CHAPTER 8

FURTHER ANALYSIS OF SELECT ALTERNATIVES

As a general note common to all alternatives in this section: on-site wetlands delineations were not conducted for the project areas in the following alternatives as this work is outside the scope of this report. Rather, Mike Burgan in the Wyoming office of the U.S. Army Corps of Engineers was contacted. He recommended consulting National Wetlands Inventory Maps for a preliminary overview of possible wetlands infringements. He stated that in Wyoming, these NWI maps are generally accurate and reliable. Digital NWI maps were obtained from the U.S. Fish and Wildlife Service website for the project area. These maps are included as Appendix E. From a preliminary review of these maps, it does not appear that the project will be impacting any wetlands. At the design stage of a project, on-site wetlands delineation would need to be conducted to confirm this conclusion. Accordingly, no mitigation costs were included in the economic analysis provided herein.

8.1 SHORT-TERM ALTERNATIVES

In Chapter 7, short-term source options were reviewed and three viable short-term sources were selected for further study:

1) Expand the Fort Union Source
2) Treat the Fox Hills/Lance Source
3) Use Coal Bed Methane Water

Each of these sources is developed into an alternative with discussion below. The time frame to fully implement a long-term water supply source is reasonably assumed to be 6-7 years until a long-term solution is in place. The integrity of Gillette's water system is dependent on providing a sufficient amount of water for this time period, so a time horizon of 8 years will be used to ensure water supply is adequate until the long-term water supply project is commissioned. While this is an approximation of project time, it is heavily dependent on a number of factors, including timing of funding. Substantial delays to any critical path task on the schedule will result in delays of the overall project and will require revision of this interim plan.

For the short-term alternatives, it is assumed that the populations of Crestview, Antelope Valley, and Sleepy Hollow will continue to be served off their existing systems until a long-term supply is implemented. With these populations removed from the demand, the total design population for the short-term alternatives in 2015 is 34,915 with a peak water demand of 21.4 MGD. Given the existing system has a maximum capacity of approximately 14.5 MGD, this leaves a deficit of 6.9 MGD, or 4,791 gpm with 24 hours per day of pumping time. The maximum existing system capacity of 14.5 MGD is not firm capacity. Using firm capacity methodology, the existing firm capacity is 12.1 MGD leaving a deficit of 9.3 MGD or 6,450 gpm pumping 24 hours per day. Firm capacity should be used in all planning efforts to ensure system redundancy. The incremental 2015 flow rate of 6,450 gpm will be planned for in the short-term portion of this study.

8.1.1 Expanding the Fort Union Source

The uncertainties of using the Fort Union as a long-term water source were discussed in Chapters 5 and 7. However, when the concept is analyzed with the assumptions of a shorter project life time horizon and lower withdrawal rate, adding wells in the Fort Union is a viable short-term strategy.

Aside from the long-term effects of pumping the Fort Union as a primary long-term source discussed in Chapter 5, the Fort Union is a relatively “known” aquifer. The City of Gillette has been producing from the Fort Union for a substantial period of time and currently uses the Fort Union well group as its “core,” or preferred, source. The City of Gillette currently has 13 wells in the Fort Union formation.

Water quality data are readily available from the in-town Fort Union wells Gillette currently operates. All of these wells operate above 1.0 mg/l of fluoride, with 7 of the wells operating above 2.0 mg/l, and one well over the US EPA Primary standard of 4.0 mg/l fluoride. Other quality concerns are iron levels (3 wells above 0.3 mg/l), total dissolved solids (5 wells over 500 mg/l) and sodium (3 wells over 250 mg/l). These problems can generally be mitigated simply by blending the well water with the elevated level with other well water from the other Fort Union or Madison wells with lower concentrations. The exception to this statement is fluoride – particularly using the 2.0 mg/l limit. The EPA currently has the MCL for fluoride set at 4.0 mg/l with the SMCL set at 2.0 mg/l. There is speculation that the EPA may consider lowering the

MCL to 2.0 mg/l at some time in the future. No pending or proposed legislation is known to be currently active. With this lower limit, Fort Union water will likely require treatment for fluoride or blending with a lower fluoride source. Unfortunately, none of the sources in the area have low fluoride levels. The Madison water has some capacity for blending, however the average fluoride concentration for the Madison water is still just over 1.0 mg/l, leaving a limited capacity to blend with high fluoride water.

As discussed in Chapter 7, the average flow rate per well for new Fort Union wells for the purpose of this study is 275 gpm. At this flow rate, this alternative will require 24 wells to provide firm capacity. This sizing and concept is similar to the Southern Well Field project proposed by Wester-Wetstein and Associates.

During this study, discussions have been held regarding expanding the existing in-town Fort Union well system to provide short-term capacity. It is assumed that this is not a viable approach given the quantity of water and the associated quantity of wells that is being proposed here. For planning and estimating purposes, this alternative is substantially similar to the Southern Well Field Concept.

This alternative involves constructing a well field south of town, collecting the well water at a common point, providing chlorination and air stripping for gas removal, and piping the finished water into town where it will be blended at some point in the system, presumably at or near reservoir Z1-R4. This concept is in essence the same layout proposed by Wester-Wetstein and Associates in the Southern Well Field Concept with a slightly different connection strategy to the existing system. A complete cost estimate for this alternative can be found in Appendix A.

The estimated cost for this short-term alternative is $88.2 million. This translates into a cost of approximately $13,675 per gpm.

8.1.2 Treating the Fox Hills Source

The Fox Hills source can be made into a viable short-term source by treating the water to EPA standards. This alternative will address treatment of the high levels of fluoride and total dissolved solids (TDS) found in the Fox Hills water; other water quality information for additional

contaminants can be found in the *City of Gillette – Water Master Plan Report (Wester-Wetstein & Associates, 2004)*.

TDS is on the Secondary Drinking Water contaminants List with an SMCL of 500 mg/L. EPA has set a Primary Drinking Water standard of 4.0 mg/L as well as a Secondary Drinking Water standard of 2.0 mg/L for fluoride to protect against dental fluorosis. The SMCL of 2.0 mg/L is intended as a guideline for an upper boundary level in areas which have high levels of *naturally occurring* fluoride. It is *not* intended as a substitute for the lower concentrations (0.7 to 1.2 mg/L) which have been recommended for systems which *add* fluoride to their water. Dental fluorosis, in its moderate or severe forms, may result in a brown staining and/or pitting of the permanent teeth. This problem occurs only in developing teeth, before they erupt from the gums. Children under nine should not drink water that has more than 2 mg/L of fluoride.

Table 8-1 shows water quality data for the Fox Hills Formation. The fluoride levels for the Fox Hills Well range from 6.0 mg/L to 9.4 mg/L; these levels are well above the Primary Contaminant level of 4.0 mg/l and the Secondary Contaminant level of 2.0 mg/L. The TDS levels for the Fox Hills Wells are more than double the Secondary Contaminant standard of 500 mg/L, with the majority of dissolved solids comprised of sodium bicarbonate. The water in the Fox Hills Formation is considered brackish water for treatment purposes. Water is considered brackish when the TDS levels range between that of fresh water (500 mg/L) and sea water (greater than 30,000 mg/L).

TABLE 8-1
Water Quality Data for Fox Hills Wells

Contaminant	FH-3	FH-4	FH-5
Flouride as F, ppm (1)	8.3	8.1	--
Flouride, mg/L (2)	9.4	7.0	6.0
TDS, mg/L (2)	1,280	1,240	1,030
pH (1)	8.3	8.3	--
pH (2)	8.75	8.36	8.58

(1) Samples collected August 1, 2006.

(2) Samples collected November 8, 2004 – Reported in the City *of Gillette – Water Master Plan Report (Wester, Westein & Associates, 2004)*. These results were obtained using a gravimetric TDS analysis which results in loss of HCO_3 due to heating of the solution. Actual TDS based on an ion balance is in the range of 1800 mg/L for all three wells.

8.1.2.1 Water Treatment Goals

The treatment alternatives should meet all state and national water quality standards. The following standards set by the Environmental Protection Agency (EPA) and the State of Wyoming Department of Environmental Quality (WYDEQ) should be considered.

Fluoride ≤ 2 mg/L – Secondary Maximum Contaminant Level set by the EPA and adopted by the WYDEQ.

Fluoride ≤ 4 mg/L – Primary Maximum Contaminant Level set by the EPA and adopted by the WYDEQ.

Total Dissolved Solids ≤ 500 mg/L – Secondary Maximum Contaminant Level set by the EPA and adopted by the WYDEQ.

8.1.2.2 Treatment Alternatives

The following treatment alternatives were evaluated to determine their effectiveness in reducing the fluoride and TDS levels in the drinking water.
As previously stated, these are the main contaminants of concern in using the Fox Hills Formation ground water as a potable water source. The treatment alternatives considered are:

- Reverse osmosis (RO)
- Electrodialysis (ED) and Electrodialysis Reversal (EDR)
- Ion Exchange (IX)
- Ion Exchange plus Reverse Osmosis (IX/RO)
- Electrodialysis and Electrodialysis Reversal plus Reverse Osmosis (ED-EDR/RO)

8.1.2.2.1 Treatment Alternative 1 - Reverse Osmosis

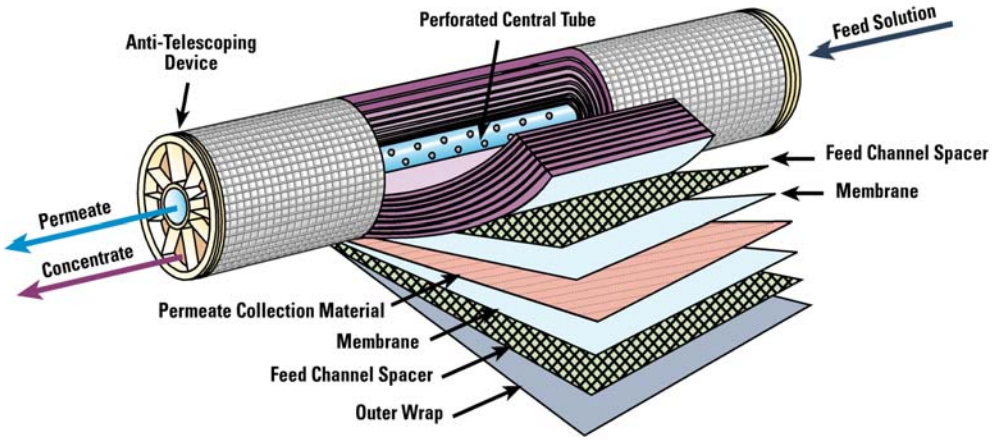
Reverse osmosis (RO) processes were evaluated for the treatment of Fox Hills Formation because they are excellent at removing TDS and fluoride. RO is a pressure-driven membrane process, which removes dissolved constituents based on their molecular weight. Higher driving pressures are required to achieve this separation and because the membranes cannot be

hydraulically backwashed, water must first be pretreated to remove particulate matter that would otherwise foul membranes.

RO systems generally utilize both pretreatment and post-treatment processes for operation and water quality. Typical pretreatment processes include adding acid, antiscalant, or both to the feed solution to prevent salt scaling during the membrane filtration step. A cartridge filter is typically placed ahead of the RO membranes to remove suspended solids. Although it is not always necessary in groundwater applications, microfiltration or conventional filtration is often used ahead of RO to remove high levels of suspended solids. The surface of the RO membranes is comprised of microscopic pores that range in size from 300 to 10,000 µm, which do not allow ions over a certain molecular weight to pass through the membrane. This process is known as the molecular weight cutoff (MWC). The pressure driven process retains all ions with a molecular weight larger than the MWC on one side of the semi-permeable membrane while allowing water and smaller ions to pass across the membrane. Following the membrane filtration, the water is post-treated using processes such as disinfection or degasification to stabilize the water. RO membranes are also excellent barriers to microbial contaminants, viruses, and numerous organics, and can also be used to remove endocrine disrupting compounds.

Figure 8-1 shows the general flow process of feed solution as it enters a RO membrane. The layers of the membrane are shown. As the feed solution moves through the membrane, ions that are larger than the pores are not able to penetrate the membrane. The permeate or the treated water moves through the membrane to the post-treatment step. The disposal of the concentrate water is regulated and is discussed further in Section 6.5.

Figure 8-1
RO Membrane (Courtesy of GE Osmonics)



RO System for Fox Hills Wells

The RO system is proposed to reduce total dissolved solids (TDS), hardness, and fluoride. The initial phase of the RO systems would be designed to produce a maximum capacity of 1.95 MGD (1354 gpm) using four (4) trains at a rated capacity of 0.487 MGD per train. This system will give the RO System a firm capacity of 1.46 MGD with one train out of service. The system is sized to treat the total capacity of the Fox Hills Wells, 1,800 gpm at 75% recovery. A layout for the proposed RO system is shown in Figure 8-2.

The number of stages required for the RO system is dependent on the desired recovery rate. For a recovery of 75%, a two-stage system is typically used. For higher recovery rates, such as 90%, a three-stage system is typically used. Figure 8-3 shows a two-stage and three-stage RO System. To achieve the higher recovery with the Fox Hills water, large quantities of antiscalant or sodium-based ion exchange is needed upstream of the RO system to reduce scaling. Four trial runs, using a RO design program (IMSDesign) developed by Hydranautics were conducted to determine a preliminary layout. The trial runs were developed on a pilot scale basis instead of the entire capacity of the three wells for simplicity. The results from the trial runs and information on the Hydranautics RO membranes are provided in Appendix C-1. Note that there are several manufacturers of high quality RO membranes in addition to Hydranautics that could be utilized with a similar design.

RO Membrane Booster Pumps Cartridge Filters and Feed Pumps:

The existing Pump Station No. 1 will be used as a booster to increase the feed water pressure that is delivered to the RO units. The RO feed water will be filtered through 5-micron cartridge filters to protect the RO membranes from suspended solids as mentioned previously. Note that many membrane element manufacturers will not provide their standard warranty in installations without cartridge filters immediately upstream of the membranes. These cartridge filters will also protect the RO membranes from a catastrophic event, such as failure of a pump impeller that may introduce debris into the feed line.

Water will be fed from the cartridge filters to a dedicated feed pump on each RO skid utilizing the pressure provided by Pump Station No. 1. The feed pumps provide high pressure, supplying the required force to drive the feed water through the semi-permeable RO membranes. Each pump will need to be VFD controlled and linked to the permeate flow meter on the skid to maintain a flow set point. As the membranes slowly foul over time and the pressure required to maintain the flow set point increases, the pump speed will increase. An automatic high pressure safety shutdown switch should be used to protect the system if pressure exceeds recommended levels and a rupture disk should be used to ensure the pressure vessels never exceed their rated pressure.

A sample RO membrane cleaning protocol is included in Appendix C-2. RO cleaning procedures are site and element specific. The cleaning protocol developed specifically for this site should list suggested chemicals that will be needed for the cleaning procedure.

Figure 8-2

Figure 8-3

Table 8-2 gives the advantages and disadvantages of RO Systems.

TABLE 8-2
Advantages and Disadvantages of RO Systems

Advantages	Disadvantages
Removes charged and uncharged molecules (fluoride and TDS)	Concentrate disposal
Minimum usage of chemicals for pretreatment	Scaling or Biofouling
Small footprint	High pressure requirements (150-400psi)
	High energy requirements

8.1.2.2.2 Treatment Alternative 2 – Electrodialysis (ED) and Electrodialysis Reversal (EDR)

Electrodialysis is an electrochemical separation process in which ions are transferred through cation (C) and anion (A) selective membranes. Ions move from a less concentrated to a more concentrated solution as the result of the flow of a direct electric current. The treated water flows tangentially to the membrane, while the ions are driven perpendicularly across the membrane. Figure 8-4 shows the flow of ions (red and orange arrows) and feed water (black arrows). The ED process always forces ion movement in the same direction, which can result in scaling on the membrane surface over time. EDR processes routinely and automatically reverse the polarity across the membrane (two to four times an hour), causing ions to travel in the opposite direction. The polarity reversal provides flushing of scale-forming materials from the membrane surface. Therefore, little or no pretreatment is required to minimize the fouling or scaling potential of the feedwater for the EDR process. Figure 8-5 represents the EDR system before and after the polarity has been reversed. Both the ED and EDR processes are discussed because of their ability to remove TDS, hardness and other ionized contaminants.

Figure 8-4
ED System

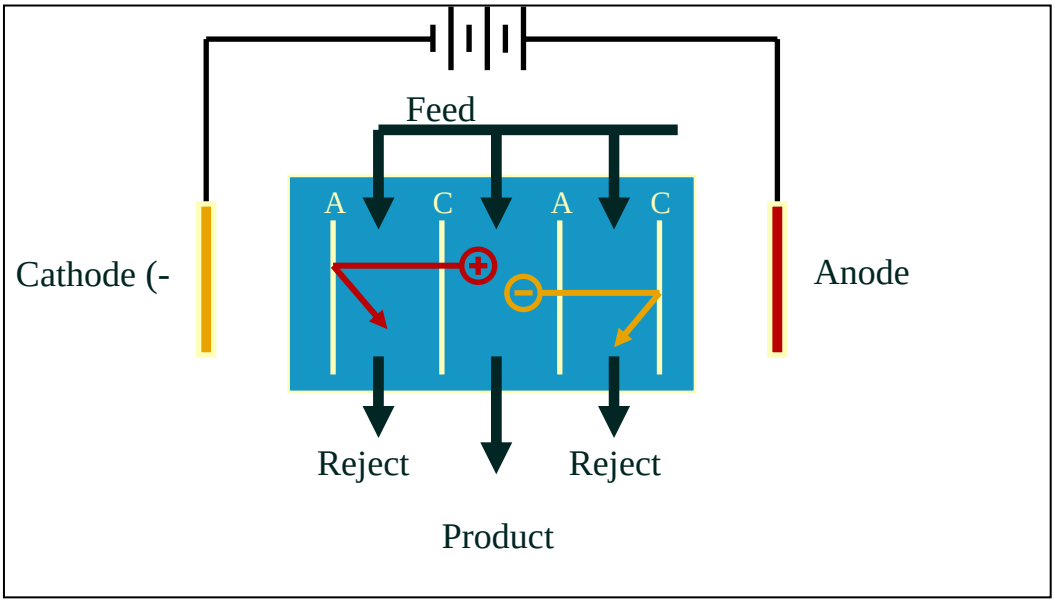
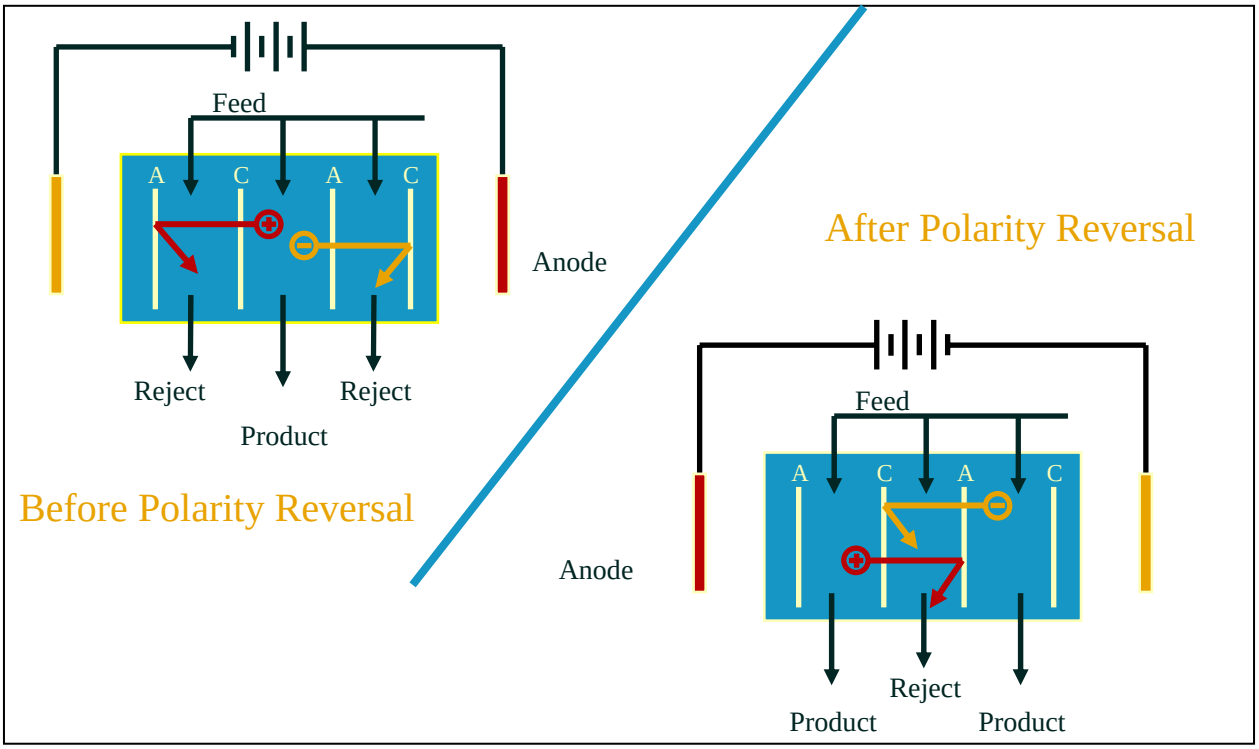


Figure 8-5
EDR System



EDR System for Fox Hills Wells

Due to the limited use of ED and EDR, the primary supplier of this technology is GE Infrastructure. The GE Infrastructure EDR System for the Fox Hills Wells would treat blended water from the three wells. The EDR system would include two EDR 20205/3 systems with each system having 5 parallel lines of three stages in series EDR membrane stacks. If redundancy was desired, a 3rd unit could be added. The EDR can operate at up to 93% water recovery on this water supply, however, extensive chemical pretreatment is required to prevent scaling. Figure 8-6 shows the system would supply 1,691 gpm of blended water and only 108 gpm of concentrate. To obtain the 93% water recovery, acid (specifically HCl) and antiscalant would need to be added. As an alternative, the EDR system could be operated at a lower recovery (i.e. 85%), but the pumping requirements for the increased concentrate offsets the DC power savings by having EDR run with a reduced 10,000 ppm brine (instead of 21,000 ppm). For the same power requirements, EDR can deliver more usable blended water (1,691 gpm) at 93% recovery. It should be noted that the EDR process is not ideal for removal of fluoride due to the ion selectivity of EDR membranes. The selectivity sequence is similar to ion exchange as shown later in Section 8.1.2.2.3. The majority of competing ions must be removed before the fluoride is removed, which results in a process that is not as efficient as RO in this application.

