left abutment. (May 6, 2013)

Photo 17: Top of left abutment. Notice the orientation of the rock joints. (May 6, 2013)

Photo 18: Left abutment. (May 6, 2013)

Photo 19: Left abutment and downstream face. (May 6, 2013)

Photo 20: Right abutment. (May 6, 2013)

Photo 21: Right abutment. (May 6, 2013)

Photo 22: Right abutment and downstream face. (May 6, 2013)

Photo 23: Right abutment. (May 6, 2013)

Photo 24: Roof of outlet works building. Internal seepage pipe was rerouted (as shown) because it was regularly releasing water onto the roof. (May 6, 2013)

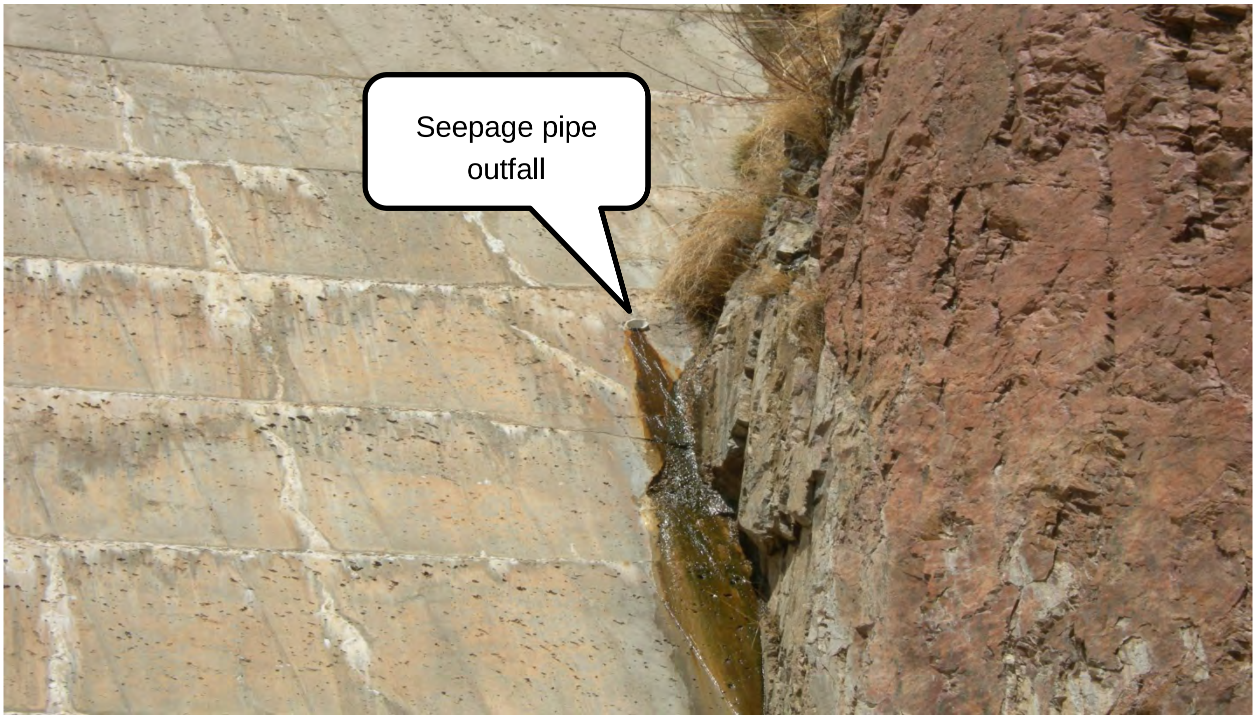

Photo 25: Internal seepage pipe in left abutment about mid way up the face of the dam. Regularly releasing water. (May 6, 2013)