Figure 8-6
EDR System for Fox Hills

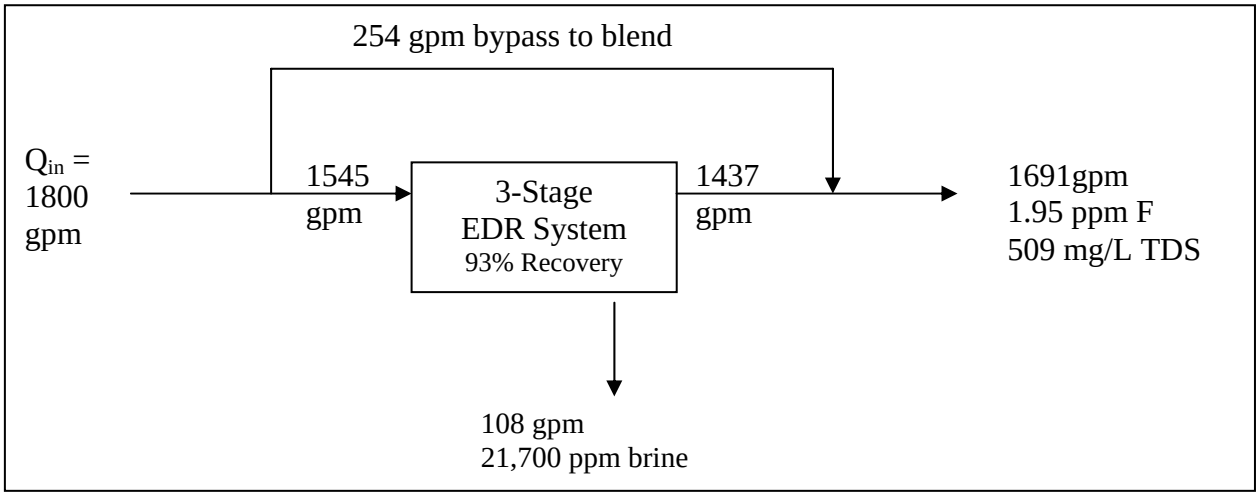


Table 8-3 gives the advantages and disadvantages of EDR Systems.

TABLE 8-3
Advantages and Disadvantages of EDR Systems

Advantages	Disadvantages
Does not require pretreatment	High power requirements
Low pressure requirements	Disposal of concentrate
Self Cleaning (EDR)	Low selectivity for Fluoride
	Does not remove uncharged molecules (including suspended solids)

A preliminary cost opinion was provided by GE Infrastructure; their preliminary cost opinion included *only* the cost of EDR equipment and installation. The preliminary cost from GE Infrastructure was $3.85 million, compared to a similar RO equipment quote of $1,000,000 (discussed below) . Due to preliminary cost opinions and the selectivity of the membranes, EDR is not a preferred option for the treatment of the ground water from the Fox Hills formation.

8.1.2.2.3 Treatment Alternative 3 - Ion Exchange

Ion exchange (IX) processes are yet another way to remove cations and anions present in raw water. There are four basic steps for ion exchange systems: 1) exhaustion, 2) backwash, 3) regeneration and 4) slow and fast rinse. IX uses a presaturant ion (the adsorbent) on a solid particle to exchange with an unwanted ion in the water. The presaturated IX media is contained in a packed bed while the source water is passed through the bed allowing the exchange to take place. Ultimately, the adsorbent is exhausted and an unacceptable concentration of the unwanted ion is present in the IX effluent. The IX media is then regenerated using a brine solution or replaced. IX processes are discussed because of their ability to remove the ions associated with TDS and hardness.

The ion exchange media can be made of gel or macroporous particles for both anion and cation exchange resins. The specific resin needed depends on the contaminants present in the source water. The water in the Fox Hills Wells would require strong acid cation (SAC) and strong base anion (SBA) resins designed to reduce TDS and remove fluoride.. A brine solution made with NaCl can be used to regenerate the resin.

The selectivity of ion exchange resins is as follows for cation and anion resins:

Cationic Preference Series

$Ba^{2+} > Pb^{2+} > Sr^{2+} > Ca^{2+} > Ni^{2+} > Cd^{2+} > Cu^{2+} > Co^{2+} > Zn^{2+} > Mg^{2+}$
$> Ag^{+} > Cs^{+} > K^{+} > NH_4^{+} > H^{+}$

Anionic Preference Series

$SO4^{2-} > I^{-} > NO_3^{-} > HCrO_4^{-} > Br^{-} > Cl^{-} > CN^{-} > HCO_3^{-} > OH^{-} > F^{-}$

Ion Exchange System for Fox Hills Wells

For the Fox Hills Wells, Dowex® UPS resins (Dow Chemical Corporation) were evaluated. The UPS technology allows the diameter of the particles to be similar in size (95% of the particle diameters are within a 100 microns of the average diameter) while conventional resins can range in size from 300-1200 microns. A counter current ion exchange regeneration system is proposed. The regenerant is applied in the opposite direction of the feedwater flow, resulting in reduced chemical consumption, improved water quality and less waste volumes compared to traditional co-current regenerated systems. One type of counter current regeneration system is the packed bed system. These may be up-flow feedwater service with down-flow regeneration or down-flow feedwater service with up-flow regeneration.

The ion exchange vessels should be made from materials such as rubber-lined carbon steel or fiberglass. The vessel should have distribution / collector systems that give a good distribution of fluids during all phases of the operation. For this reason, the maximum vessel diameter should not be greater than 3.5 m (11.5 feet). It is advisable to install sight-glasses in order to check resin levels and separation in the case of layered beds and mixed beds. The optimum column diameter must be a balance between the resin bed height, the ratio of resin height to diameter (H/D) and the linear velocity. H/D should be in the range 2/3 to 3/2. The minimum depth of the resin must be at least 30-inches.

Vessel sizing should be adjusted to allow for resin expansion if backwashing is performed (80-100% of the settled resin bed height), resin swelling during service (5-8% for SAC and 15-25% for SBA), the minimum bed height requirements (1.2 meter minimum for counter current regeneration) and other manufacturer's design guidelines. The total rinse requirement for these resins is 3-6 bed volumes. These values are for reference and should not be regarded as exclusive to all applications. Some applications may function outside of the guidelines.

To help determine the number of vessels and number of lines needed for the IX system for TDS and fluoride reduction for the Fox Hills Formation, the DOW CADIX System Design Software for the design of IX systems was used. Water quality data for the Fox Hills Formation was entered into the CADIX Software. Based on the results of the design evaluation, the loading rate on the ion exchange resin will be extremely high resulting in a large number of vessels. In addition, the evaluation indicates that the quantity of chemical required to regenerate the resin is not practical. Therefore, utilizing IX for TDS and fluoride reduction was removed from further consideration. For reference, the software output is provided in Appendix C-3 for a flowrate of 25 gpm.

Table 8-4 shows the advantages and disadvantages of ion exchange systems.

TABLE 8-4
Advantages and Disadvantages of IX Systems

Advantages	Disadvantages
Ease of Operation, highly reliable	Requires salt storage and regular regeneration
Resins will not wear out with regular regeneration	Strongly basic anion exchange resins susceptible to organic fouling
Suitable for small and large installations	Waste disposal

8.1.2.2.3 Treatment Alternative 4 - Ion Exchange + Reverse Osmosis

This alternative utilizes an ion exchange vessel designed only for removal of scale forming cations placed upstream of the RO membranes. The alternative may still require the addition of acid or antiscalant to prevent the buildup of salts during the filtration process, but the amount of chemical should be dramatically reduced.

The CADIX results show that for the three and four vessel system with a 14 day regeneration time, the tank diameter should be a minimum of 2500 mm (8.2 ft). For the 7 day regeneration time, the tank diameter should be a minimum of 2100 mm (6.9 ft). Additional information for the specified IX System regarding regeneration chemical (NaCl 10%), dosage, regeneration sequences and waste composition can be located on the CADIX results data sheets provided in Appendix C-4.

For this alternative, a sodium-based strong acid cation (SAC) ion exchange resin must be utilized. The use of this form of ion exchange resin can achieve removal efficiency of Ca^{2+}, Ba^{2+}, and Sr^{2+} to greater than 99%. The strong acid cation exchange resin can be regenerated using NaCl. Figure 8-7 shows the layout for the IX and RO systems.

The resin selected for pilot testing in summer of 2007 will be supplied by DOW and will be the uniform particle size (UPS) resin with capacity cation exchange abilities for softening and demineralization applications. The resin was chosen after isotherm testing was completed by the City of Gillette. The City tested four types of resin from DOW Chemical Corporation: Marathon C-10 (SAC), Marathon MSC (SAC), and MAC-3 (WAC). The MAC-3 is a weak acid cation. Cut sheets from Dow Chemical Corporation are in Appendix C-5. Based on the isotherm testing, the Marathon C-10 resin is most suitable for pretreatment of the Fox Hills Formation water. The necessary volume of the optimum resin is given on the CADIX results data sheets.

Design guidelines indicate if the flow is greater than 1,800 gpm, it is best to have at least three vessels; two in parallel with one used as a standby. The CADIX model was run for both a three (3) vessel system (two in parallel and one standby) and for a four (4) vessel system (three (3) in parallel and one standby). A regeneration time of 7 days and 14 days was run to determine the best possible system. For the IX system in this application, it was determined that three vessels

Figure 8-7

could be used rather than four because this system is being used as a pretreatment method for RO. The RO system will still utilize four trains to ensure the entire capacity of the Fox Hills Formation is used.

8.1.2.2.4 Treatment Alternative 5 - ED/EDR + Reverse Osmosis

RO can be used in combination with either ED or EDR to effectively remove microbial contaminants and enteric viruses. The combination of these two treatment alternatives will also concentrate the waste stream further. The ED/EDR System would be placed upstream of the RO System. Design, operation and maintenance, and chemical addition properties remain the same for each system as previously described.

Due to the preliminary cost opinions presented in 8.1.2.2.2, the evaluation indicates that the use of EDR as a pretreatment process for RO is not a feasible option.

8.1.2.3 Residual Handling Alternatives

Concentrate disposal is a critical element in the design of the RO plant and a potential major cost issue. The composition of the waste stream is dependent on influent water quality, as well as the design and actual performance and operation of the RO system. Furthermore, the design of the membrane system is highly dependent on the method of disposal, since the target recovery (and corresponding concentrate salinity) may differ depending on the selected method of disposal. In many cases, it may be necessary to either minimize the quantity of reject water (high recovery design) or minimize the concentration of certain constituents (low/moderate recovery design).

For concentrate disposal considerations the most important water quality parameters are:

- The general TDS level
- The levels of ions and silica that might limit membrane system recovery
- Levels of constituents that are closely regulated
- The presence of any contaminants that might render concentrate, brine, or solids derived from the concentrate to be hazardous.

Over 98% of the approximately 260 municipal membrane desalination plants in the 50 states (of size over 25,000 GPD) dispose of concentrate via five 'traditional' disposal options (1, 2). The options and percent use are:

- Surface water disposal 46%
- Disposal to sewer 27%
- Subsurface injection 13%
- Land application 8%
- Evaporation pond 4%

Other possible disposal options include, but not discussed due to high operational costs:

- Spray irrigation
- Enhanced evaporation
- Rapid infiltration

Preliminary capital cost opinions for some of the residual handling alternatives are presented in Appendix A. Preliminary conceptual cost opinions were developed for Deep Well Injection and Evaporation Ponds.

8.1.2.3.1 Surface Water Disposal

The City of Gillette is located in the east-central Powder River Basin (PRB). The PRB is a 20,000 square mile area occupying most of the northeast quadrant of the state of Wyoming, bounded by the Black Hills to the east, the Big Horn Mountains to the west, and the Laramie Range/Hartville Uplift to the south. The PRB has five stream drainages basins forming the Upper Cheyenne River, the Belle Fourche River, the Little Missouri River, the Powder River, and the Tongue River. None of these streams produce much water. The lack of water in the streams does not provide an appropriate location to dispose of RO concentrate as the concentrate would not be adequately diluted and the RO concentrate could adversely affect downstream users. Therefore, disposal by discharging to a surface water source is judged NOT to be feasible due to lack of viable surface water options.

8.1.2.3.2 Discharge to Sewer/Wastewater Treatment Plant

Discharge of membrane concentrates to WWTPs is an acceptable practice as long as the concentrate is not toxic to the biomass at the WWTP and does not adversely impact clarifier settling or restrict final effluent disposal. Often, disposal of RO concentrate to sewer is the easiest and most cost-effective option available. Many WTPs list TDS and turbidity as the main reasons to send RO concentrate to WWTPs. However, the number of RO systems discharging to WWTPs is very small due to a number of issues. This section of the technical memorandum will discuss permitting, the City of Torrington, Wyoming's RO concentrate disposal to their wastewater lagoons, and potential challenges the City of Gillette may face in disposing RO concentrate from the Fox Hills Wells to the Gillette WWTP.

Roland Peterson[11], WYDEQ, was contacted to determine permitting procedures that would need to be followed to discharge the RO concentrate to the Gillette WWTP. He indicated that the City of Gillette would need to consider possible permitting issues, including:

- The WWTP may experience problems achieving NPDES permit conditions if the concentration of pollutants into the WWTP is too high
- All discharges to the WWTP must comply with EPA general pretreatment requirements, under which, WTP residuals are classified as industrial wastes. However, RO concentrate is not regulated as a primary industrial point source.
- Other permit obstacles could develop as a result of high TDS levels, unstable pH due to concentration of carbonate species and addition of anti-scaling agents, high concentrations of heavy metals, and other corrosive causing constituents.

The City of Torrington currently disposes of RO concentrate at their WWTP, and were contacted to obtain information useful in this evaluation for the City of Gillette. The City of Torrington obtains municipal drinking water from ground water wells, which is known to have high nitrate and TDS concentrations. The RO system utilized by Torrington went online in the year 2000 and consists of various satellite RO facilities around the community. RO concentrate originating from the various facilities is discharged to the sewer and ultimately conveyed to the City wastewater lagoons for treatment of municipal wastewater. Previously the City diluted the RO concentrate with North Platte River water and then pumped to the lagoons. This practice was

changed after WYDEQ determined the City does not need to dilute with river water before discharging the RO concentrate to the wastewater lagoons.

Based on discussions with Jim Craig[12], with the City of Torrington, it is clear the blending ratio conveyed to the wastewater plant would be critical. Mr. Craig mentioned it would be more important to understand the blending ratio for the case of the Gillette WWTP, which has an activated sludge process. RO concentrates tend to be briny solutions with high salinity concentrations and high TDS concentrations that could be toxic to wastewater treatment facility microbial populations.

The treatment of the Fox Hills Wells with RO will result in a concentrate amount as high as 0.65 MGD that would require disposal. The Gillette WWTP currently treats approximately 2.5 MGD. Therefore, the RO concentrate stream would be a significant influent flow addition. The total RO concentrate stream, if combined with the municipal wastewater influent flow, would be 21% of the total combined peak flow.

The Fox Hills Wells tend to have higher than normal fluoride concentrations. Fluorine is a naturally occurring element in the earth's crust and is often bound to calcium or magnesium. The naturally occurring high concentrations of calcium fluoride compounds present in the Gillette source water could disassociate into calcium and fluorine ions as a result of pH changes that occur in sewer conveyance systems and treatment structures. Consequently, liberated fluorine ions could potentially poison the wastewater microbiology at the Gillette WWTP and upset the treatment process by acting as an enzyme inhibitor and kill the bacteria.

The Gillette WWTP bacterial growth could also be inhibited as a result of unstable pH. An unstable pH could result from addition of the RO concentrate because it can either have a high pH as a result of concentration of carbonate ions or a low pH because cleaning agents and anti-scaling agents are often added to remove membrane foulants. Low pH not only inhibits many bacterial metabolic processes, it can cause serious corrosion to facility conveyance and treatment structures.

Since the RO concentrate typically would have a high TDS concentration, the Gillette WWTP overall treatment process efficiency may go down. An important factor that would require further investigation is that Gillette's WWTP effluent is reused by the Black Hills Corporation's

Wyodak Power Plant as cooling water. It appears the option to treat RO concentrate at the WWTP and/or to blend the RO concentrate with wastewater effluent could adversely impact the power plant cooling operations and agreement between the two entities because of poorer effluent quality that may result. In the event that the concentrate were piped to the WWTP for blending with the plant effluent, capital cost on a worst case basis would be for the construction of a sewer pipeline between the proposed Fox Hills RO WTP to the WWTP. If the pipeline were constructed along major roadways in the City, the distance between the WTP and the WWTP would range between 13,000 to 15,000 feet (approximately 2.5 miles to 2.9 miles). The preliminary construction cost would include the capital cost of the pipeline construction materials, excavation costs, and labor. The pipeline would not be able to flow by gravity and would require pumping due to surface elevation changes along the various pipeline routes that were examined.

Based on the reasons provided, disposal of RO concentrate at the Gillette WWTP is not a preferred alternative.

8.1.2.3.3 Deep Well Injection via Class 1 Well

Disposal of concentrate via deep well injection requires a Class I injection well due to the industrial classification of municipal desalination concentrate. The practical requirements for feasibility include finding an aquifer:

- of sufficient capacity to take concentrate over the life of the plant
- of great enough porosity and permeability to take up concentrate at a reasonable rate (otherwise multiple widely spaced wells would be required)
- that is structurally isolated from overlying drinking water aquifers

Deep well injection is a disposal method in which concentrate is injected hundreds or thousands of feet below the groundwater surface into an injection zone. Injection zones consist of materials with high TDS and an overlaying confining layer. In the past, deep well injection wells were developed by the oil industry to dispose of oil field brines. Typically deep well injection requires a pump but in the case of reverse osmosis facilities, the concentrate pressure may be utilized for injection. Design issues with a deep well are the injection well shaft, casing, cement

grout, injection zone, storage, and regulatory monitoring. The injection well shaft should be compatible with the high TDS concentrate stream. The deep well is always a multi-cased well with more casings at the top of the well. This configuration protects the lower TDS waters that are located closer to the surface. Storage of concentrate should be provided to allow for maintenance of the well if required. Regulatory monitoring of these wells on a monthly basis would probably be required. The use of a Class I injection well is a good match for the high strength discharge that will be disposed of and is an environmentally acceptable method. Issues related to deep well injection include:

- Possible storage requirements
- Regulatory monitoring
- RO recovery dependent on injection zone

Dennis Lamb, WYDEQ, was contacted to see if there was a Class I injection well near the Fox Hills Wells that could be used as a deep injection well for the RO concentrate.[3] Mr. Lamb confirmed a Class I injection well would be required. Mr. Lamb indicated there are two (2) Class I injection wells in the vicinity of the City of Gillette. One well is located approximately 15 miles away in Rozet, Wyoming (Campbell County) and the other well is located approximately 45 miles away in Sheridan, Wyoming (Sheridan County).

The first well, located east of Gillette in Rozet, Wyoming, is owned by Kissack Water & Oil Service, Inc. (6 Mine of Rozet, Rozet, Wyoming 82727). The Wyoming UIC Permit on file for this well is listed as “Kissack Disposal Well #1”, Facility # WYS 005-110. Jo Kissack was contacted to discuss the possibility of being able to discharge the concentrate into the Kissack Disposal Well #1.[4] Ms. Kissack indicated that as long as it is proven that the waste is not hazardous, they would consider accepting it. She indicated that the capacity of the well is 7,000 barrels/day (0.294 MGD). Ms. Kissack indicated this well accepts waste from other customers so the capacity that they could accept may be lower than 0.294 MGD.

Ms. Kissack indicated that approximately 180 barrels (or 7,560 gallons) could be picked up and trucked back to the well at a rough cost of approximately $1,500 - $2,500/load. If a pipeline was to be built to the well, this pipeline would need to be roughly 15 miles.

The second well, located northwest of Gillette in Sheridan, Wyoming, is owned by Emit Technologies in Sheridan (772 Airfield Lane, Sheridan, Wyoming 82801-5857). Per discussions with Dennis Lamb, Wyoming DEQ, the capacity in this well is either leased or purchased by the coal bed methane gas users. Since this well is approximately 45 miles away for the City of Gillette, this well was not evaluated in further detail.

Another option is for the City of Gillette to permit and construct a Class I injection well. The preliminary evaluation indicates that disposal by deep well injection is likely feasible. Mr. Lamb indicated getting the permit for a Class I injection well is a very tedious process, with a lot of requirements for the well itself as well as integrity monitoring, etc. Dennis Lamb indicated to develop a new well would most likely cost approximately, $6- $8 million dollars. The preliminary conceptual cost opinion developed by Burns & McDonnell is provided in Appendix A.

8.1.2.3.4 Evaporation Ponds

Evaporation ponds do not require any permits if there is no chance of interference with ground water or surface water. A liner can be used to prevent the brine solution from seeping into the ground water supply. The ground water near the evaporation pond should be monitored to ensure that no seepage is occurring.

Evaporation ponds are used only for small volume disposal in areas with low net evaporation such as Gillette. This alternative can be illustrated by assuming reasonable values for the net evaporation rate and cost per acre of evaporation pond. Using a value of 42.45 inches per year of evaporation (PAN data)[5] and 15 inches per year of precipitation, a net PAN evaporation rate of 31.4 inches per year is obtained. The City of Gillette annually receives between 11-15 inches of precipitation every year; 15-inches of precipitation was used to size the evaporation pond to help ensure the pond would be large enough to accommodate the amount of brine solution. The PAN based net evaporation rates are multiplied by a 0.74[6] factor to account for the decrease in evaporation of brines and the changing salinity of the ponds over the pond life.

At this net evaporation rate, 0.65 MGD (450 gpm) of concentrate would require about 625 acres of evaporation surface area. For the purpose of this report, it was assumed that the City of Gillette owns approximately 625 acres near the location of the Fox Hills Wells and would not need to purchase additional land.

Requirements and costs for evaporation pond include:

- Level land – For the preliminary cost opinion, it was assumed the City of Gillette owns the land.
- Land clearing
- Earth moving (substantial volumes)
- Pond construction – liner and dike to prevent interference with ground water and/or surface water
- Monitoring
- Conveyance of concentrate to ponds

8.1.2.4 Preliminary Cost Opinions for Treatment Alternatives

The Preliminary Conceptual Cost Opinions for the feasible treatment alternatives (Alternatives 1 and 4) are given in Appendix A. Detailed cost estimates were not prepared for the alternatives determined to not be feasible as described in detail in this report (Alternatives 2, 3, and 5). Below is a brief overall summary of the preliminary conceptual cost opinions for the feasible alternatives.

Treatment Alternative 1:

- RO System – $ 9.4 million

Refer to the preliminary conceptual cost opinion shown in Appendix A for a complete break-out of costs for the entire RO system. Note that costs associated with RO do not include brine disposal and potential treatment of brine prior to disposal. Additional funds will be required for disposal as outlined in the next section. It should be noted that these cost estimates are based on the water quality data available at the time of this evaluation, and do not include treatment for silica or suspended solids.

Treatment Alternative 4:

- Ion Exchange – RO System - $12.3 million

Refer to the preliminary conceptual cost opinion shown in Appendix A for a complete break-out of costs for the entire IX-RO system. Note that costs associated with RO do not include brine disposal and potential treatment of brine prior to disposal. Additional funds will be required for disposal as outlined below. It should be noted that these cost estimates are based on the water quality data available at the time of this evaluation, and do not include treatment for silica or suspended solids.

8.1.2.5 Preliminary Cost Opinions for Concentrate Disposal

Preliminary cost opinions for the feasible concentrate disposal options are provided in Appendix A. These conceptual cost opinions do not account for unforeseen regulatory issues or costs that may arise, and should be treated as purely conceptual until further investigation and predesign efforts can be completed.

Deep Well Injection:

- Class 1 Well – $7.7 – 8.9 million (depending on RO recovery)

Evaporation Ponds:

- 625 Acre Evaporation Ponds – $45.6 million

Based on the cost opinions and preliminary investigation, deep well injection is the preferred method of concentrate disposal.

8.1.2.6 Total Alternative Capital Cost

The total alternative capital costs for treating Fox Hills water are:

Reverse Osmosis	$18.3 million, or $13,555 per gpm of finished water.
Reverse Osmosis + Ion Exchange	$20.0 million, or $12,679 per gpm of finished water.

8.1.3 Utilize Coal Bed Methane Water

The most reasonable delivery point for CBM water would be the City's existing Pump Station #1. This pump station has gas stripping equipment that is already setup for stripping gases form

Fox Hill and Fort Union sources and it is toward the west end of town. CBM water would likely come from the west form the Powder River area. While there is some excess capacity in the Pump Station #1 now, in order to handle the additional 8-year projected demand, the station would need to be expanded.

Discussions with CBM operators have shown that there is some interest in providing water to the City of Gillette. A letter inviting CBM producers in the Gillette area to a meeting to discuss the use of CBM water is included in Appendix G. A sign-in sheet of those attending that meeting is included in the same appendix. At that meeting concerns were raised about liability, delivery rates, cost sharing, and timing. At the present time, these issues have not been resolved, and it is not clear if they can or can not be resolved in the future.

In these discussions with CBM operators, it was clear that their preference would be to deliver the water at a rate as close to constant as possible. Delivering water on a peaking basis would be a difficult task for them to comply with and would likely make this venture unattractive to them as a water disposal method. Constant delivery is in direct competition with the varying demand imposed on Gillette's system. Given these differing delivery and receiving goals, storage most likely is required. The optimal location for CBM water storage is west of Gillette, while the optimal location for Madison surface water storage (discussed further in the long-term alternatives) is on the east side of Gillette. It is estimated that a short-term water source would require storage on the order of 1,000 ac-ft. Storage of this size is obviously only feasible in a surface water or groundwater impoundment (aquifer storage and recovery). Both of these alternatives present problems as a short-term solution.

Once groundwater is discharged to a surface impoundment, it is considered surface water for treatment regulation purposes. This situation means that a typical surface water treatment plant would need to be constructed to treat the water before delivering it as potable water. Construction of a reservoir and treatment plant would require a significant amount of time. This time requirement is a critical flaw in a short-term alternative.

An aquifer storage and recovery (ASR) project is also undesirable for a short-term alternative. There are no ASR projects in the area which have proven this concept is feasible. While it may be feasible, pursuing it poses a number or regulatory and technical challenges that at this time do not have clear answers. Any of these issues could ultimately become a fatal flaw which

would make this concept unviable. The pursuit of such a program is not inherently unfeasible; however, the time requirement to pursue such a problem is a critical flaw in a short-term alternative.

Given the storage problems discussed above, the only short-term alternative available using CBM water is to route the water through Pump Station #1 on a real-time as-needed basis (no storage). Currently, Pump Station #1 has an ultimate treatment capacity of 3,500 gpm with a firm treatment capacity of 1,750 gpm. Pump Station #1 has an ultimate pumping capacity of 3,680 gpm (Wester-Wetstein 2004 Master Plan Report) and a firm pumping capacity of 2,620 gpm. The peak flow pumped through Pump Station #1 is approximately 2.9 MGD (2,013 gpm pumped 24 hours/day). This currently exceeds the treatment firm capacity and is approximately 800,000 gpd, or 555 gpm, under the firm pumping capacity. This leaves little room for firm capacity improvement by bringing CBM water into Pump Station #1.

Expanding Pump Station #1 then becomes necessary to make use of CBM. However, this still leaves the problem of variable demands and delivery rates. The average per capita winter flowrate is 92 gpcd. The 2015 population of 34,915 produces a winter demand of 3.2 MGD. Averaging this over 24 hours gives a flow rate of 2,230 gpm. Assuming a constant intake rate, this means that assuming the most extreme case, all of Gillette's winter water supply would come from CBM water. Even in this extreme instance, with a constant flow rate, CBM water would only be able to be delivered at a maximum rate equal to the average winter demand. Assuming an expansion of Pump Station #1, only 1,675 gpm of additional flow capacity would be available under a constant delivery scenario.

The level of uncertainty surrounding this alternative is very high. There are a number of issues that would need to all be worked out to make this viable. It can be seen from the discussion above how critical the rate of delivery and constancy of delivery rate is. Further uncertainties are discussed in more detail in Chapter 9. The high level of uncertainty surrounding this alternative at this time, including the uncertainty surrounding funding split, introduces a number of variables into this alternative that makes cost-estimating impossible at this time.

Due to large number of unknowns discussed above, at this time it is concluded that using CBM water as a short-term source for the City of Gillette is not a reliable option and as such can not

be considered feasible. If the issues discussed above and in Chapter 9 that make this a high-risk endeavor at this time can be worked out, CBM water could be a viable option.

8.1.4 Short-term Alternative Recommendations

The summary in Table 8-5 outlines the parameters associated with each short-term option.

TABLE 8-5
Short-Term Alternatives Summary

Alternative	Capital Cost	(Estimated) Flow Provided (gpm)	Cost ($/gpm)	Time To Implement (Yr)
Expand Fort Union Source	$74,249,000	6,450	$11,500	5
Fox Hills Treatment - RO	$18,307,000	1,350	$13,600	2.5
Fox Hills Treatment - RO/IX	$20,014,000	1,530	$13,081	2.5

It can be seen from this summary that no alternative that provides the full 6,450 gpm projected incremental flow increase can be constructed in less than 5 years. The Fox Hills treatment options are the most expensive on a $/gpm basis and only provide approximately 21-24% of the projected flow. None of these alternatives are feasible short-term solutions for the City of Gillette.

Given the costs and time frames associated with these alternatives, another approach must be taken as an alternate to the traditional flow projection and firm capacity planning analysis. It is obvious from these costs and time frames that the short-term situation in Gillette is dire. The importance of implementing a long-term water supply for Gillette can not be over-emphasized. It is absolutely crucial that a long-term supply be implemented as soon as possible.

In the meantime, the following measures should be undertaken to provide stop-gap measures to attempt to allow Gillette's water supply to meet demand until a long-term solution is commissioned:

8.1.4.1 Conservation

The City of Gillette is currently engaged in a water conservation campaign implemented for the 2007 peak water use period. The effects of this effort should be closely studied. If necessary, this program should be escalated in campaign aggressiveness first, and in mandatory watering restrictions second. If conservation efforts can reduce the usage of Gillette water users by 10% on the peak day, this is the equivalent of a 1,675 gpm source. For 15% conservation on the peak day, an equivalent of approximately 2,500 gpm can be expected. It is important to communicate to the public that they do not have to ration water year-round, only during periods of extreme demand. There are no incremental costs associated with this program as Gillette is already engaged in such an activity.

8.1.4.2 Continued Fort Union Development

The City should continue to replace existing poorly performing wells with gravel-pack constructed wells capable of increased yields. This approach allows reuse of raw water pipe infrastructure where feasible. This approach can be used to increase production up to the limits of Pump Station #1, or an approximate ultimate increase of 1,487 gpm without any work to Pump Station #1. Well drilling efforts beyond this point will require an increase in the capacity of Pump Station #1. At an estimated new yield of 300 gpm, upgrading the remaining Fort Union wells constructed prior to 1980 (S-9, S-17, S-18, and S-19) should realize an increased yield of approximately 813 gpm. When added to the increased yield from the redrilled S-20 and the soon-to-be-drilled S-12, the total anticipated yield assuming no interference between wells is 1,288 gpm. Redrilling one of the poorly performing (100 gpm) newer Fort Union Wells will bring this increase up to approximately 1,488 gpm of increased flow, which essentially places Pump Station #1 at its maximum treatment capacity. The construction costs for the S-12 and S-20 replacement wells recently bid by Gillette were approximately $915,000 each. With engineering, these wells are approximately $1.1 Million. Drilling prices are difficult to predict in the Gillette area due to the currently lucrative oil and gas drilling business climate. A program of replacing more wells at one time such as all of the Fort Union wells listed for replacement above, could draw more drilling interest and bring better prices.

8.1.4.3 Temporarily Treat the Fox Hills Source

Rather than construct a permanent treatment plant, a skid mounted RO (or IX/RO if one can be located) plant should be pursued. The residuals from this plant could be split between the Kissack deep injection well and whatever the WWTP can accept. This option should be pursued only as an alternative of last resort due to the limitations, difficulties, and costs associated with disposal of the residuals from this process. This alternative could produce between 1,350 and 1,530 gpm depending on the membrane reject rate and assuming no residuals disposal limitations. How the Gillette Wastewater Treatment Plant (WWTP) can deal with a short-term, limited duration residuals stream will significantly impact the residuals handling of this process and the associated possible flow limitations related to residuals disposal. The costs of finding and procuring or renting a skid mounted unit appropriate to Gillette's treatment needs are unknown at this time. Perhaps a larger unknown are the residual disposal costs and associated issues/costs at the WWTP. It appears that some availability in the Kissack well is obtainable, but the exact extents of this are unknown at this time. At the Kissack quoted price of $1,500-2,500 per 7,560 gallons load, daily disposal costs for this alternative could be as high as $75,000 per day, or approximately 18 times the revenue the City could collect from selling that water.

8.1.4.4 Expediting the Long-Term Solution

Each year that passes before a long-term solution is implemented results in an increased peak day demand of approximately 185 gpm. Obviously expediting the long-term solution in conjunction with the other emergency alternatives will only help the short-term water supply shortage. This further reinforces the importance of implementing a long-term water supply as soon as possible.

8.2 LONG-TERM ALTERNATIVES

The conclusion from the long-term water sources discussion in Chapter 7 was that the Madison was the only reliable long-term water source that could produce enough water to meet the increased water demands in Gillette in 2037. Three alternatives were developed to make use of the Madison source:

1. New Madison Well Field and Transmission Main
2. Surface Water Impoundment of Madison Water with a Surface Water Treatment Plant and New Transmission Main
3. Groundwater Storage and Recovery of Madison Water in the Fort Union Aquifer

All of these alternatives rely on continued use of the existing Madison system. There are currently on-going efforts to determine the condition and expected life of the existing Madison system. Since this is a maintenance activity that is common to all alternatives, and it is being addressed under other studies, the topic is not covered in-depth herein.

Another problem common to all alternatives is that of fire flow and water quality issues in the City of Gillette distribution system. These problems were preliminarily analyzed as part of this study to determine if they require any additional requirements of the long-term water supply system. Problems of both natures were found in the system, but there are a number of solutions that can be pursued with the distribution system, such as rezoning, fire booster, repiping the existing reservoirs, and operational and extension policy decisions that can help with these issues. These solutions are by far more cost effective than working on either of these issues with the long-term water supply system. With sources significant distances from use centers, any upsizing or additional requirements on the long-term supply create drastic cost increases. Since contractors will likely be in the vicinity of some of the tanks with mixing problems already, the costs to alleviate these problems have been included in each long-term alternative. The City of Gillette is currently pursuing the other distribution oriented solutions to these problems, which are not discussed further here as they are outside the scope of this report.

8.2.1 New Madison Well Field and Transmission Main

This alternative consists of building another parallel system similar to the existing Madison infrastructure. This system would include a new Madison well field, new pumping and storage facilities near the existing Madison Pump Station, new transmission main into town with new booster stations as necessary to get the water into Gillette.

This section presents an analysis overview of the proposed transmission main alternative for supplying water to the City of Gillette from the Madison Well Field. Numerous alternatives were

formulated and evaluated for the approximately 52 mile long transmission main, including various pipeline diameters, pump station locations, and pipeline materials.

Each of the proposed alternatives were analyzed to determine their respective hydraulic grade lines (HGL) and required pump heads to supply the City of Gillette Terminal Reservoir with a peak flow of 13,000 gpm from the Madison Well Field. A cursory cost estimate was performed for each of the developed alternatives to reduce the selection down to the two most cost viable alternatives. The estimates included values for both capital infrastructure costs and operational costs associated with power consumption by the proposed pump stations.

Once the two most economical alternatives were determined, further hydraulic analysis was conducted, including transient analysis (also known as surge or water hammer analysis). This more detailed analysis evaluated the effects of potential transient pressure waves within the proposed transmission main alternatives. The transient analysis determined the appropriate surge mitigation devices required for protection of the transmission main alternatives, such as surge relief valves or surge bladder tanks. It also showed areas of the proposed transmission mains that required an increase in pipeline pressure class above that of the operational or static pressure, where total mitigation of transients was not possible.

A final cost estimate was prepared once the in depth hydraulic and surge analysis was accomplished. This cost estimate takes into account specific appurtenances such as isolation valves, fittings, and surge mitigation devices, and is presented at the end of this section.

8.2.1.1 Existing Transmission Main

The existing transmission main begins at the Madison Well Field Pump Station. Groundwater collected from the well field is accumulated and pumped to the Pine Ridge Reservoir. The operating high water surface elevation of the Pine Ridge Reservoir is 4,537 feet, and the low water surface elevation is 4,530 feet. From the reservoir, a 30-inch diameter transmission main conveys the water via gravity flow to the Donkey Creek Pump Station. This transmission main was designed for a flow rate of 7,000 gpm with a Hazen-Williams coefficient of 130. However, a recent flow test indicated a tested C value of 149. The total length of transmission main from the reservoir to pump station is approximately 26 miles. This portion of the transmission main also has two turnouts to the municipalities of Rozet and Moorcroft.

The design inflow HGL of the existing transmission main at the Donkey Creek Pump Station is approximately 4,371 feet. The design discharge HGL of the transmission main leaving the pump station is 4,844 feet. The difference of these two HGL's is 473 feet, which corresponds to the existing pumps total dynamic head (TDH) required to deliver water to the 7 million gallon Terminal Reservoir. The transmission main from the Donkey Creek Pump Station to the Terminal Reservoir is a 30-inch diameter pipeline with a design flow rate of 7,000 gpm and Hazen-Williams coefficient of 130. This section of the existing Madison system operates very near the pipe pressure ratings in a number of locations. The total length of transmission main from the Donkey Creek Pump Station to the Terminal Reservoir is approximately 15 miles. The operating high water surface elevation of the Terminal Reservoir is 4,762 feet, and the low water surface elevation is 4,750 feet.

The existing Donkey Creek Pump Station has a receiving wet well for water delivered from the Pine Ridge Reservoir. Water is delivered from the wet well to the terminal reservoir via a gallery of vertical turbine pumps.

The existing transmission main can deliver between 8,300 and 8,400 gpm. The City of Gillette is working on adding surge controls to this line to bring the capacity up to a maximum of 8,800 gpm.

8.2.1.2 Alternative Development

The first step in developing alternatives for a new transmission main from the Pine Ridge Reservoir to the Terminal Reservoir was to determine a pipeline alignment. The existing transmission main alignment was used for alternative analysis due to the availability of information. The James M. Montgomery Madison Transmission Main plan set was utilized to develop proposed alternative transmission main alignments parallel to the existing transmission. The information provided by the plan set was a ground surface profile from the Pine Ridge Reservoir to the Terminal Reservoir (Z1-R3). With an established ground surface profile, preliminary hydraulic grades lines could be determined for proposed alternatives.

Alternatives for the new transmission main were based upon the design flow rate of 13,000 gpm. Nominal pipe diameters ranging from 30-inches to 58-inches were evaluated with proposed pump station locations varying in location along the pipeline alignment. Varying

pipeline materials were also evaluated due to pressure class constraints, friction coefficients, corrosion resistance, and cost. Taking into account these variables, three primary transmission main alternatives were developed.

- **Primary Alternative 1:** This alternative utilized a new pump station at the Pine Ridge Reservoir. Water discharged from the reservoir would be immediately boosted, and water could be supplied to the Terminal reservoir without additional pumping. However, this alternative requires the highest pipe pressure classes compared to the other alternatives. This transmission main alternative would require the highest maximum pipe pressure class to accommodate pumping and static water HGL conditions.

- **Primary Alternative 2:** This alternative utilized a new pump station at the same location as the existing Donkey Creek Pump Station. Water discharged from the Pine Ridge Reservoir would be transferred to the new pump station via gravity flow through a new transmission main. The pump station was assumed to be an end-suction or in-line centrifugal booster which would not require a wet well like the existing vertical turbine pumps of the Donkey Creek Pump Station. The centrifugal booster pumps would require 20 psi (46.2 feet) of suction pressure. The pump station would discharge to new transmission main from the pump station to the Terminal Reservoir. This alternative would have the highest pumping TDH of the alternatives.

- **Primary Alternative 3:** This alternative utilized a pump station between the Terminal Reservoir and the existing Donkey Creek Pump Station. The pump station would transfer water via gravity from the Pine Ridge Reservoir in a new transmission main to the new pump station. Larger diameters would be able to transfer water via gravity closer to the Terminal Reservoir, thus reducing the required pumping TDH to deliver water to the Terminal Reservoir. As with primary alternative 2, the pump station was assumed to be an end-suction or in-line centrifugal booster without a wet well, but requiring 20 psi (46.2 feet) of suction pressure. The pump station would discharge to a new transmission main from the pump station to the Terminal Reservoir. This alternative would require the lowest maximum pipe pressure class of the other primary alternatives, and have a similar pumping TDH to that of primary alternative 1.

Of these alternatives, Alternative 1 has the highest maximum pressure class pipe and Alternative 3 has the lowest maximum pressure class pipe.

Each of these three primary alternatives were evaluated with pipe diameters ranging from 30-inches to 58-inches with pipe materials including ductile iron, steel, PVC, FRP, and HDPE. Due to the possible corrosive nature of soils in the areas along the pipeline alignment, PVC and HDPE were given primary consideration. Due to the proximity to PVC material prices, FRP was given second priority to the commonly more used PVC material. Ductile iron and steel pipe materials were used as with secondary precedence to accommodate high pressure zones along the transmission mains. PVC and HDPE are also more favorable for surge mitigation. For these reasons, PVC was used up to a cost multiplier of 1.5 measured against steel. For instance, if steel pipe was $100 per foot and corrosion protection was $50 per foot, PVC would remain the preferred alternative up to $225 per foot for the same size and pressure class of pipe. Their material properties yield substantially lower surge wave speeds than metallic pipe materials. The key variables in computing surge wave speeds are Young's Modulus and Poisons Ratio, in addition to the pipe internal diameter and wall thickness.

PVC and HDPE are both limited by the maximum diameter of certain higher pressure classes that are currently manufactured. PVC can not be obtained above pressure class 165 in 42-inch diameter. HDPE can be limited by the wall thickness of high pressure classes. HDPE pipe wall thicknesses in high pressure classes reduces the internal diameter of the pipe substantially, and in some instances necessitates up-sizing the entire transmission main to produce a similar HGL to that of PVC, steel, or ductile iron. This upsizing, combined with the material costs associated with substantially larger wall thicknesses essentially removes HDPE as an economically viable transmission main material for this alternative.

Ductile iron and steel pipeline materials were also directly compared. Costs were compared for the two materials based upon diameter and pressure class. Steel proved to be the more economical selection for all diameters and pressure classes with the exception of the 350 psi pressure class. For this reason, steel was utilized as the metallic pipeline material for managing high pressure zones in the alternative analysis.

Two ending alignments were developed for the transmission main routing through town:

- A “Northern” route that generally follows the existing Madison pipeline route to terminal reservoir Z1-R3
- A “Southern” route that skirt the southeast corner of Gillette then runs up Southern Drive to terminal reservoir Z1-R4.

To minimize modeling duplication, the lengths and profiles of these two routes were compared. Based on the preliminary routes and profiles shown in Figures 8-8 and 8-9, these routes are substantially the same. The southern route is approximately 1,093 feet shorter, bit the actual route lengths could vary by this much in final design. The unit headlosses in this pipe are small, even at peak flows, so this difference in length makes little difference in the total headloss of the line. Further, points along the profiles differ by a maximum of 40 feet in elevation, making surge and pressure class determinations very similar. The water surface elevations in each of the terminal reservoirs should be the same as they are located in the same zone unless headloss in the distribution system causes a difference in water surface elevations.

Since these alternatives are essentially the same, they have been treated the same for evaluation and surge modeling purposes. For cost estimating purposes, the longer length of the northern route has been used to provide a conservative estimate. There are a number of reasons to consider utilizing the southern route which should be thoroughly evaluated in the design phase of the project. For the purposes of this study, these alternatives are substantially similar from a transmission hydraulics perspective and will not be evaluated separately hereafter in this report.

Figure 8-8

Figure 8-9

8.2.1.3 Alternative Analysis (Cursory Cost Comparison)

As previously stated, alternatives for the new transmission main were based upon the design flow rate of 13,000 gpm. Nominal pipe diameters ranging from 30-inches to 58-inches were evaluated with proposed pump station locations described in Section 8.2.1.2. Varying pipeline materials were also evaluated due to pressure class constraints, friction coefficients, and corrosion resistance. Final transmission main diameters and materials were determined based upon a cursory cost comparison.

The three primary transmission main alternatives with the most economical transmission main diameters are described below.

- **Alternative 1:** This alternative utilized a new pump station at the Pine Ridge Reservoir and a transmission main diameter of 42-inches. Water discharged from the reservoir would be immediately boosted, with a pump discharge HGL of approximately 4,876 feet. Assuming the Pine Ridge Reservoir is operating at the low water surface elevation of 4,530 feet, the TDH required from the new pump station would be approximately 346 feet (150 p.s.i.). With a pump station located at the Pine Ridge Reservoir, water could be supplied to the Terminal Reservoir without additional pumping. However, this alternative requires the highest pipe pressure classes compared to the other alternatives. The transmission main would require a maximum pipe pressure class of 350 psi to accommodate pumping and static water HGL conditions. A HGL profile for this preliminary alternative is shown in Figure 8-10.