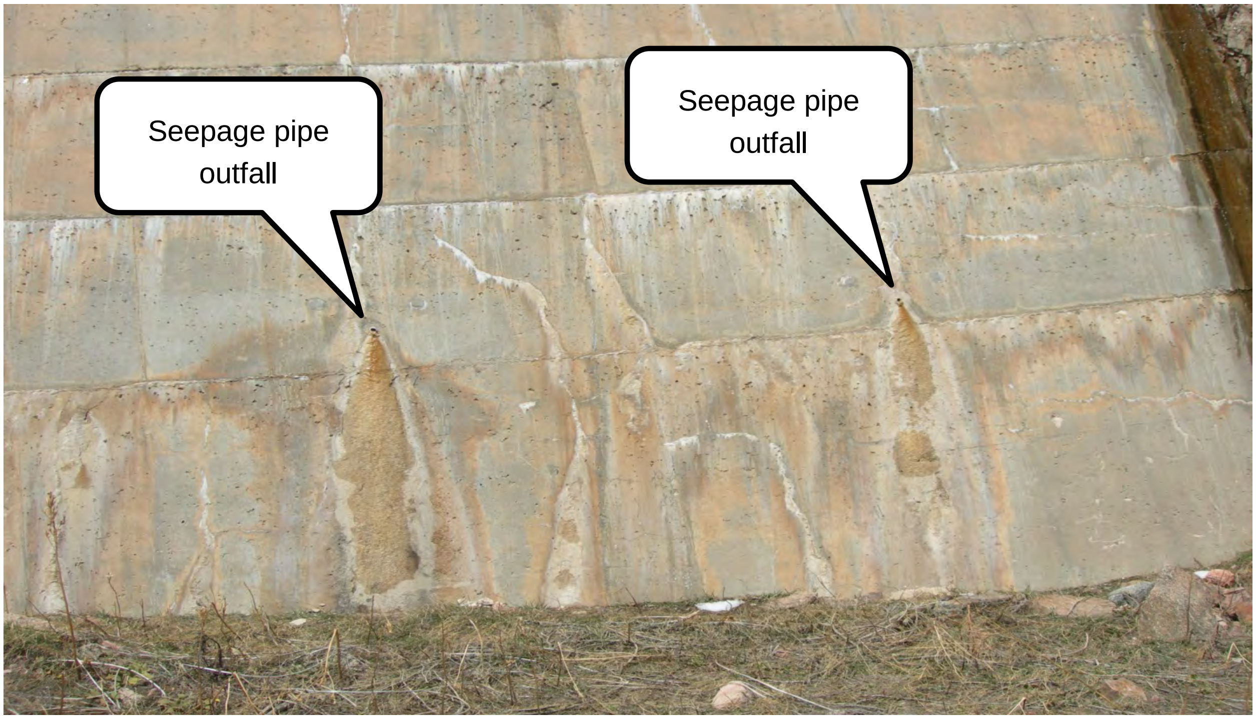

Photo 26: Internal seepage pipes near left abutment. Calcium deposits developing below outfalls. (May 6, 2013)

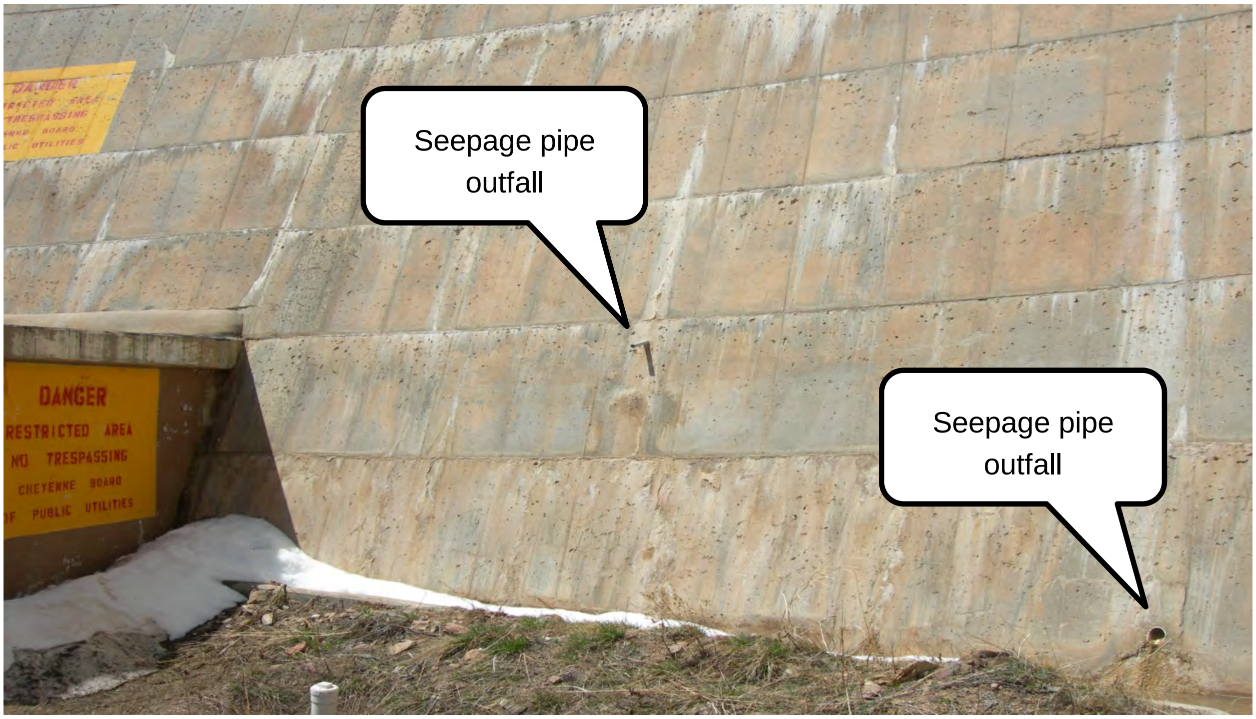

Photo 27: Internal seepage pipes to the left of outlet works building. Calcium deposits developing below the pipe in the bottom right. (May 6, 2013)

Photo 28: Downstream face of dam left of centerline. Some calcium deposits forming at joints. (May 6, 2013)

Photo 29: Downstream face of dam right of centerline. Some calcium deposits forming at joints. (May 6, 2013)

Photo 30: Outlet works buildings. 30” and 50” pipes carry water to a water treatment plant downstream.