- **Alternative 2:** This alternative utilized a new pump station at the same location as the existing Donkey Creek Pump Station. Water discharged from the Pine Ridge Reservoir would be transferred to the new pump station via gravity flow through a 36-inch diameter transmission main. The pump station was assumed to be an end-suction or in-line centrifugal booster which would not require a wet well like the existing vertical turbine pumps of the Donkey Creek Pump Station. The centrifugal booster pumps would require 20 psi (46.2 feet) of suction pressure. The pump station would discharge to a HGL of 4,801 feet, requiring a pump station TDH of approximately 429 feet (186 p.s.i.). The transmission main from the pump station to the Terminal Reservoir would be a 42-inch diameter pipeline requiring a maximum pipe pressure class of 250 psi to

accommodate pumping and static HGL conditions. A HGL profile for this preliminary alternative is shown in Figure 8-11.

- **Alternative 3:** This alternative utilized a pump station between the Terminal Reservoir and the existing Donkey Creek Pump Station. The pump station is approximately 9 miles from Terminal Reservoir as opposed to the 15 mile distance of the existing Donkey Creek Pump Station. Water from the Pine Ridge reservoir would be transferred to the new pump station via gravity flow through a 42-inch diameter transmission main to the new pump station. As with Alternative 2, the pump station was assumed to be an end-suction or in-line centrifugal booster without a wet well, but requiring 20 psi (46.2 feet) of suction pressure. The pump station would discharge to a HGL of 4,798 feet, requiring a pump station TDH of approximately 342 feet (148 p.s.i.). The transmission main from the new pump station to the Terminal Reservoir would be a 42-inch diameter pipeline requiring a maximum pipe pressure class of 200 psi to accommodate pumping and static HGL conditions. A HGL profile for this preliminary alternative is shown in Figure 8-12.

This cost comparison evaluated pipe costs, pipe installation costs, pump station capital costs, and pump station operational costs. The operational costs are based upon a present 5.5 cents per Kilowatt hour, and a yearly inflationary increase of 1.5%. The cursory cost estimate for each of the three described alternatives is shown in Table 8-6 thru Table 8-8.

Figure 8-10

Figure 8-11

Figure 8-12

TABLE 8-6

Alternative 1 Cursory Cost

CAPITAL

Item	Total Units	Unit	Cost ($/unit)	Total Cost
42" PVC 125	28,733	LF	$127	$3,649,091
42" PVC 165	16,725	LF	$200	$3,345,000
42" Steel 200	28,481	LF	$107	$3,047,467
42" Steel 250	30,784	LF	$134	$4,125,056
42" Steel 300	46,496	LF	$161	$7,485,856
42" Steel 350	68,782	LF	$187	$12,862,234
Steel Corrosion & Cathodic Protection	174,543	LF	$40	$6,981,720
42" Excavation, Bedding, Installation, Backfill	220,001	LF	$52	$11,440,052
Pump Station (346-ft TDH)	1350	Hp	Exponent	$1,135,832
TOTAL CAPITAL COST				$54,072,218

OPERATIONAL (theoretical maximum based on continuous operation)

Item	Cost ($/KW-hour)	Cost ($/KW-day)	Cost ($/KW-month)	Cost ($/KW-year)
Power Consumption	$55.10	$3,305.78	$99,173.44	$1,206,610
Power Consumption 10-year life	$63.94	$3,836.49	$115,094.83	$1,400,320
Power Consumption 20-year life	$74.21	$4,452.41	$133,572.24	$1,625,129
Power Consumption 30-year life	$86.12	$5,167.20	$155,016.04	$1,886,029

OPERATIONAL PRESENT WORTH ANALYSIS (based on projected pumping)

Year	Average Deficit (gal) gal/yr	Pumping Hours @12,900 gpm	Cost Per Hour	Cost Per Year	Present Worth
5	145,617,350	188	$59.52	$11,197.86	$47,170
10	205,486,357	265	$63.94	$16,975.19	$53,437
15	279,392,736	361	$69.08	$24,934.18	$58,650
20	354,151,665	458	$74.21	$33,955.55	$59,688
25	438,705,984	567	$80.17	$45,437.81	$59,679
30	539,944,310	698	$86.12	$60,077.52	$58,965
TOTAL OPERATIONAL					$337,590
TOTAL CAPITAL & OPERATIONAL COST					**$54,400,000**

TABLE 8-7

Alternative 2 Cursory Cost

CAPITAL					
Item		Total Units	Unit	Cost ($/unit)	Total Cost
36" PVC 125		22,030	LF	$95.00	$2,092,850
36" PVC 165		46,707	LF	$123.00	$5,744,961
36" PVC 200		69,852	LF	$175.00	$12,224,100
42" PVC 125		28,732	LF	$127.00	$3,648,964
42" PVC 165		16,725	LF	$200.00	$3,345,000
42" Steel 200		28,480	LF	$107.00	$3,047,360
42" Steel 250		7,474	LF	$134.00	$1,001,516
Steel Corrosion & Cathodic Protection		35,954	LF	$40.00	$1,438,160
36" Excavation, Bedding, Installation, Backfill		138,589	LF	$45.49	$6,304,414
42" Excavation, Bedding, Installation, Backfill		81,411	LF	$52.09	$4,240,699
Pump Station (429-ft TDH)		1,666	Hp	Exponent	$1,250,715
TOTAL CAPITAL					$44,338,739
OPERATIONAL PRESENT WORTH ANALYSIS (based on projected pumping)					
Year	Average Deficit (gal) gal/yr	Pumping Hours @13,000 gpm	Cost Per Hour	Cost Per Year	Present Worth
5	145,617,350	188	$73.80	$13,883.50	$58,483
10	205,486,357	265	$79.28	$21,047.75	$66,257
15	279,392,736	361	$85.65	$30,915.49	$72,720
20	354,151,665	458	$92.01	$42,100.12	$74,005
25	438,705,984	567	$99.40	$56,337.44	$73,995
30	539,944,310	698	$106.78	$74,489.99	$73,111
TOTAL OPERATIONAL					**$418,570**
TOTAL CAPITAL & OPERATIONAL					**$44,800,000**

TABLE 8-8

Alternative 3 Cursory Cost

CAPITAL					
Item		Total Units	Unit	Cost ($/unit)	Total Cost
42" PVC 125		119,751	LF	$127.00	$15,208,377
42" PVC 165		47,931	LF	$200.00	$9,586,200
42" Steel 200		52,318	LF	$107.00	$5,598,026
Steel Corrosion & Cathodic Protection		52,318	LF	$40.00	$2,092,720
42" Excavation, Bedding, Installation, Backfill		220,000	LF	$52.09	$11,459,800
Pump Station (342-ft TDH)		1327.84	Hp	Exponent	$1,129,929
TOTAL CAPITAL					$45,075,052
OPERATIONAL PRESENT WORTH ANALYSIS (based on projected pumping)					
Year	Average Deficit (gal) gal/yr	Pumping Hours @13,000 gpm	Cost Per Hour	Cost Per Year	Present Worth
5	145,617,350	188	$58.83	$11,068.05	$46,623
10	205,486,357	265	$63.20	$16,778.73	$52,818
15	279,392,736	361	$68.28	$24,645.40	$57,971
20	354,151,665	458	$73.35	$33,562.05	$58,997
25	438,705,984	567	$79.24	$44,910.68	$58,987
30	539,944,310	698	$85.12	$59,379.92	$58,281
TOTAL OPERATIONAL					$333,676
TOTAL CAPITAL & OPERATIONAL					**$45,400,000**

The combined total capital and present worth of 30-year operational costs shown in the lower right hand corner of each table was the value used for comparison of the alternatives. The tables show that Alternatives 2 and 3 are the most economical. The tables also show that 30-year operational costs range from 0.6% to 0.9% of the combined total capital and 30-year operational costs.

Due to the results of cursory cost estimate, Alternatives 2 and 3 were selected for in-depth hydraulic modeling and transient analysis.

8.2.1.4 Alternative Modeling

The software used for hydraulic and transient analysis was KY Pipe's PIPE 2006 v 3.011 with the Surge analysis module. The modeling process began with importing the ground surface profile of the transmission main alignment into the model. Junctions were specified at 500-foot intervals with elevations corresponding to the ground surface profile at the same station as the existing transmission main. Pipe segments were then input to connect each junction. The unique pumps station locations, pipe diameters, and pipe materials were then specified along the transmission main alignment for each of the two alternatives.

The transmission mains and pump stations were analyzed and calibrated to flow 13,000 gpm under steady state conditions, thus reproducing the HGL's presented in Section 8.2.1.3. Figures 8-13 and 8-14 show the steady state HGL of the alternatives selected for the in-depth hydraulic and transient analysis.

Figure 8-13

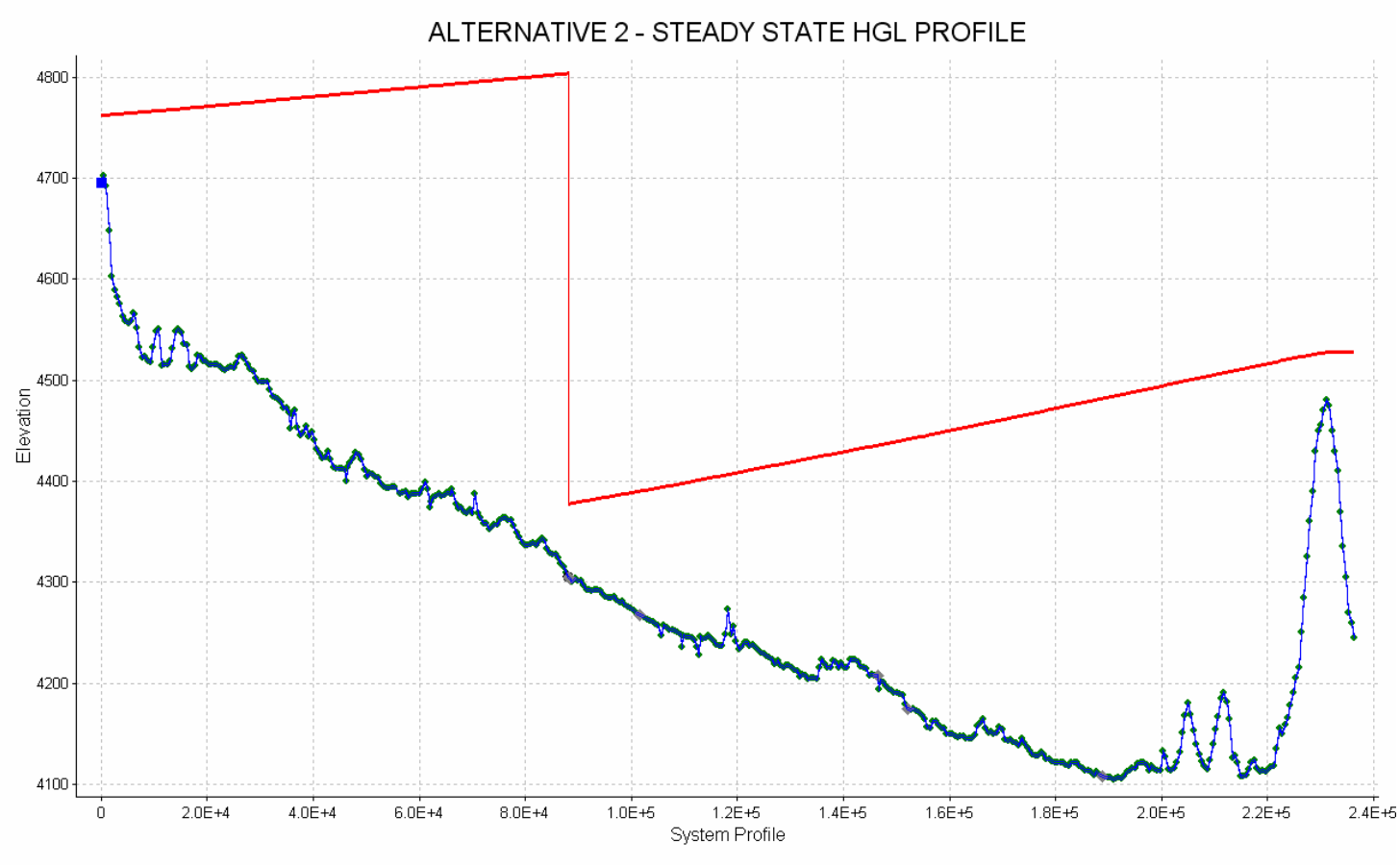
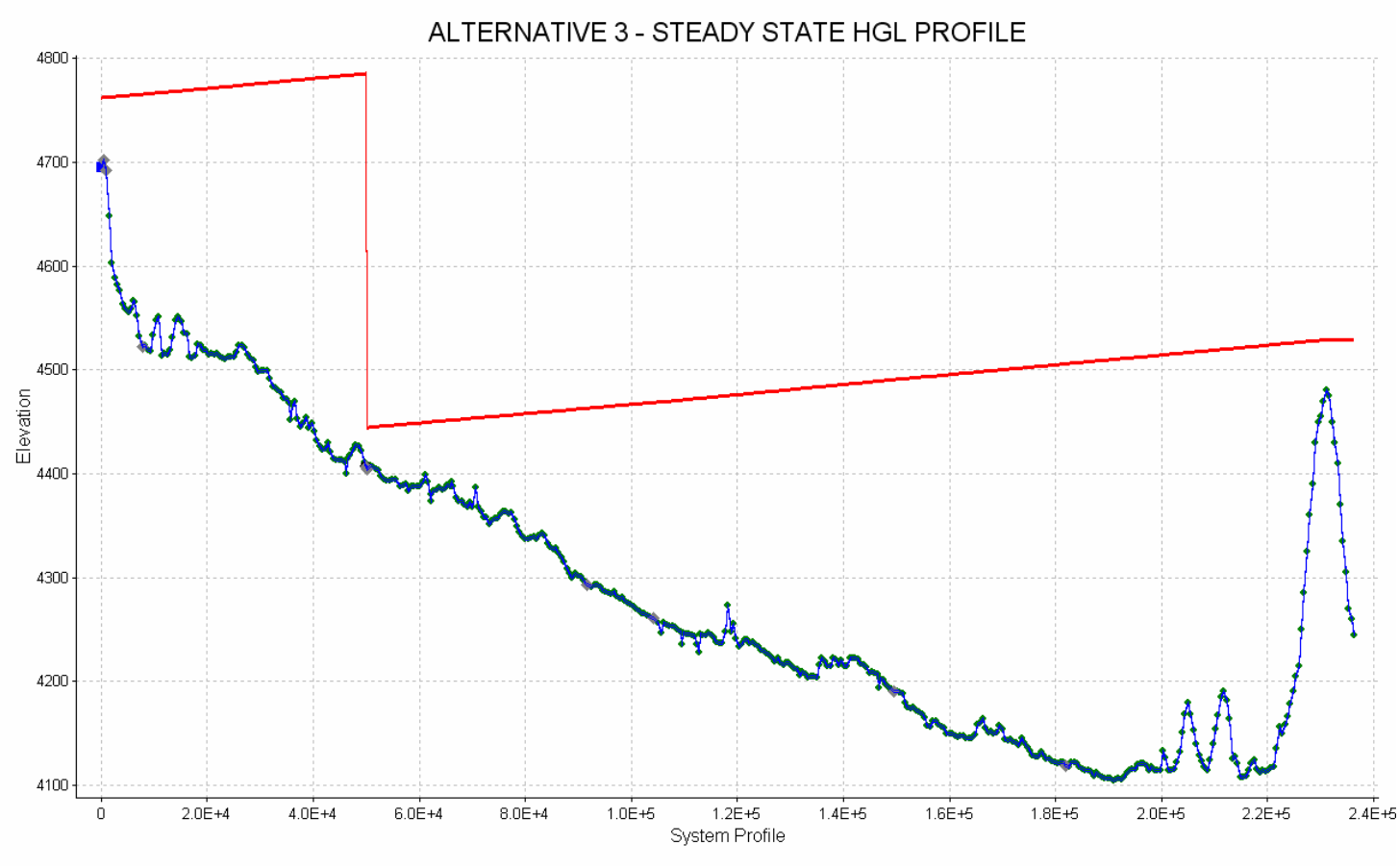

ALTERNATIVE 2 - STEADY STATE HGL PROFILE
Elevation
4800
4700
4600
4500
4400
4300
4200
4100
0
2.0E+4
4.0E+4
6.0E+4
8.0E+4
1.0E+5
1.2E+5
1.4E+5
1.6E+5
1.8E+5
2.0E+5
2.2E+5
2.4E+5
System Profile

Figure 8-14



For both alternatives, the steady state reservoir conditions are as follows:

- Pine Ridge Reservoir at low operating water surface elevation = 4530
- Terminal Reservoir at high operating water surface elevation = 4762

These conditions necessitate that the maximum pump TDH be used to supply water to the Terminal Reservoir from the Pine Ridge Reservoir. Other assumptions include:

- Steel pipe Hazen-Williams Coefficient = 130
- PVC pipe Hazen-Williams Coefficient = 150
- Minor losses assumed negligible
- Pump stations were assumed to be an end-suction or in-line centrifugal booster without a wet well, requiring 20 psi (46.2 feet) of suction pressure

Another important factor in accurately modeling the steady state performance of the transmission main is internal pipe diameter. The internal diameter was specified for each of the pipe types used in the modeled alternatives. This detail ensures more accurate resultant HGL's by specifying the actual hydraulic area of the conduits, instead of the nominal diameters. Flow area and friction losses may vary significantly between pipe pressure classes due to increasing wall thicknesses reducing internal diameters. Table 8-9 shows the nominal versus internal diameters of pipes utilized in the modeled alternatives.

8.2.1.5 Alternative Surge Analysis

The PIPE 2006 Surge module solves the basic equations of fluid mechanics for transient flow (surge or water hammer) of an incompressible fluid in a pipe network. Fluid transients are generated at any point in a flow system where a disturbance causes a flow change in the system. This change can include a valve opening or closing, a pump starting or shutting down, a change in a reservoir pressure, or a change in an inflow or outflow for the system. Pressure waves generated during these changes are the main source of damaging effects to pipe systems. These pressure waves can also be generated in both gravity and pumped systems.

The Surge program was developed to analyze transient flow in a pipe network using the "wave plan" method. Pressure waves are generated at positions in the pipe system where the pressure flow characteristics or other conditions are time dependent. For the purposes of analyzing the Surge model, the assumption that the most critical surge condition would be power failure at the pump station, causing the pumps to shut-down. This particular flow change condition differs from pump start-up, valve operation, etc. due to the fact that power failure is uncontrollable. Pump start-up and valve operation can be controlled to vary flow changes over specified time durations. Pump shut-down due to a power failure is not controllable, and has the potential to result in a near instantaneous flow change from the maximum operating pumped flow rate to no flow.

Wave speed is another critical factor for proper transmission main surge analysis. The specific wave speeds for each pipe diameter, material, and pressure class were calculated. Table 8-9 summarizes the wave speeds appropriated to the modeled pipe segments.

TABLE 8-9
Pipe Wave Speed

Pipe Material	Pipe Class (psi)	Pipe Diameter (in)	Internal Diameter (in)	Hazen-Williams Coefficient	Wave Speed (ft/sec)
Steel	200	42	43.56	130	3292
Steel	250	42	43.46	130	3383
Steel	300	42	43.36	130	3464
Steel	350	42	43.24	130	3550
PVC	125	36	35.944	150	1055
PVC	125	42	41.762	150	1055
PVC	165	36	35.236	150	1205
PVC	165	42	40.94	150	1205
PVC	200	36	34.652	150	1317

The steady state model pumps operating at 13,000 gpm were specified to stop over a 1 second interval, stopping the flow, and generating the worst case surge conditions. The pumps were allocated check valves, and the simulation was analyzed over a time duration of 250 seconds. Figures 8-15 and 8-16 show the surge wave pressure envelopes, bound by the maximum up surges and down surges, for the two modeled alternatives.

Figure 8–15

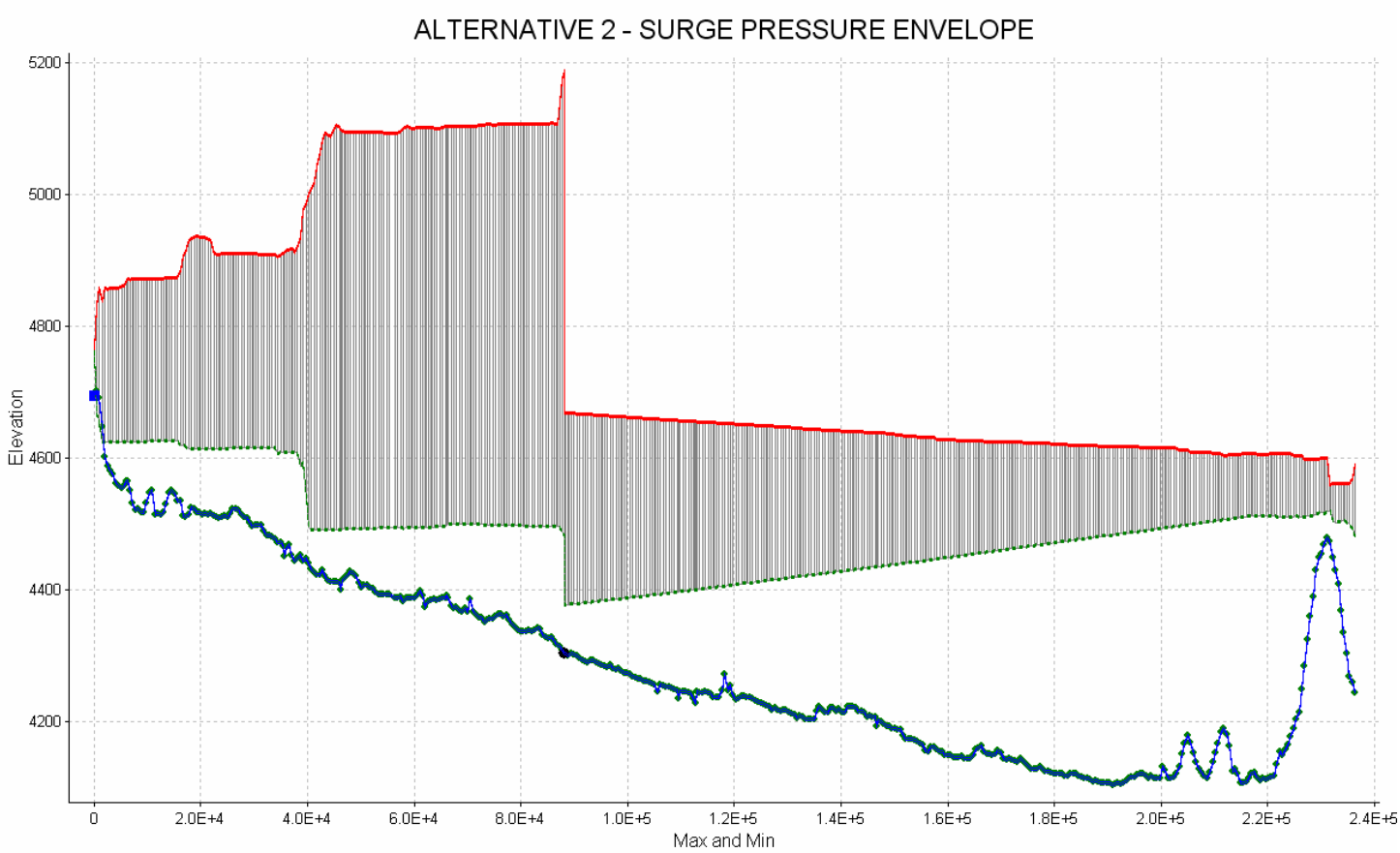


Figure 8-16

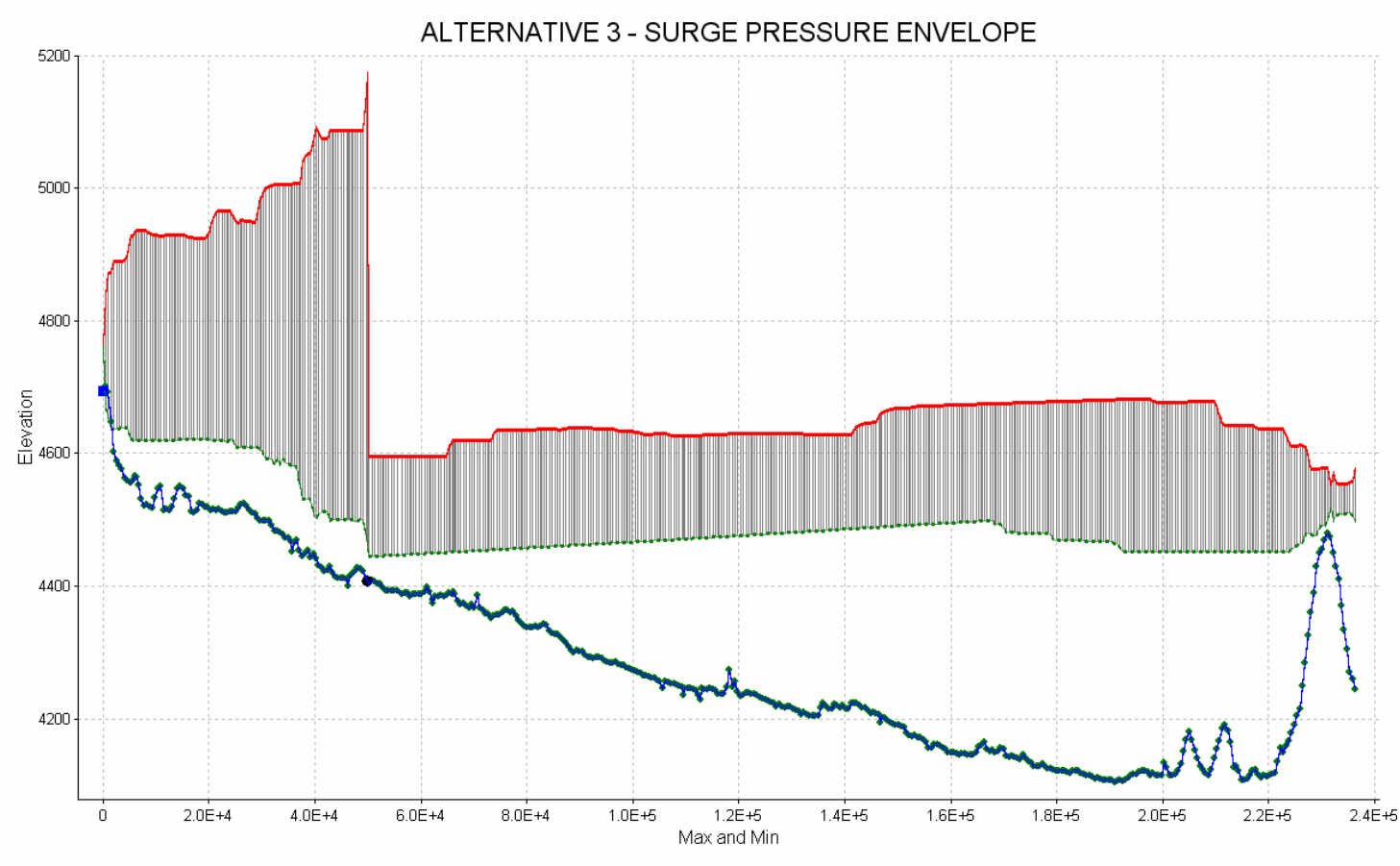


Figure 8-15 and 8-16 show that the pressure envelopes for both modeled alternatives necessitate much higher pressure class pipes to handle the surge conditions generated from pump shut-down. It is readily apparent from the figures that surge mitigation devices need to be employed to reduce the maximum upsurge boundaries.

The process of modeling surge mitigation devices is an iterative process of device selection and sizing. Each surge device will affect the transmission main surge envelope, but will also affect other surge mitigation devices along the transmission main. The surge mitigation devices evaluated for potential surge protection in the two modeled alternatives are listed below:

- **Surge Bladder Tanks:** This type is a closed surge tank with a gas-filled bladder. This tank will activate at the set pressure for the bladder and will act as a closed surge tank for pressures above the set pressure. For pressures below the set pressure, the tank has no effect.

- **Surge/Pressure Relief Valves:** A pressure relief valve is one which opens at a designated pressure and closes at a second (lower) pressure or head.

- **Combination Air Release and Vacuum Valves:** This device takes air in when the pipeline pressure drops below atmospheric and releases air when the pipeline pressure exceeds atmospheric.

- **Surge Anticipation Valves:** A surge anticipation valve is a device normally located at a pump discharge which opens on a downsurge at a specified pressure to provide protection for a subsequent upsurge. Once activated this device works on a timed cycle.

- **Closed Surge Tanks:** A closed surge tank has a volume of gas, usually air, in the space above the liquid. The gas expands and compresses as flow leaves and enters the tank.

Modeling results indicate that the portion of transmission main from the proposed pumps stations to the Terminal Reservoir were most effectively protected against high surge pressures by bladder surge tanks. Located within the pump stations, the tanks were the most effective at limiting high transient pressure events and any subsequent “secondary” surge events delivery pressure waves back to the pump stations. The bladder surge tanks were sited on the discharge side of the pumps. Due to the relatively high flow rate of 13,000 gpm, the surge bladder tanks required large volumes, with initial bladder pressures of approximately 60% of the normal operating pressure.

Combination air release and vacuum valves were also employed in the model near the Terminal Reservoir to remove negative pressures that occurred from downsurge as the transmission main gains elevation before discharging into the reservoir. Air release and vacuum valves as well as blow off hydrants are standard in the final design of transmission mains. Blow off hydrants are typically located at all low elevation points along the pipeline alignment to faciilitate flushing and draining. It is also standard to site air release and vacuum valves at all high points along the pipeline alignment. The model did not incorporate combination air release and vacuum valves at the high points as doing so complicates the surge model. These elements do

little to alleviate surge, but are an important part of pipeline operations and should be included in the design. Only valve assemblies specifically required to mitigate negative pressure due to down surge were included in the model. Blow off hydrants were also not included in the modeled transmission main alternatives.

The portion of transmission main from the Pine Ridge Reservoir to the new pump stations was most effectively protected against surge waves by surge/pressure relief valves. These valves were placed along the transmission main at specific locations to reduce the upsurge that occurred during a pump station shut-down scenario. As the upsurge travels back to the Pine Ridge Reservoir, the surge/pressure relief valves reduce the maximum pressures at key locations dependent upon ground surface and transmission main material or class transitions. Inclusion of these valves along this portion of the transmission main reduces the pipeline lengths that would have otherwise been increased in pressure class to handle the maximum pressures due to upsurge.

Since complete surge pressure wave removal from a transmission main is not always possible or economical, both alternatives required that portions of higher pressure class pipe be extended beyond the lengths initially stipulated to handle static and normal pumping operation conditions. Table 8-10 quantifies the pipe lengths by pressure class and material, revised pump station required TDH, and the proposed surge devices specific to each of the modeled alternatives.

TABLE 8-10
Alternative 2 Modeled Quantities

Item	Total Units	Unit
36" PVC 125	21,130	LF
36" PVC 165	49,500	LF
36" PVC 200	66,500	LF
42" PVC 125	15,900	LF
42" PVC 165	22,000	LF
42" Steel 200	29,500	LF
42" Steel 250	16,900	LF
Pump Station (430-ft TDH)	2,428	Hp
10,000-gal Surge Bladder Tank	3	EA
Pressure Reducing Station (36" main line)	5	EA

Alternative 3 Modeled Quantities

Item	Total Units	Unit
42" PVC 125	79,062	LF
42" PVC 165	63,000	LF
42" Steel 200	79,368	LF
Pump Station (345-ft TDH)	1,948	Hp
10,000-gal Surge Bladder Tank	2.5	EA
Pressure Reducing Station (42" main line)	5	EA

8.2.1.6 Alternative Cost Comparison and Recommendation

The final step towards recommending a transmission main alternative was to complete a detailed cost estimate. Both alternatives were evaluated similarly to the cursory cost estimates shown in Section 8.2.1.3, with the addition of major appurtenant items required for a complete and functional transmission main.

Cost Estimates in Appendix A provide an itemized cost break-down for the two alternatives. The cost estimates again track capital and operational costs. The operational costs were again computed based upon a present 5.5 cents per Kilowatt hour, and a yearly inflationary increase of 1.5%.

As demonstrated by these cost estimates, Alternative 2 is the more economical alternative by a margin of approximately $5.7 million dollars (3.8%). Alternative 2 requires a pump station TDH approximately 85 feet more than Alternative 3, but this higher TDH yields a present worth 30-year power consumption operational cost difference of approximately $136,000, or 2.4% of the capital cost difference between the alternatives.

The major difference making Alternative 2 the most economical selection is the smaller diameter pipeline from the Pine Ridge Reservoir to the Pump Station. Alternative 2 utilizes a 36-inch diameter transmission main for this portion, while Alternative 3 employs a 42-inch diameter transmission main the entire distance from the Pine Ridge Reservoir to the Terminal Reservoir.

Based upon this cost estimate, it is recommended that the proposed transmission main Alternative 2 be selected to supply the design requirement of 13,000 gpm from the Madison Well Field to the Terminal Reservoir. A final hydraulic and transient analysis should be

conducted to ensure that all surge conditions are thoroughly evaluated, and any elevation discrepancies in the proposed parallel alignment to the existing transmission main are corrected.

8.2.1.7 Madison Well Field

The source for this alternative will be a new Madison well field. The specifics of the Madison aquifer are discussed at great length in Chapter 5 of this report. The new Madison field will generally abut the existing Madison field to the east-northeast. Ten wells are planned with assumed yields of 1,430 gpm each providing firma capacity for 12,879 gpm. This yield has been achieved in existing Madison wells M-9 and M-10. The wells in the existing Madison field that are completed in caverns make this yield with minimal drawdown. Wells outside caverns that have been hydraulically fractured, presumably into these caverns also produce large yields. It is assumed that any wells in the new Madison field that do not hit caverns will be hydraulically fractured to improve their yield. A layout of the proposed new Madison well field is included as Figure 8-17. Table 8-11 summarizes the known data pertaining to this proposed well field and proposed well sites.

8.2.1.8 Total Alternative Costs

Cost of New Madison Well Field and Transmission Main: $159,393,000.

Table 8-11 Proposed Madison Well Field Data

8-17 New Madison Well Field Layout

8.2.2 Surface Water Impoundment of Madison Water with a Surface Water Treatment Plant and New Transmission Main

Large water distribution systems typically make up peak daily demands by depleting storage rather than adding more capacity at the source. Table 8-12 shows the annual volumes of water needed to make up the demand in excess of well yield and transmission line capacity at Gillette for a 30-year planning period. The annual deficit volumes on Table 8-12 are equivalent to the storage capacity that must be provided to meet the peak demands. The annual deficit pumping duration is the number of days per year it is necessary to use storage to augment the well field and transmission line capacities.

Table 8-12
Annual Storage Requirements to Meet Peak Demands

Year	Annual Deficit or Storage Requirement (gallons)	Annual Deficit Pumping Duration (days)
5	145,617,350	74
10	205,486,357	82
15	279,392,736	92
20	354,151,665	96
25	438,705,984	102
30	539,944,310	111

This alternative makes use of the existing Madison pipeline capacity during off-peak times to fill a surface reservoir where water is stored until peak flows exceed existing system capacities. If the issues surrounding its use can be worked out satisfactorily, CBM water could partially or completely replace the Madison water as a source. Once groundwater is brought to a surface water impoundment, it is considered surface water from a regulatory treatment perspective. Thus, the water is pulled from the reservoir storage, treated, and pumped into Gillette via a new transmission main generally parallel to the existing Madison line. A schematic representation of this alternative is shown in Figure 8-18.

8.2.2.1 Reservoir Alternatives

Two alternatives were evaluated as possible reservoir construction scenarios. The first alternative considered is to build a dam to create a reservoir within an existing draw east of Gillette. The second alternative considered is to excavate and create a reservoir on an existing "flat" area, also east of Gillette.

8.2.2.1.1 Required Storage

The amount of storage the proposed reservoir would need to provide was determined by comparing future population and demand estimates for the 30 year design period to current supply capacities. The current system can supply approximately 12 MGD, firm capacity. At year 2037, peak demand is estimated to be 30.6 MGD, 18 MGD more than the current system can supply. Average demand at year 2037 is estimated at about 9.5 MGD. This large peak demand made reservoir storage one option to supply peak flows.

Gallons of demand per capita, per day (gpcd) were determined on a monthly basis from recent water demand data from the City of Gillette. The projected population at year 30 is 50,018. At this population, from June through September the current system would not be able to supply the demand. The total deficit at year 30 for months June through September is 539.9 million gallons, or about 1657 ac-ft. Including dead storage of 10%, and a factor of safety of 30%, the reservoir will need to provide approximately 2400 ac-ft of storage.

Figure 8-19 illustrates the average monthly consumption in comparison to the amount the system is able to deliver per person at 5 year increments through the design period of 30 years.

Figure 8-18

Figure 8-19

8.2.2.1.2 Site Selection

For a dammed reservoir, several possible sites were identified along the current Madison pipeline alignment between 7 and 11 miles to the east of the City of Gillette. Many variables would go into choosing an ideal reservoir site if this alternative is selected including ownership, topography, soil conditions, spatial relation to other improvements, access and suitability for dam construction. The sites were not investigated to this level of detail for the conceptual analysis herein as such detailed work is outside the scope of this report. What appears to be a reasonable option was chosen based on maximizing the ratio of volume to surface area in order to minimize losses of water to evaporation and infiltration. The reservoir site used in the analyses in this section is RS4, as shown on Figure 8-20.

Table 8-13 outlines the physical characteristics of the reservoir sites that were considered.

TABLE 8-13
Reservoir Site Characteristics

Proposed Reservoir Site	HWL, ft	Storage Capacity, ac-ft	Approx dam length, mi	Approx max dam height, ft	Vol/Surface Area ratio
RS1	4500	2898	0.6	90	37
RS2	4420	3200	0.5	50	17
RS3	4500	1145	0.2	70	30
RS4	4440	2632	0.4	70	37
RS5	4450	2851	0.3	80	36
RS6	4480	3309	0.3	70	31

Jim Williams, Vice President of Wyodak Resources was contacted regarding possibly using their pit as a reservoir. He indicated they had no plans to sell the property at the present time, but a sale may be possible in the spirit of being a good neighbor to Gillette. The rough dimensions of the pit that he gave indicate it is in the general size range needed. However, he noted that it has long been used to dispose of power plant ash. Coal-fired power plant ash has a tendency to contain high levels of heavy metals and toxic substances, including elevated levels of lead, mercury, arsenic, chromium, zinc, and nickel. This site has not been specifically tested for any of these parameters in conjunction with this project. While the site could be lined,

Figure 8-20

the risk inherent to utilizing such a site for storing drinking water are numerous and severe. Accordingly, this alternative is not evaluated further in this study.

8.2.2.1.3 Reservoir Construction Costs

This section discusses factors that will contribute to the cost of constructing a reservoir including: land purchase, reclamation, reservoir outlet structure, excavation/damming, and the materials and installation cost for a liner. Both reservoir options considered here would be lined to minimize infiltration losses since the supply water for the reservoir would involve pumping costs.

For either reservoir alternative, land purchase would be required. Several realtors in the Gillette area were contacted and land prices were determined to be approximately $2500/acre. No sites that would be viable reservoir locations are currently for sale, but it is assumed that if need be, the City of Gillette could enter into an agreement with the appropriate landowners to obtain the needed property. The actual land purchase price would vary with location and changes in the real estate market.

Site reclamation costs after construction were estimated at $50.00/thousand square feet.

Alternative 1 - Dammed reservoir construction

For reservoir site RS4, the approximate area is 0.111 square miles, or 71 acres with an estimated land purchase price of $177,500 if the price were $2500/acre. However, due to the fact that no usable sites are presently for sale, it is assumed that the actual purchase price could be up to 25% - 40% above this level yielding an estimated price of $248,500.

It is assumed that no significant excavation would be conducted at this site, as a natural swale exists that could be dammed off to create the reservoir.

Several different dam types were considered, and the following Table 8-14 shows a summary of dam costs for this site. It should be noted that these costs are for raw dam construction only and do not include markup, profit, cost escalations, or costs for their design/engineering. *They are to be used for comparative purposes only and do not represent final costs of any alternative*.

These costs were developed from past MMI projects with factors incorporated to adjust for dam length and fill depth.

TABLE 8-14
Summary of Dam Costs

Dam Type	Roller Compacted Concrete	Earthfill	Zoned earthfill/rockfill	Concrete faced rockfill
Cost	$36,400,000	$4,600,000	$ 5,000,000	$ 5,900,000

As the water for the reservoir is to be pumped from wells rather than from natural drainage, it is recommended that the reservoir be lined to minimize infiltration losses which would lead to costly additional pumping. Liner costs were determined for several different types of liners including a compacted clay liner (CCL), High Density Polyethylene (HDPE), and a Geosynthetic Clay Liner (GCL). For a Compacted Clay Liner, clay may be purchased in two states: semi-crude (field dried) or refined. The semi-crude is more affordable, but will require heavy mixing with the soil in the field to ensure even distribution before installation and compaction. The refined comes as small pellets and mixes more easily with the soil, though it is more expensive. The CCL is the most cost effective liner for the reservoir applications considered in this alternative.

A compacted clay liner was estimated to cost $2,057,000 for a site with this approximate geometry. This liner is assumed to be comprised of 2 lifts, or layers, each 1 foot thick, with a composition of 8% bentonite clay mixed with soil. Soil tests would need to be conducted to determine the appropriate clay ratio if this option is chosen. Using the above parameters, the liner material would total approximately 122,000 cubic yards of material and approximately 9,775 cubic yards of clay per 1 foot layer. It was assumed that installation and compaction would cost approximately $8.50 per cubic yard of liner material. Further investigation may reveal a clay source of suitable parameters near the final project area which could be used as a borrow source in lieu of the proposed mixing.

The total costs including land purchase, dam construction, and lining are summarized in the cost estimates in Appendix A.

Alternative 2 - Excavated reservoir construction:

The total construction costs of an excavated site will vary with the geometry of the excavation as far as depth and area. Two options were analyzed: 2a is a theoretical site with a depth of 45 feet and a surface area of about 75 acres; 2b is a theoretical site, 10 feet deep and a surface area of 250 acres.

Excavation costs were determined by contacting a local Gillette earthwork contractor who quoted earthwork costs to be approximately $3/cubic yard for onsite cut and fill. Geometry for both sites considered a square area that would contain the water by both excavation and building of an embankment with the excavated material. The crest width was approximated at 10 feet wide, with 4:1 slopes inside the reservoir and 3:1 slopes on the outside.

Similar to the reservoir-dam alternative, a CCL liner using the semi-crude clay material was chosen as the most reasonable option due its relative cost compared to the other liner options. However, due to the smaller depth of these sites, only one lift would be required, rather than two lifts for the dammed reservoir liner.

A cost comparison of these two options is presented in Table 8-15.

TABLE 8-15
Cost Summary for 2 Excavated Reservoir Options

	Top Area, (ac)	Depth (ft)	Earthwork	Liner - CCL	Land Price	Restoration	Total
Option 2a	75	45	$6,145,000	$1,028,500	$262,500	$117,000	$7,553,000
Option 2b	250	10	$4,032,000	$3,300,000	$875,000	$327,600	$8,534,600

It should be noted that these costs are for raw embankment construction only and do not include markup, profit, cost escalations, or costs for their design/engineering. *They are to be used for comparative purposes only and do not represent final costs of any alternative*. It can be seen from the comparative analysis above that Option 2b is not economically feasible and will not be considered further.

8.2.2.2 Pumping Costs

All pumping costs were determined as Net Present Value of the total cost over the 30 year design life of the system. Based on information given by the City of Gillette, an interest rate of 6% was used for the Net Present Value calculations, with an electricity cost of $0.055/kW-hr and a 1.5% inflation rate on electricity costs. Total efficiency of the pumping was assumed to be 64%. All pumping costs also used a Hazen-Williams C value of 130 for the pipelines, as this was reasonable given C value tests run by the City of Gillette.

Total Increase in Volume to be Pumped

Demand deficits were determined graphically at five year increments, comparing monthly projected demand at these years to the capacity of the current system. From this comparison, the yearly deficit amount that would need to be made up by the reservoir was determined. The yearly deficits are summarized in Table 8-16 below.

TABLE 8-16
Average Deficit in Supply at 5-Year Increments

Year	Pop.	Avg Demand, gpd	Peak demand, gpd	Flow Avail., Current Sys (gpd)	Flow Avail., Current Sys (gpcd)	Ann. Deficit From Avg Consumption Figure (gpc)	Average Deficit, gal/yr
5	38,050	7,236,593	23,312,604	12,065,760	317	3,827.00	145,617,350
10	40,189	7,643,402	24,623,134	12,065,760	300	5,113.00	205,486,357
15	42,448	8,073,033	26,007,186	12,065,760	284	6,582.00	279,392,736
20	44,835	8,527,008	27,469,661	12,065,760	269	7,899.00	354,151,665
25	47,356	9,006,468	29,014,236	12,065,760	255	9,264.00	438,705,984
30	50,018	9,512,744	30,645,199	12,065,760	241	10,795.00	539,944,310

Water Balance

Other factors that will affect the necessary pumping include evaporation from the reservoir, infiltration of water into the underlying soil, and the volume of water that will enter the reservoir from precipitation and runoff.

A water balance to determine the net effect of these factors was performed. Infiltration was assumed to be negligible, since the site would be lined. Evaporation was determined using the pan evaporation from a pan station, Gillette 9 ESE, with a period of record from 1925-2005. The yearly evaporation loss from the reservoir was determined to be 73,313,000 gallons.

This loss is partly offset by two factors that will contribute water to the reservoir: direct precipitation and runoff from the contributing drainage area. The average annual precipitation in the Gillette area is 17 inches. Over the 0.111 square mile surface area of the reservoir, the direct precipitation will be approximately 32,796,000 gallons/year.

The contributing drainage area to this reservoir site is 0.137 square miles, and will have an estimated runoff of 4,065,000 gallons/year.

Combined, the evaporation, direct precipitation, and runoff have a net loss of about 36,451,800 gallons per year which will need to be made up for by additional pumping. The total yearly increase over current volumes to be pumped is shown in Table 8-17 below.

Where a pipeline and pumps are already in place, the incremental increase to pump this additional volume in addition to current flows was determined. In cases such as pumping to fill the reservoir, and pumping from the proposed treatment plant through a new pipeline and into town, the cost to pump only the new flow was determined.

TABLE 8-17
Additional Required Volume in Gallons at 5-Year Increments

Year	Total demand deficit	Yearly net loss	Total additional volume
5	145,617,350	36,451,778	182,069,128
10	205,486,357	36,451,778	241,938,135
15	279,392,736	36,451,778	315,844,514
20	354,151,665	36,451,778	390,603,443
25	438,705,984	36,451,778	475,157,762
30	539,944,310	36,451,778	576,396,088

8.2.2.3 Surface Water Impoundment Treatment

Once the water is stored in an uncovered surface water impoundment, the water would no longer be classified as “groundwater” and would be considered “surface water”. The classification of “surface water” would dictate that the water would become subject to the regulatory requirements of the U.S. Environmental Protection Agency’s (EPA) Surface Water Treatment Rule. To comply with the Surface Water Treatment Rule, the City would be required to install filtration and redundant disinfection systems for the surface water impoundment supply.

A conceptual layout has been prepared for the City of Gillette’s new Water Treatment Plant (WTP) with an ultimate plant capacity of approximately 18 million gallons per day (MGD). Layout drawings provided herein are for a WTP capacity of 9 MGD with the intent that the plant would be constructed in two phases and a mirror expansion of the process train would be implemented when the City reaches a demand that approaches 9 MGD.

8.2.2.3.1 Treatment Goals and General Design Considerations

The treatment plant should be able to meet all state and national water quality standards. The following standards set by the EPA and the State of Wyoming Department of Water Quality (WYDEQ) should be considered.

The following are general design considerations (Section 8 of Chapter 12 Regulations) per Wyoming Department of Water Quality for surface water supplies:

- Chemical addition/coagulation, flocculation, sedimentation, filtration, and disinfection; or
- Where the raw water maximum turbidity is less than 50 NTU and is not attributable to clay and maximum color is less than 30 NTU, treatment facilities may include slow sand filtration and disinfection; or
- Where the maximum monthly average raw water turbidity is less than 25 NTU, the color is less than 30 NTU and fecal coliform organisms are less than 100 mpn/100 mL, treatment facilities may be diatomaceous earth filters and disinfection.

The following Table 8-18 summarizes the existing and proposed federal regulations that apply to the new WTP.

TABLE 8-18
Regulatory Summary

Source	SWTR(1)	IE SWTR(1)	LT2 ESWTR(1)	FBRR(1)	D/DBPR(1)	GWR(1)
Surface Water Impoundment of Madison Supply	X	X	X	X	X	

Note: (1) where SWTR = Surface Water Treatment Rule
IE SWTR = Interim Enhanced Surface Water Treatment Rule
LT2 ESWTR = Long Term 2 Enhanced Surface Water Treatment Rule
FBRR = Filter Backwash Rule
D/DBPR = Disinfection/Disinfectant Byproduct Rule
GWR = Groundwater Rule

A comprehensive description of these above mentioned regulations is included in Appendix B of the report.

8.2.2.3.2 New Water Treatment Plant

The new 9 MGD WTP will include the following:

- Process Building
- Clearwell and High Service Pump Station

To achieve 18 MGD, a mirrored expansion of the 9 MGD plant will be completed when required.

8.2.2.3.2.1 Process Building

The plant process building will be constructed in two phases: Phase I- 9 MGD and Phase II – additional 9 MGD for a total capacity of 18 MGD.

The process building consists of two different layout options (Alternative A and Alternative B) depending on the selected treatment filtration process (membranes or conventional filtration) chosen by the City. A schematic of Alternative A is shown in Figure 8-21 and Alternative B in Figure 8-22. Figures 8-23 and 8-24 show a conceptual layout of Alternative A (upper level and lower level plan) of the new 9 MGD WTP which consists of two trains, each with a rapid mix basin, 4-stage flocculation basin, plate sedimentation, followed by submerged microfiltration (MF) membranes. At ultimate capacity, a mirror expansion will be utilized to provide a total capacity of 18 MGD. Figures 8-25 and 8-26 show a conceptual layout of Alternative A (upper level and lower level plan) of the new 18 MGD WTP which consists of four trains, each with a rapid mix basin, 4-stage flocculation basin, plate sedimentation, followed by submerged microfiltration (MF) membranes. In addition, the building contains chemical storage, operator control room, laboratory, restrooms, electrical/mechanical shop, kitchen/break room and locker room. Alternative B as shown in Figures 8-27 and Figure 8-28 provides a conceptual layout (upper level and lower level plan) for a second option that utilizes two trains, each with a rapid mix basin, 4-stage flocculation basin, plate sedimentation, and conventional filtration. At ultimate capacity, a mirror expansion will be utilized to provide a total capacity of 18 MGD. Figures 8-29 and 8-30 show a conceptual layout of Alternative B (upper level and lower level

plan) of the new 18 MGD WTP which consists of four trains, each with a rapid mix basin, 4-stage flocculation basin, plate sedimentation, followed by conventional filtration.

The new WTP will include the following major processes:

- Raw Water Conveyance
- Coagulant Induction System (coagulant injection at rapid mix basin)
- Flocculation
- Plate Settlers (sedimentation)
- Microfiltration (MF) Membranes or Conventional Filtration
- Chemical Feed
- Clearwell and High Service Pump Station

Figure 8-21

Figure 8-22

Figure 8-23

Figure 8-24

Figure 8-25

Figure 8-26

Figure 8-27

Figure 8-28

Figure 8-29

Figure 8-30

Raw Water Conveyance:

Conceptually, there are two main raw water conveyance options for the new City of Gillette WTP. The first option is to pump directly from the Madison formation to the head of the WTP or flow via gravity from a new reservoir to the WTP.

Coagulant Induction System (Rapid Mix):

A submerged Coagulation Induction Unit (CIU) will be installed to provide a flash mix of the coagulant in the rapid mix basin. The CIU also provides final complete mixing of chemicals that could be fed prior to the flocculators in the future.

Flocculation Basins:

The purpose of the flocculation basin is to provide time for low intensity mixing to allow the destabilized particulate matter (the coagulation particles) in the water to collide and agglomerate, forming larger settleable particles ("floc"). This step is best accomplished by providing multiple intensity mixing zones with positive separation. To accomplish this, four mixing zones are provided, each with their own mixing mechanism. The tapered flocculation, or reduced mixing intensity provides a large amount of initial particle contact when the particles are small and shear is not a problem. As the particles grow, the mixing intensity is lower, less collisions are required because floc is larger and the lower mixing intensity allows the large shear-sensitive floc to remain intact.

Horizontal paddle wheel flocculators are shown in Figures 8-23, 8-25, 8-27, and 8-29. The flocculation units will include electric variable speed drives with one drive per mixing zone for a total of four per train. Drive motors shall be variable frequency and flow through the basin will be parallel to the flocculator horizontal shaft and pass through wall openings at the end of each basin. The flocculator drive motors and gear reducers will be located on the operating floor on a pad cantilevered over the top of the basin wall. The units will be chain-driven with underwater bearings. One paddle wheel will be located in each flocculation zone. The horizontal shaft paddle wheels will be used to provide well-distributed tapered flocculation across the cross-section of the basins.

Plate Sedimentation:

The purpose of the sedimentation basin is to separate the flocculated solids via gravity settling from the process water. Sedimentation will be enhanced by installing inclined stainless steel plate settlers in the basin. The treated water will pass over weirs supported by the plate settler assembly and be directed to either the submerged membrane basins or conventional filtration, depending on whether Alternative A or B is used. The settled solids will be removed via a cable-driven or pneumatic solids collection system installed at the bottom of the basin. An air header/sparger will be included to assist in cleaning the plate settlers located above. Plates will be fabricated of stainless steel, spaced a minimum of 2-inches apart, at an angle of inclination of 55 degrees. Plate assembly will be supported on concrete beams or stainless steel I-beams located where necessary. Adequate clearance space will be provided below for the vacuum solids system and for cleaning.

For filtration there are two alternatives, Alternative A which considers MF membranes and Alternative B which considers conventional filtration.

Alternative A-Microfiltration (MF) Membranes:

Under this alternative, MF membranes will be utilized to provide filtration of the settled water and provide an additional barrier to microorganisms. The layout for the MF membranes is shown in Figure 8-23, 8-24, 8-25, and 8-26. MF membranes will be installed in concrete basins, directly downstream of the sedimentation basins. Each MF train will consist of the following equipment:

- A permeate pump to transfer the filtered water (filtrate).
- Connections to an air system for scouring of the membranes.
- Connections to a membrane backpulse system for reverse flow cleaning of the membranes.
- Connections to the clean-in-place system for chemical cleaning procedures.
- Valves for isolation.
- Filtrate turbidity meter, particle counter, and instrumentation to monitor performance and perform the pressure decay integrity test.

- A reject water system to remove rejected water from the process tank on a continuous basis.

Alternative B- Conventional Filtration:

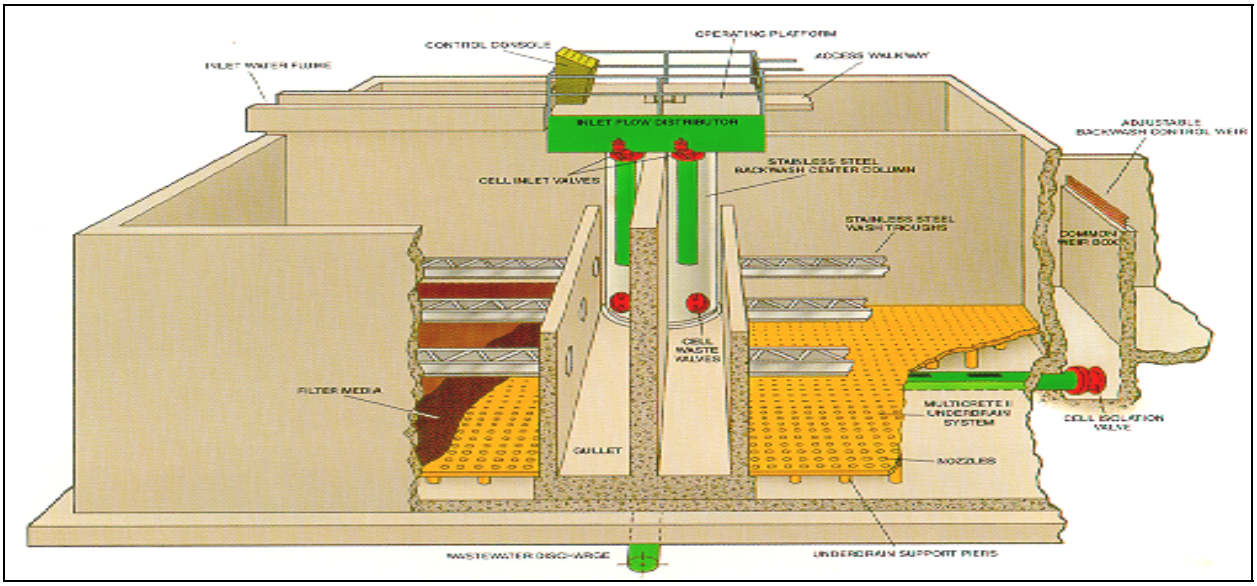
Under this alternative, conventional filtration will be utilized to provide filtration of the settled water. The layout for conventional filtration is shown in Figures 8-27, 8-28, 8-29, and 8-30. Conventional filters will be installed in concrete basins, directly downstream of the sedimentation basins.

General

The filters remove small suspended floc from the sedimentation basin effluent. Flow enters the filters through a flume from the sedimentation basins and is discharged to an effluent channel, over a weir, and then to the clearwell. Backwash wastewater drains by gravity to the backwash ponds where it will be returned to the head of the treatment plant and recovered. The filters are designed in a cluster of four cells with each cell designed to provide a total of approximately 2.25 MGD filtering capacity for a total of 9 MGD. The filter media has a total depth of 30 inches consisting of anthracite and sand.

Figure 8-31

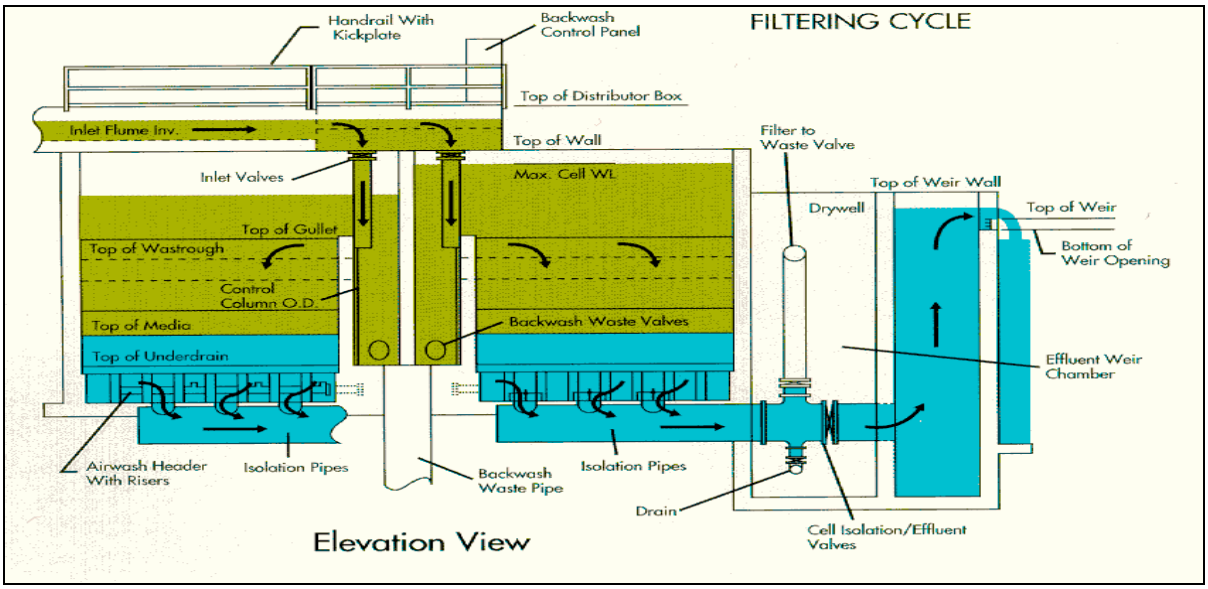
Filter Arrangement (Courtesy of EIMCO)
Filter Operation



The water from the sedimentation basins flows to the filter flow distribution box that divides water equally to each of the four filter cells by equal length weirs. The water then flows into each filter cell gullet through valved inlet lines (as shown in the figure), passes through the filter media and filter underdrains, and is collected in an effluent chamber below. Water from the effluent chamber flows through an individual pipe for each cell, over the effluent weir, into a small wetwell, and by gravity to the existing clearwells.

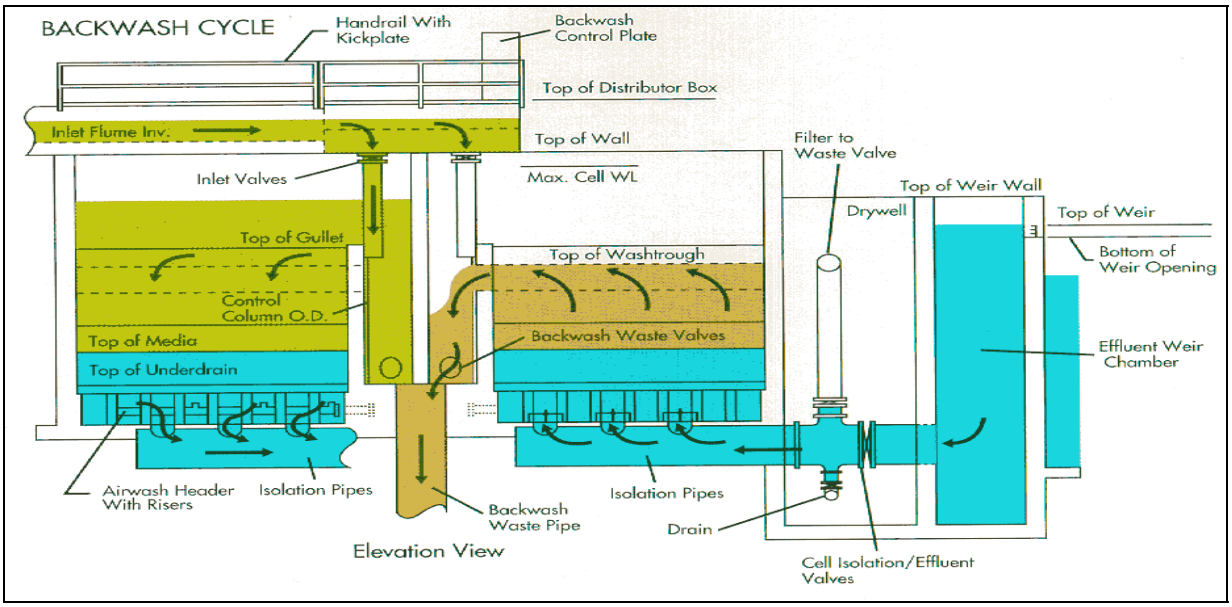
Figure 8–32

Filter Operation (Courtesy of EIMCO)
Filter Backwash



The backwash system uses combined air and water for cleaning the filter media as shown in the figure. The system is designed to allow flow from the filter cells which are on-line to provide water for the cell being backwashed. Excess water from the filters cells which are on-line not being used for backwash flows over the effluent weir into the clearwell. The clearwell would be used as the source for backwash water.

Figure 8-33
Filter Backwash (courtesy of EIMCO)



Each conventional filtration train will consist of the following equipment:

- Two backwash pumps (one in operation, one standby).
- Connections to an air system for scouring of the filters.
- Valves for isolation.
- Filtrate turbidity meter and particle counter to monitor performance of the filters.
- A backwash waste water system to remove wash water from the filters on a continuous basis.

<u>Chemical Feed:</u>

Chemicals will be used throughout the plant as part of the treatment process as well as for membrane cleaning operations (if Alternative A is used).

The following chemicals that would be recommended to enhance the treatment processes are:

- Potassium permanganate – May be added to the raw water for iron and manganese oxidation.
- Powdered activated carbon (PAC) – May be added to the raw water for taste and odor control and organics reduction.
- Chlorine – Added to the finished water for disinfection and part of the membrane cleaning process.
- Alum/Ferric chloride – May be added to the raw water for coagulation.
- Sodium hydroxide – May be added to the raw water and/or the finished water for pH adjustment and for neutralization of membrane acid cleaning solutions.
- Sodium bisulfite – May be added to membrane cleaning solution for dechlorination.
- Phosphoric acid or citric acid– May be added to the raw water for pH adjustment and part of the membrane cleaning process.
- Zinc orthophosphoric acid – May be added to the finished water as a corrosion inhibitor.

8.2.2.3.2.2 Clearwell

A 2 million gallon (MG) below grade, cast-in-place, baffled, concrete clearwell is proposed for the new 18 MGD WTP. The clearwell can be built in two phases, the first phase at 1.0 MG and the second phase at 1.0 MG for a total capacity of 2.0 MG. It is usually more cost effective to build the entire clearwell at one time, so costs for the clearwell are included in the 9 MGD plant. The 2-MG clearwell will have dimensions estimated at 140-feet long by 90–feet wide and 29-feet deep. These dimensions include a 2-foot thick floor slab, 1.75-foot thick roof slab, and 25-foot high walls at a thickness of 2-feet. The high water level depth will be at 23-feet, which will provide approximately a total volume of 2,167,993 gallons. If 2-feet of dead space is included at the bottom of the tank, which is typically included, the active storage volume will be approximately 1,979,472 gallons. Storage assumed for CT is 1.0 MG. The 25-foot wall height was chosen based on ease of construction, thus minimizing construction costs.

Clearwell storage provides the treatment plant with operational flexibility. The following summarizes the issues:

- Provide emergency storage, and operational flexibility.
- Provide adequate contact time (CT).

- Potential water storage for the backpulse, flushing, and chemical clean-in-place (CIP) processes for the MF system.
- Consistent plant startup and shutdown of process equipment due to varying demands (balancing plant output with high service pumps output).
- Minimize frequency of high service pumps start/stop during MF backwash sequence.
- Minimize wear and tear due to start/stop of mechanical equipment such as raw water feed pumps, chemical feed pumps, MF vacuum pumps, RO booster pumps, high service pumps, valves and actuators which increase equipment maintenance and replacement.
- Provide more water availability for plant use and distribution system demands during plant shutdown due to process equipment repair/maintenance.

8.2.2.3.2.3 CT Requirements of the Surface Water Treatment Rule

One of the primary functions of the clearwell is to provide disinfection CT requirements. "CT" is defined as the product of the disinfectant residual, in mg/L, and the disinfectant contact time "T10", in minutes. The Surface Water Treatment Rule currently requires all water treatment facilities that utilize groundwater under the influence of surface water or surface water supplies to provide 3-log removal/inactivation of *Giardia* cysts and 4-log removal/inactivation of viruses through the CT concept.

The EPA Guidance Manual for Compliance with the Filtration and Disinfection Requirements for Public Water Sources states that a conventional WTP with sedimentation/ filtration will receive credit of 2.5 log removal of *Giardia* and 2.0 log removal of viruses. Therefore, with conventional filtration (Alternative B), disinfection at the WTP must provide at least 0.5-log inactivation for *Giardia* and 2.0-log for viruses. If the WTP uses MF membranes for filtration (Alternative A), additional *Giardia* credits would be given by the State, so disinfection at the WTP would only have to provide 2.0-log for viruses and maintaining chlorine residual throughout the distribution system.

While the installation of MF filtration will meet all current and recently promulgated regulations, the bin classification for Cryptosporidium under the recently promulgated LT2 ESWTR is not known at this time for the surface water impoundment supply. Therefore the log removal requirement is not known. The plant using MF filtration will achieve at least 4-log of

Cryptosporidium, however conventional filtration typically achieves 3-log of Cryptosporidium. Depending on what bin classification the water supply falls under, if using conventional filtration, additional Cryptosporidium removal may be required. To achieve additional Cryptosporidium removal, UV disinfection is a process alternative that could provide additional disinfection at a low cost and footprint. The impact of adding a future UV facility to the current plant, as currently laid out, is minimal. Since it is not known at this time whether additional Cryptosporidium removal is required, this was not evaluated in any further detail.

A CT evaluation was performed for the new 2.0 MG clearwell to ensure that all CT requirements would be met with the new WTP. The following summarizes the criteria used:

WTP Finished Water Flows: 5 MGD, 10 MGD, 15 MGD, 18 MGD
Free Chlorine Residual: 1 mg/L
Clearwell Volume: 1.0 MG (assumes clearwell ½ full)

t_{10}/T for clearwell: 0.5
Finished Water pH: 8.5
Temperature: 0.5^{o} C and 20° C

The evaluation was based on an additional 0.5-log reduction for *Giardia* and 4-log reduction for viruses. Based on the criteria stated above, for cold water conditions with a temperature of 0.5^{o} C, CT for both *Giardia* and viruses are met at an ultimate firm capacity of 18 MGD.

8.2.2.3.2.4 High Service Pump Station

The proposed high service pump station will be located above the new clearwell with space for six vertical turbine pumps. Vertical turbine pumps were chosen since they provide a wider flow range and require lower net positive suction head (NPSH). Each high service pump will provide a capacity of 3.6 MGD for a firm capacity of 18 MGD. During the first phase of construction, only three (3) vertical turbine pumps will be installed. Infrastructure will be in place to provide ease of installation of the pumps when Phase 2 construction begins to bring the WTP to ultimate capacity. See Figures 8-34 and 8-35 for conceptual details of the high service pump station. Each pump will be provided with pump control valves to minimize surges into the system.

Figure 8-34

Figure 8-35

In addition, surge tanks will be incorporated to minimize surge pressure at the pump station in the event of a power outage.

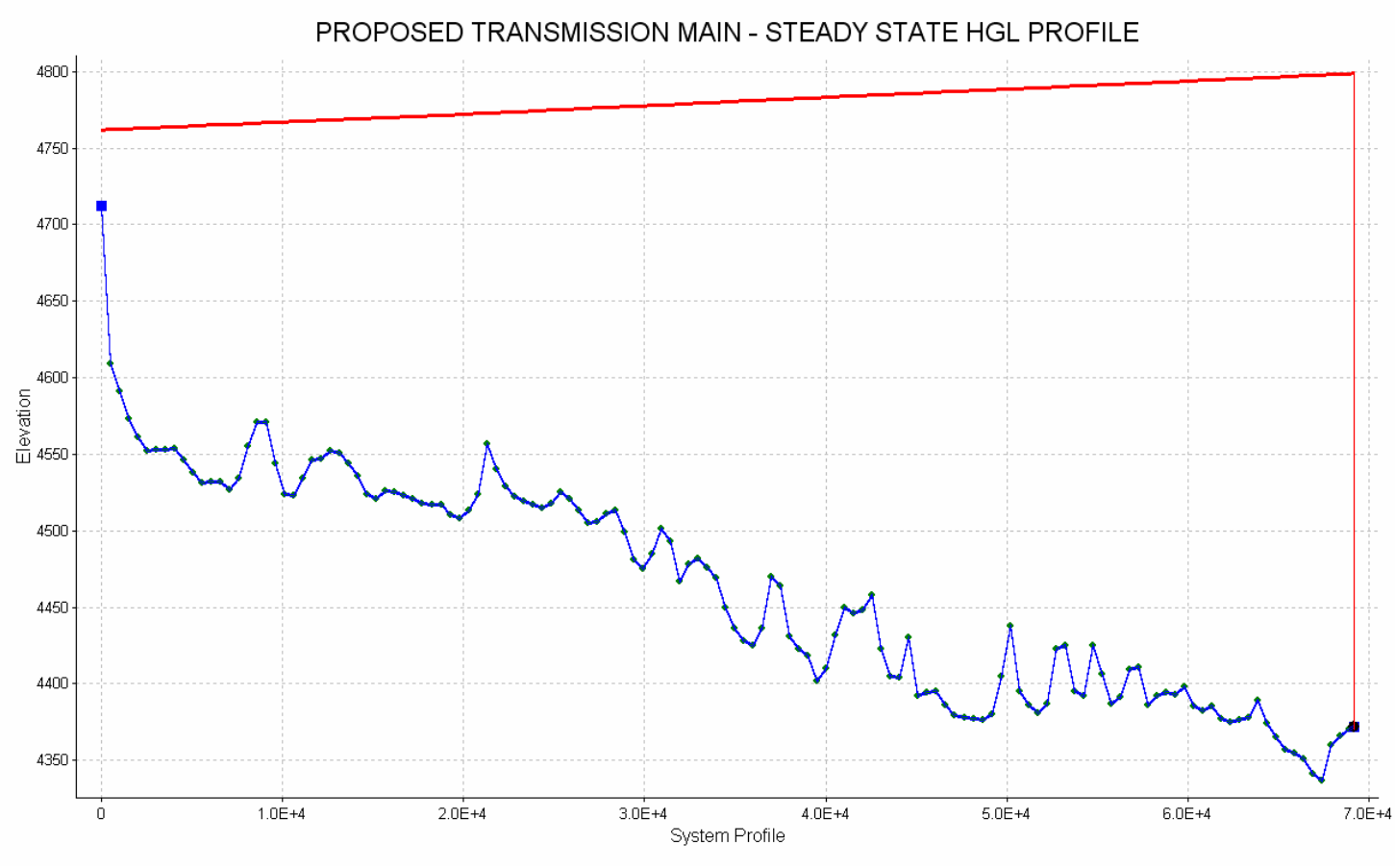
8.2.2.4 New Transmission Main

The proposed transmission main selected for hydraulic and transient analysis was a 42-inch diameter pipeline from the pump station located at the treatment facility clear well. The pump station was operated under a high head operating condition which was pumping water from the clear well with the lowest water surface elevation of 4370, to terminal reservoir 1-R4 operating at the highest water surface elevation of 4762. The design flow rate of 13,000 gpm required the pump station to have a total dynamic head (TDH) of approximately 430 feet to operate under this condition.

The software used for hydraulic and transient analysis was KY Pipe's PIPE 2006 v 3.011 with the Surge analysis module. The modeling process began with importing the ground surface profile of the transmission main alignment into the model. Junctions were specified at 500 foot intervals with elevations corresponding to the ground surface profile at the same station as the existing transmission main or digital elevation model. Pipe segments were then input to connect each junction. The pump station location, pipe diameter, and pipe materials were then specified along the proposed transmission main alignment.

The transmission mains and pump stations were analyzed and calibrated to flow 13,000 gpm under steady state conditions. Figure 8-36 shows the steady state HGL of the transmission main.

Figure 8-36



The steady state reservoir conditions are as follows.

- Proposed clear well at low operating water surface elevation = 4370
- Terminal reservoir Z1-R4 at high operating water surface elevation = 4762

These conditions necessitate that the maximum pump TDH be used to supply water to terminal reservoir Z1-R4 from the clear well. Other assumptions include:

- Steel pipe Hazen-Williams Coefficient = 130
- PVC pipe Hazen-Williams Coefficient = 150
- Minor losses assumed negligible
- Pump stations were assumed to be an vertical turbine

Assumptions and values used were similar to those discussed in depth in the previous alternative.

The steady state model pump operating at 13,000 gpm was specified to stop over a 1 second interval, stopping the flow, and generating the worst case surge conditions. The pumps were allocated check valves, and the simulation was analyzed over a time duration of 180 seconds. Figure 8-41 shows the surge wave pressure envelope bound by the maximum up surges and down surges, for the modeled transmission main.

Figure 8-37

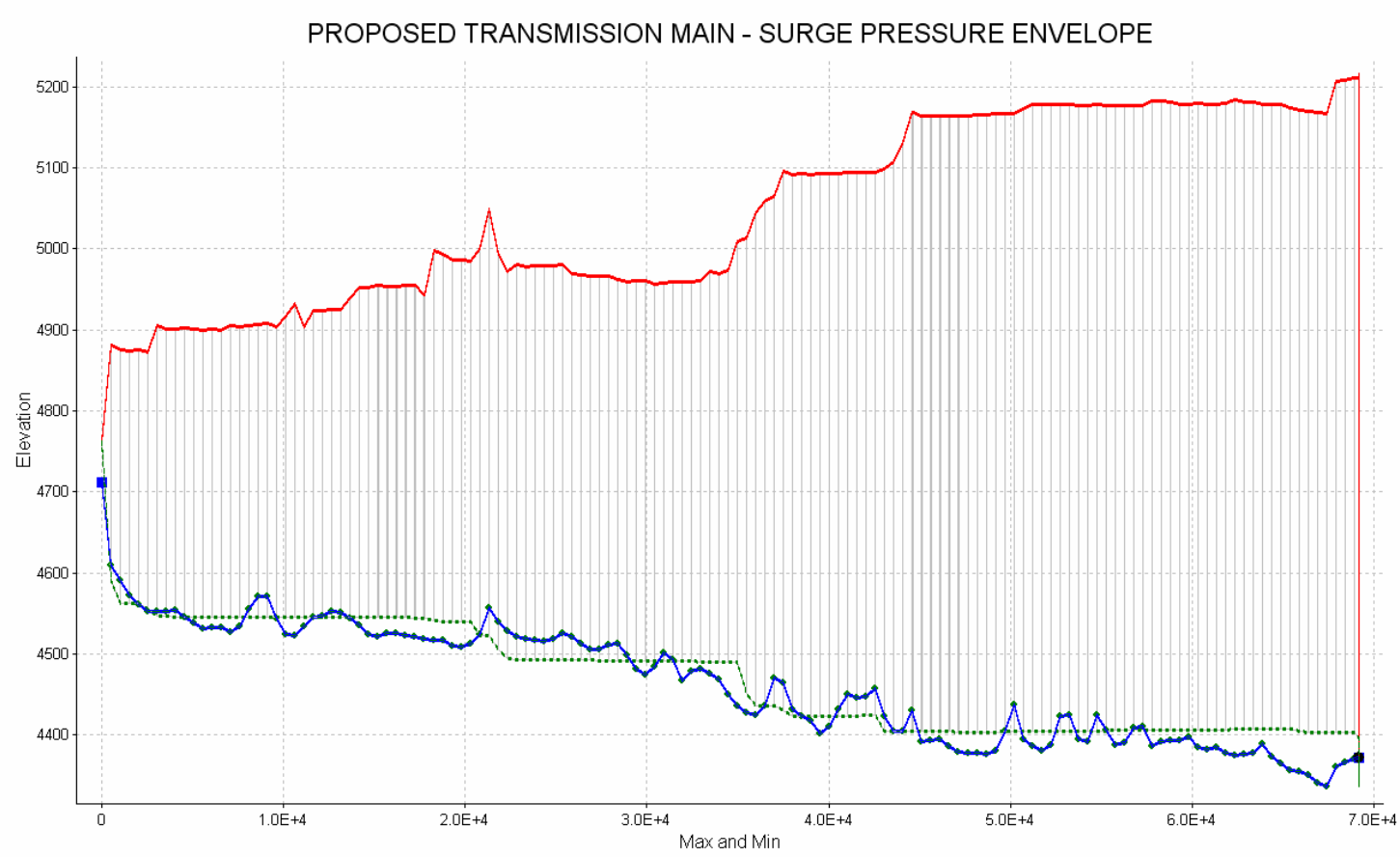


Figure 8-37 shows that the pressure envelope for the modeled alternative necessitates much higher pressure class pipes to handle the surge conditions generated from pump shut-down, compared to that of the high head steady state operating condition. It is readily apparent from the figures that surge mitigation devices need to be employed to reduce the maximum upsurge and downsurge boundaries.

Modeling results indicate that the transmission main was most effectively protected against high surge pressures by bladder surge tanks. Located within the pump stations, the tanks were the most effective device for limiting high transient pressure events and any subsequent "secondary" surge events delivery pressure waves back to the pump stations. The bladder surge tanks were sited on the discharge side of the pumps. Due to the relatively high flow rate of 13,000 gpm, the surge bladder tanks required large volumes, with initial bladder pressures of approximately 60% of the normal operating pressure.

Blow off hydrants are typically located at all low elevation points along the pipeline alignment to facilitate flushing and draining. It is also standard to site air release and vacuum valves at all points of vertical intersection along the pipeline alignment. The model did not incorporate combination air release and vacuum valves at the high points. Only valve assemblies specifically required to mitigate negative pressure due to down surge were included in the model. Blow off hydrants were also not included in the modeled transmission main alternatives.

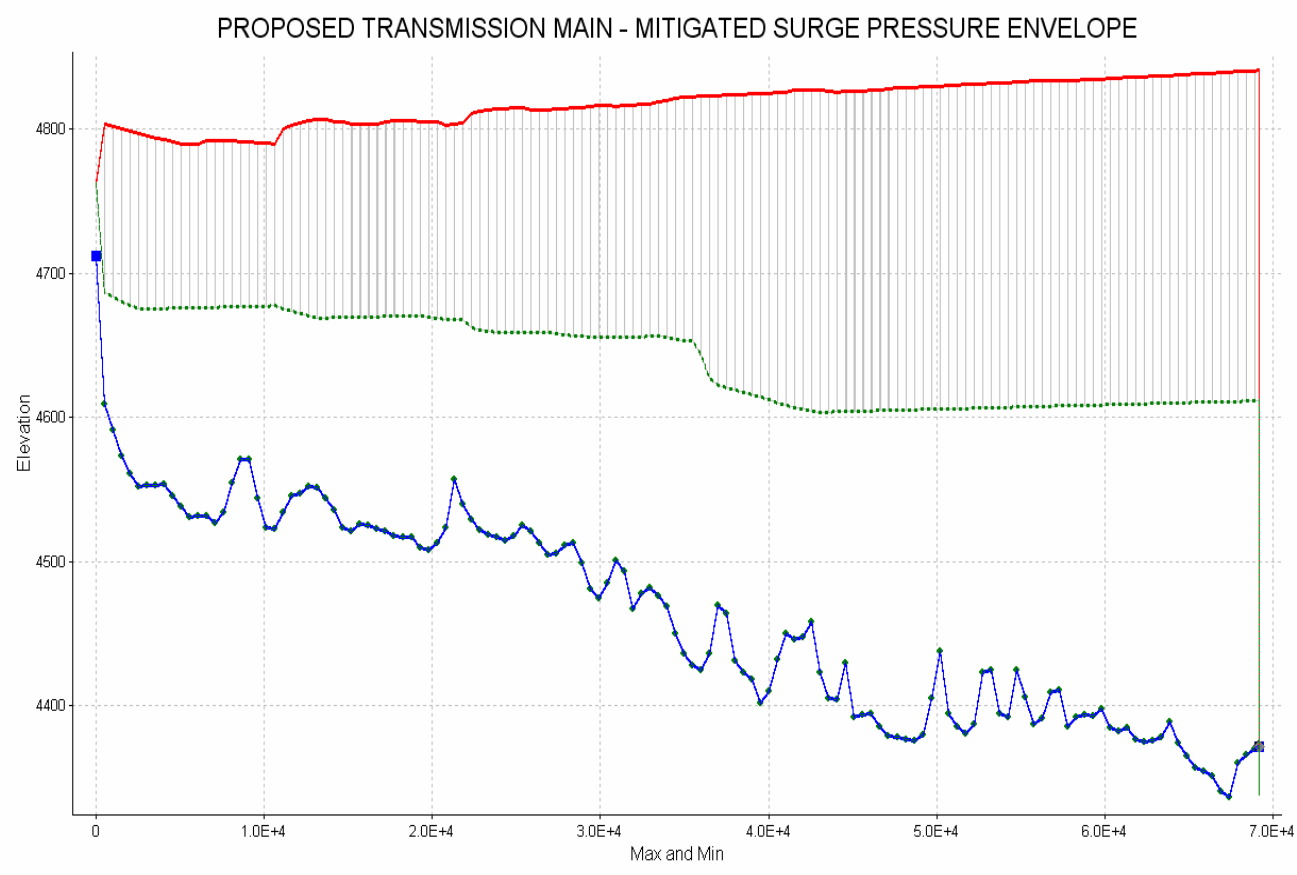
Since complete surge pressure wave removal from a transmission main is not always possible or economical, both alternatives required that portions of higher pressure class pipe be extended beyond the lengths initially calculated to handle static and normal pumping operation conditions. Table 8-19 quantifies the pipe lengths by pressure class and material, pump station required TDH and horsepower, and the recommended surge devices.

TABLE 8-19
Proposed Transmission Main Modeled Quantities

Item	Unit	Total Units
42" PVC 125	LF	17500
42" PVC 165	LF	17000
42" Steel 200	LF	29000
42" Steel 250	LF	4725
Pump Station (430-ft TDH)	Hp	2428
10,000-gal Surge Bladder Tank	EA	1.5

Figure 8-38 shows the final surge pressure envelope after transient mitigation with surge bladder tank/tanks having a total volume of 15,000 gallons, and portions of pipe classes increased to accommodate the remaining surge pressures that were not removed by the surge bladder tank.

Figure 8-38



Total Project Cost – 9 MGD: $101,103,000

Total Project Cost – 18 MGD: $114,834,000

8.2.3 Groundwater Storage and Recovery of Madison Water in the Fort Union Aquifer

When considering storage based alternatives, annual volumes of storage required are so large that they limit reasonable options for storage to either a surface water impoundment (discussed above) or underground storage in an available aquifer. The source of water for storage will be either the Madison aquifer, which is the only source in the Gillette water system that potentially offers excess pumping capacity that might be used to fill storage during part of the year, or treated coal bed methane (CBM) groundwater provided by local CBM well operators. Underground storage in an aquifer would involve injecting the water into one or more of the three aquifers under the City of Gillette area. The same wells used for injection would later be used to pump water out of storage in the aquifers. This type of operation is referred to as an "aquifer storage recovery" (ASR) project.

In comparison to the surface water alternative, groundwater stored in an aquifer by an aquifer storage recovery (ASR) project will not require surface water treatment beyond chlorination so long as the water remains in a closed system from the Madison aquifer or CBM source until it is injected into an aquifer. Therefore, the ASR option trades off the cost of ASR injection/production wells and operating costs against costs for a dam and surface water treatment plant for the surface storage option.

The aquifers available to store groundwater at Gillette include sandstone strata in the Wasatch, Fort Union, and Lance/Fox Hills formations. Typically, water to be stored in a confined aquifer, such as the Fort Union and Lance/Fox Hills aquifers at Gillette, is injected through the same wells that later pump the water back out of the aquifer for use as municipal water supply. If existing wells are to be used for injection, storage of water in the groundwater system would be limited to the Fort Union and Lance/Fox Hills aquifers that are currently penetrated by wells operated by the City of Gillette.

The longest period of required pumping from storage is 111 days at the demand level projected for 30 years from now. This period leaves 254 days per year for injection of water for storage in an aquifer. The annual storage volume of 539,944,310 gallons per year from Table 8-16 can be injected in a period of 254 days with 13 injection wells, assuming an injection rate of 120 gpm per well and injection 24 hours per day. However, it will take considerably more wells to extract the stored water to meet peak demands. For example, current experience with the Fort Union

aquifer indicates that new wells in the formation may be expected to yield from 275 to 325 gpm. Assuming the groundwater levels in the wells have been elevated by ASR injection, a yield of 300 gpm might be used for planning.

Simultaneous pumping from 43 ASR wells at a yield of 300 gpm per well is required to provide the peaking flow of 13,000 gpm. Thus, far more wells are required to extract groundwater at the peak pumping rate than are required to inject water into storage in the aquifer at smaller rates over a longer period of time. The requirement for at least 43 wells to provide the peak pumping requirement, over and above the existing well field and transmission line capacities, is the greatest single disadvantage of the ASR concept applied to the City of Gillette water system, assuming all the technical issues can be resolved. At an estimated cost of nearly $1 million per ASR well, this is a considerable expense.

It might be possible to increase the yield of ASR wells by stimulating them with hydraulic fracturing. Hydraulic fracturing can be used to increase the effective hydraulic radius of a well, i.e., the surface area of aquifer yielding water to a well and therefore improve its hydraulic performance and yield. If a several fold increase in the Fort Union well yields can be obtained by hydraulic fracturing of the sandstone formations, the number of Fort Union aquifer wells required to meet the peak pumping demand of 13,000 gpm might be reduced dramatically. Information about the effectiveness of hydraulically fracturing sandstone units in the Fort Union aquifer is not presently available.

The largest obstacle to providing the latter technical solution to the limitation imposed on ASR concepts by current Fort Union aquifer well yields is regulatory policy. The current policy of the Wyoming Department of Environmental Quality (WYDEQ) prohibits hydraulic fracturing of aquifer formations during injection of water. Injection pressures are limited to 0.7 psi per foot of overburden. This policy exists to prevent CBM well operators who are injecting poor quality groundwater or brine into a receiving aquifer from fracturing that aquifer and allowing the injected fluids to leak into other aquifers that contain fresh water.

The latter type of injection is different as it proposes injecting good quality groundwater into an aquifer where the quality of water in the receiving aquifer will be maintained or improved. Moreover, the Fort Union formation is quite thick and water-bearing zones consist of multiple sandstone layers individually separated by and confined between inter-layered shale and

mudstone strata. Hydraulic fracturing of this formation could be limited to selected layers of sandstone in the middle of the formation, leaving the overlying and underlying sandstone and shale units as a buffer to any interformational leakage that might result from hydraulic fracturing of the select layers. Following hydraulic fracturing of the select layers in the formation, injection pressures would be maintained well below the pressure required to induce fracturing. This technical solution would allow stimulation of ASR well yields without letting injected water potentially leak into formations above or below the Fort Union aquifer or allowing cross contamination of the Fort Union aquifer strata by poor quality water from the overlying Wasatch Formation or the underlying Lance/Fox Hills aquifer.

The current WYDEQ policies do not allow the type of hydraulic fracturing program suggested above. In a conversation with the WYDEQ staff, they indicated a willingness to review plans for this type of project; however, they did not indicate that such a project could be approved or what the mechanism of approval would be under the present WYDEQ rules. Under these circumstances, current economic and financial analysis should assume that it will not be possible to stimulate increased well yield from Fort Union aquifer ASR wells and that up to 43 ASR wells may ultimately be required to develop the 13,000-gpm peak demand flow plus two backup wells to provide firm capacity and redundancy for a total of 45 ASR wells..

8.2.3.1 Wasatch Formation

Very little quantitative information about the Wasatch Formation in the Gillette area was found in the existing records. The report titled City of Gillette – Water Master Plan Report, dated December 2004, by Wester-Wetstein and Associates provides the following information about the Wasatch Formation:

> "***Although the city of Gillette has not produced water from the Wasatch Formation since 1981, at one time the Wasatch Formation supplied a large portion of the City's water needs. Water from the Wasatch Formation is characterized as "very hard" with hardness levels of 2,000 mg/L as $CaCO_3$. Wasatch Formation water also has increased levels of TDS, sulfates, manganese, and iron concentrations that exceed the USEPA SMCL's. Wasatch Formation water has TDS, sulfates, manganese, and iron concentrations at 2,800 mg/L, 1,700 mg/L, 0.4 mg/L, and 0.4 mg/L,***

respectively. The City of Gillette treated the Wasatch Formation water at an electro dialysis plant and the treated water was then blended with water from the Fort Union Formation and Lance/Fox Hills Formation. Development of the Madison Formation Wells allowed the City of Gillette to discontinue producing water from the Wasatch Formation."

The Wasatch Formation may offer potential aquifer storage for an ASR project. However, the current information about the aquifer is not adequate to support any conclusions regarding how much groundwater it may be possible to store in the formation on a seasonal basis. The information provided in the Wester-Wetstein and Associates report indicates the groundwater quality in the Wasatch Formation at Gillette is not only very poor for use as a public drinking water source, but presents the potential for a number of unfavorable geochemical reactions if injected with recharge water from an outside source. Before the Wasatch Formation can be considered for use in an ASR program, its hydraulic properties and potential for geochemical reactions with recharge water must be carefully investigated. Assuming the latter factors are acceptable, the Wasatch Formation may offer some advantages over the Fort Union Formation for use in an ASR project.

Ultimately, the attractiveness of an aquifer like the Wasatch Formation for an ASR project does not hinge upon its present water quality characteristics since the native formation water will be replaced by recharge water of a different quality. The principal attraction of the Wasatch Formation for an ASR project would be if it offers better hydraulic properties than other strata like the Fort Union Formation. For example, if well yields in the Wasatch Formation significantly exceed those in the Fort Union Formation, it would take less wells overall to operate an ASR project in the Wasatch Formation. Likewise, wells in the Wasatch Formation would be significantly shallower than wells in the Fort Union Formation, thus reducing potential capital investment and operation costs. Additional research about the Wasatch Formation is required to determine if it merits further consideration and investigation for use in an ASR project.

The report titled Phase I Interim Report for Gillette Area Master Plan, Gillette, Wyoming, dated May 1993, by HKM Associates provides the following information about the Wasatch Formation: "The City of Gillette formerly had 20 Wasatch wells that had yields of 50 to 100 gpm when they were originally drilled and completed." While the construction of these wells is unknown, it is not certain that increased yields are possible from this aquifer. It is worth noting that these yields

are 1/6 to 1/3 of those assumed from the Fort Union wells. This means 3-6 times as many wells would be required to produce the same peak flow. While the costs for drilling such wells would likely be much lower due to the shallower depth of the Wasatch, the connection piping, power distribution, and well site costs would offset that to a large degree. From an order of magnitude perspective, presumably a Wasatch development would be similar in cost to that in the Fort Union, but with many more unknowns and risks due to geochemical issues. While the Wasatch Formation may prove feasible after further exhaustive investigations, such investigations are out of the scope of this report. Given the high level of uncertainty surrounding this source it is not a preferred formation and will not be discussed further.

8.2.3.2 Fort Union Formation

The Fort Union Formation may offer very attractive conditions for use as a storage unit for an ASR project. The Fort Union Formation contains multiple layers of sandstone separated by shale and mudstone offering relatively low permeability. This stratigraphic sequence may allow “stacking” of storage of injected recharge in multiple zones, thus increasing the efficiency of the storage and recovery of injected recharge water. The presence of multiple zones may also provide a properly designed ASR project with a buffer between overlying and underlying aquifer systems such as the Wasatch Formation and Lance/Fox Hills aquifer.

Disadvantages of the Fort Union Formation as a storage unit for ASR operations include the requirement for relatively deep wells and a current absence of knowledge about how many of the multiple sandstone zones offer groundwater production and/or storage potential. Also lacking is good information about the lateral continuity and horizontal interconnection of individual sandstone layers in the subsurface. If the individual sandstone layers are not laterally extensive, the available groundwater storage volume for ASR may be limited. However, the available pumping tests of the formation suggest that even if it offers lateral boundaries where individual sandstone lenses pinch out, more than adequate groundwater storage volume is available for ASR operations due to the opportunity to stack stored water in multiple aquifers penetrated by an individual ASR well. In the latter case, recovery efficiency should be 100 percent, particularly when the existing water quality in the Fort Union Formation is acceptable for a public drinking water supply such that recovery of mixed water or native groundwater is not a public health or water quality issue. If an ASR program is pursued, the Fort Union Formation is the preferred formation based on the information available at this time.

8.2.3.3 Lance/Fox Hills Aquifer

Aquifer test data for the Lance/Fox Hills aquifer were not found in the course of this investigation. However, wells FH-4 and FH-5 typically yield 550 to 650 gpm when used by the City of Gillette. In comparison, typical yields from the older Fort Union aquifer wells operated by the City of Gillette are less than 200 gpm. It is assumed that properly constructed Fort Union aquifer wells with groundwater levels elevated by recharge would offer yields with an upper range of 250 to 300 gpm. Therefore, the Lance/Fox Hills aquifer may be attractive for ASR operations because it would potentially require only half the number of wells needed in the Fort Union aquifer. This advantage would be offset to an unknown extent by the fact the Lance/Fox Hills aquifer is deeper and will require deeper wells. It is assumed that the native water quality of the Lance/Fox Hills groundwater will be replaced by the quality of the recharge water. While the Lance/Fox Hills Formation may prove feasible after further exhaustive investigations, such investigations are out of the scope of this report. Given the high level of uncertainty surrounding this source it is not a preferred formation and will not be discussed further.

8.2.3.4 Risk Management

The aquifer storage and recovery (ASR) concept presents a number of unanswered questions that offer a relatively high level of risk. Some of the risk is related to site-specific issues such as the number of ASR wells required to provide the peak demand flow as discussed above. Some of the risk is related to regulatory issues. Some of the risk is related to other technical issues pertaining to injection of Madison aquifer water or treated CBM water into the Fort Union sandstones in the Gillette area. In general, it can be anticipated that the technical issues and, hopefully, the regulatory issues can be resolved. However, the initial level of effort and financial investment should be limited while the degree of risk is high. This could be accomplished by a phased approach to an ASR project so that technical and non-technical issues can be investigated and resolved in progressive steps before the effort is expanded. It is anticipated that at least the following steps will be appropriate to an ASR program for the City of Gillette:

Phase 1: Preliminary Feasibility Assessment and Conceptual Design
Phase 2: Field Investigations and Test Program
Phase 3: ASR Facilities Expansion

The primary objective of an ASR recharge project at Gillette is to provide water distribution system storage to satisfy peak demands and to do so in a manner that results in economic benefits to the City of Gillette. Secondary objectives may include restoration of groundwater levels in the Fort Union aquifer at Gillette, improvement of water quality in the recharged aquifer, enhancement of existing well field production, and deferred expansion of water facilities or avoidance of a surface water treatment plant. The latter secondary objectives must be considered in the conceptual design and may broaden the base of support for the recharge program. It is also possible that recovery of stored water during peak demand months may help maintain adequate pressures in certain parts of the water distribution system while augmenting the peak flows.

The purpose of the preliminary feasibility assessment is to identify the viability of the ASR concept for Gillette. The current perception of the ASR concept is that it may offer economic benefits, even if it is not possible to hydraulically fracture the aquifer and reduce the number of ASR wells required to provide peak flows. If this is correct, the WYDEQ regulations prohibiting hydraulic fracturing of the aquifer are not a fatal flaw. Assuming that other regulatory requirements can be satisfied, a fatal flaw in the ASR concept is most likely to result from technical issues. The remainder of this chapter is therefore dedicated mostly to discussion of technical issues, why they are important, and how they should be addressed during a feasibility study.

8.2.3.5 Preliminary Feasibility Assessment and Conceptual Design

Careful attention to initial planning details of an ASR project is necessary to minimize the risk of failure and to support the issuance of permits required for initial testing and subsequent expansion of ASR operations. If a Fort Union aquifer ASR project can be accomplished with a reasonable number of ASR wells to support peak demands, the economic benefits to the City of Gillette, compared to the alternatives, will be significant. The penalty for failure is therefore potentially great. If the initial ASR effort fails because important issues are not resolved in advance, it will become necessary to pursue a potentially much less economically attractive alternative. It is therefore important to maximize the chance of success of the ASR concept so that early problems in the approach do not lead to a loss of confidence in the concept.

Success of an ASR project in the Fort Union aquifer at Gillette will require successful solutions to both technical and non-technical issues. Experience gained from other ASR projects in the United States has resolved many of the technical issues that must be solved. However, as described below, a number of technical issues must be adequately addressed before the feasibility of the ASR concept as applied to the Fort Union aquifer can be decided. In addition, the regulatory, legal, political, and environmental issues will be significant factors in the success or failure of the ASR program. Therefore, the preliminary feasibility assessment is required not only to address the technical issues, but to put the proposed recharge project in context with the broader water management plan for the City of Gillette so that public support and a consensus from the regulatory agencies can be pursued successfully.

8.2.3.5.1 Water Source

Potential sources of water for an ASR project at Gillette are the Madison aquifer currently used by the City and treated water from CBM wells. Because the Madison aquifer source is operated by the City, there are no problems associated with monthly variability in the supply, seasonal variations, long-term decline in source production, or potential for contamination. If CBM water is to be used as a source or co-source, steps must be taken to minimize variability in CBM water flows and somehow ensure that if the CBM water is somehow contaminated, that contamination can be detected before the water is delivered to the City of Gillette ASR wells or the municipal water system. Madison aquifer water may be used to offset short-term variability and/or long-term decline or cessation of the CBM source, particularly if more than one CBM operator is involved in the source, thus offering a solution to potential variability in CMB water supply. However, it is less clear how the municipal water supply can be protected from potential contamination of the CBM water and this issue must be resolved on a practical level if CBM water is to be considered as a water supply source.

8.2.3.5.2 Recharge Water Quality

The preliminary feasibility assessment must demonstrate sound plans to ensure a consistent quantity and consistent quality of recharge water (aside from contamination issues), particularly if multiple CBM operators are to provide the source water. Likewise, an important part of the preliminary feasibility assessment will be to show that recharge water quality constituent concentrations will meet primary and secondary drinking water standards.

Use of CBM wells as a source of recharge water may raise several technical water quality issues. One issue is that of mixing CBM water with water from the Madison aquifer. The two sources offer water with different chemistry. The preliminary feasibility study must address this issue and determine the result of mixing the two different chemical types of water. Likewise, the feasibility study must determine if variations in the mixing proportions between Madison aquifer and CBM well water is a significant problem in use of water from both sources, particularly if there will be variability in the rate of flow available from the CBM wells. The Madison aquifer produces hard water and the CMB wells produce soft water. Therefore, there may be a water quality issue with the end users if operation of the two sources of recharge water results in significant variability of water quality characteristics over time. The feasibility study should identify how this potential problem can be minimized in the operation of the ASR program.

8.2.3.5.3 Recharge Water TSS

One of the most crucial factors in plugging of ASR wells is the suspended solids content in the recharge water source. Significant plugging of ASR wells may occur during a relatively short period of time with total suspended solids (TSS) concentrations as low as 2 mg/L. Fine-grained formations such as the Fort Union aquifer sandstones are particularly sensitive to plugging by suspended solids. Therefore, a particularly important aspect of the preliminary feasibility study is determination of the TSS content of water from the source aquifers, including wells that might yield a small amount of sediment when first started and then produce water with little or no TSS content after a brief period of continuous pumping. Failure to identify the initial production of sediment upon well startup may result in failure to recognize a significant potential for ASR well plugging.

Addition of CBM wells as a source of water magnifies the potential for problems with TSS content in the recharge water. The source of CBM recharge water will be multiple wells, potentially operated by several different operators and potentially with several different types of well construction and associated sediment yield. Currently, individual CBM producers operate hundreds of CBM wells. Initial identification of the specific source wells for raw recharge water may be impossible during a preliminary feasibility study and individual operators may desire to change the source wells over a period of time for the convenience of the CBM operations. Accordingly, recharge water provided from CBM sources will likely require treatment to remove TSS content and colloidal flocs before it is provided for use in the ASR project.

Another potential source of TSS that could plug the ASR wells is sediment or colloidal suspensions in the existing water distribution system. The preliminary feasibility study should identify exactly how the ASR recharge water will be conveyed through the water distribution system to where it is injected into the ASR wells and how injection of existing sediment or colloids in the existing water distribution system transmission lines will be avoided.

8.2.3.5.4 Hydrogeology

Much of the hydrogeology of the aquifers under the Gillette area is known well enough that some of the feasibility study and conceptual design phase of the ASR project does not require field investigations. However, technical unknowns remain and must eventually be investigated by drilling, testing, and laboratory analysis. For example, the stratigraphy of the aquifer; its areal extent, thickness, and depth; the areal extent, thickness and depth of the confining layers; and the lithology of the aquifers and confining layers are known at a very generalized level of detail. Likewise, general information is available about typical well construction and production rates. The latter information has recently been expanded by construction of filter-packed wells by the City of Gillette and the nearby Sleepy Hollow water systems. The existing information includes some knowledge of the collective hydraulic properties of the aquifer from tests of multiple production zones; however, information about the hydraulic properties of individual water-bearing zones has not historically been available.

The available information about the Fort Union aquifer suggests it offers relatively favorable conditions to minimize mixing of recharge water and native water in the aquifer. The aquifer consists of relatively thin sandstone beds separated by shale and mudstone beds that provide vertical confinement. These conclusions are supported by the available pumping test data which do not evidence indications of vertical leakage across the confining beds between water-bearing zones. These same conditions indicate that recovery of recharge water can be 100 percent. The fact that the native water quality in the storage intervals in the Fort Union aquifer is acceptable for drinking water use means that recovery of mixed water or native water is not a drinking water quality issue.

The thin, vertically confined nature of the sandstone storage intervals in the Fort Union aquifer may offer an additional opportunity to control geochemical reactions in the aquifer that occur relatively rapidly at the surface of the aquifer mineral grains. Such reactions include oxidation,

ion exchange, and adsorption. These reactions may be controlled by initially injecting a large volume of water into the storage intervals so that a buffer of recharge water will surround the ASR well to a sufficient distance to prevent mixed water or native aquifer water from flowing to the well during the ASR operations. If the operational storage and recovery of recharge water is maintained as a “bubble” around the ASR well or well field and is in turn surrounded by a buffer zone of recharge water that is not intended to be recovered, the geochemical reactions between the recharge water and the native aquifer water will be eliminated in the operational storage. The thin, vertically confined nature of the Fort Union aquifer storage zones appears to be ideal for this type of operation.

Additional information about the hydraulic properties of the Fort Union aquifer strata will be required not only to predict ASR efficiency, but for estimating how much recharge water stored in the Fort Union might be captured by non-municipal wells.

8.2.3.5.5 Well Plugging Considerations

Considerable investigation is needed to determine the mineralogy of the water-bearing sandstones and the confining layers in that part of the Fort Union formation under consideration for use to store water under the ASR concept. The work must include careful investigation of clay content and the mineralogical type of clay content as pertains to potential for clay dispersion and plugging, sand mineralogy as pertains to reaction with recharge water, microbiota in the aquifer pores, geophysical logs of specific wells and specific stratigraphic sections, and geochemical compatibility of recharge water with the formation minerals and the existing groundwater in the formations, to name a few considerations.

Well plugging that is not caused by suspended solids or colloids in the recharge water might occur in a number of ways. Chemical reactions between the recharge water and the native water in the aquifer or between the recharge water and the minerals in the aquifer matrix may cause precipitation of calcium carbonate, iron oxides and oxyhydroxides, manganese oxide hydrates, or silicates that plug the pores in the aquifer. A second cause of well plugging is related to the presence of clays, silts, and other fine-grained materials in the water-bearing zones that are to be recharged. Geochemical reaction between recharge water and the clays may result in either swelling, dispersion, or restructuring of the clay particle arrangements to result in plugging of the aquifer pores.

Biological growth is another major source of plugging of aquifers. Plugging mechanisms include filling of aquifer pores by impermeable slimes, dead biofilm, and byproducts such as iron and manganese oxides and oxyhydroxides. Microbiologic activity may stimulate dispersion or swelling of clay minerals in the aquifer matrix. Bacteria may be responsible for much or most of the changes in groundwater chemistry in ASR wells. Accordingly, the preliminary feasibility study must consider the potential for biofouling and methods for controlling biologic activity in the ASR wells. This potential should include identification to the extent possible of the types and reactivity of bacteria in the aquifer matrix receiving the recharge water.

8.2.3.5.6 Core Samples

The basic question is if the preliminary feasibility study and conceptual planning may contain a fundamental flaw due to lack of knowledge. The greatest potential for a fundamental flaw in the preliminary feasibility study is offered by the risk of geochemical plugging. This risk is substantial because there is no local recharge experience to review and there is insufficient data available to support an assessment of the risk. This fact argues for field investigations early in the feasibility planning process because it will be necessary to obtain rock cores of the water-bearing zones and of the confining layers between the water-bearing zones in the aquifer for mineralogical, chemical, and biological analysis. The latter data are required to evaluate the potential for a fatal flaw in the ASR planning process. This information should be collected and evaluated before any substantial investment in ASR testing or operation is made.

Analyses of rock cores from the aquifer and confining layers can be analyzed by laboratories that support petroleum industry operations. Typical physical analyses include determination of physical characteristics such as vertical and horizontal permeability, grain-size distribution (percentage of silts, clays, and other fine-grained materials), and porosity. In addition to physical analyses, the rock cores are subjected to X-ray diffraction analysis to determine the types and percentage of clay minerals present in the core. This information is an important indicator of potential for geochemical plugging.

The core is also analyzed for the types and concentrations of cations in exchange positions in the aquifer mineralogy and the cation/base exchange capacity. The cation exchange capacity is an indicator of the potential for clay in the aquifer matrix to react to the recharge water chemistry and become unstable or swell. Scanning electron microscopy is used to determine where the

clay minerals are present in the mineral matrix of the aquifer and their potential to swell into the formation pores. Thin section petrography is used to evaluate the susceptibility for plugging and to identify mineral grains such as pyrite and coatings that affect the amount of oxidation reactions that might occur with the recharge water. The pores in the rock core are also inspected to determine the type of microbiota present in the aquifer. This provides better information about the types of bacteria present in the aquifer than does analysis of water samples from the aquifer.

8.2.3.5.7 Geochemical Reactions

All of the foregoing information about recharge water chemistry, aquifer water chemistry, and mineralogy of the aquifer rock is used to evaluate the geochemical processes that might occur when recharge water is injected into the receiving aquifer.

8.2.3.5.8 Biofouling

Factors affecting growth of microorganisms and well plugging include not only water temperature and pH, but correlations between total nitrogen, particulate organic carbon, dissolved organic carbon, dissolved oxygen, total iron, and biologic load.

8.2.3.5.9 Redox Potential

An important factor influencing groundwater reactions is the oxidation-reduction potential or redox potential, typically referred to as ORP, pE, or Eh. This factor must be measured *in situ*, a process that can be difficult. This measurement is required in place of Dissolved Oxygen (DO) measurements in deep groundwater with low DO concentrations and determines the degree of reduction in the water. The degree of reduction will distinguish between inorganic-dominated versus organic-dominated (biotic or microbiotic) controlled groundwater systems. Accordingly, the measurement allows the inorganic versus the microbiota reaction paths to be ascertained. This is particularly important where iron and manganese concentrations are above approximately 0.1 mg/L and there are concerns about iron-related bacteria and/or development of colloidal iron flocculants that may plug the aquifer pores.

8.2.3.5.10 Ion Exchange

Ion exchange can be a major factor in aquifer plugging. For example, if significant amounts of montmorillonite clay are present in the aquifer matrix in the Fort Union aquifer, which is sodium dominated, the clay will be stable because of the sodium in the exchange positions on the clay particles. Recharge with calcium-bicarbonate water from the Madison aquifer may cause sodium in the exchange positions to be replaced with calcium. Sodium montmorillonites have an open, porous structure whereas calcium montmorillonites have a dense, platy structure. The change from sodium montmorillonite to calcium montmorillonite in an aquifer can destabilize the clay particles attached to the mineral surfaces in the pores and allow them to move into the areas of intergranular contact where they restrict or plug the interconnection of the pores. This type of aquifer plugging is nearly impossible to reverse. Accordingly, the various tests mentioned above to determine clay content in the rock core from the aquifer and the distribution of that clay relative to the aquifer pores underscores the need to obtain the rock core samples and perform the various analyses.

8.2.3.5.11 Oxidation

Oxidation processes are very important to geochemical reactions in the aquifer. Iron carbonate (siderite) and iron sulfides (pyrite and marcasite) are the most susceptible minerals to oxidation by recharge water. The reaction between these minerals and the recharge water takes place very quickly and will generate iron oxyhydroxide floc.

8.2.3.5.12 Adsorption

Iron oxyhydroxide floc is a very sticky substance. Therefore, as it is transported through the aquifer by the recharge water, it becomes adsorbed to aquifer particles at the constricted parts of the tortuous flow path through the mineral grains. The constricted pore throats that connect the pores between the mineral grains are the most important part of the porosity and restriction or complete plugging of these interconnections between the pores permanently reduces the amount of water that can move through the aquifer. Acid treatments of the aquifer after iron floc plugging occurs will restore some of the aquifer permeability, however, if this process is allowed

to progress too far, it will not allow the acid to flow into the affected areas and permanent loss of aquifer permeability will result.

8.2.3.5.13 Dissolution

The solubility of minerals in groundwater is of particular interest to an ASR project at Gillette because one potential source of recharge water, the Madison aquifer, contains hard water with concentrations of calcium carbonate and calcium sulfate that might precipitate under proper conditions. The processes controlling the solubility of carbonates and sulfates in the groundwater are complex and involve more than one step or condition and may in part depend on microbiotic reactions. They are certainly sensitive to pH, water temperature, and water pressure, all as directly related to the partial pressure of carbon dioxide in the water and its effect on carbonic acid concentrations that are a major factor in calcite solubility.

A complete discussion of these relationships is beyond the scope of this work. However, it is important to point out that non-chemical factors also strongly influence the chemical reactions of calcium carbonate and calcium sulfate in an aquifer. For example, mineral solubility is generally controlled by the transport of the cation away from the mineral grain and to the anion in solution. Thus, dissolution is strongly dependent on the velocity of transport away from the mineral, a factor that is usually more significant with recharge and pumping abstractions than during normal aquifer flow velocities. Likewise, particle size can have a significant effect on solubility because dissolution is controlled by reactions at the surface of the mineral. Fractures in a mineral increase its solubility by increasing the surface area exposed for chemical reactions. Likewise, substitutions of certain ions in the mineral structure may increase or decrease its solubility. For example, calcium sulfate (gypsum) is more soluble than calcium carbonate (calcite). Substitution of sulfate for carbonate at the surface of a calcite grain will increase its solubility at the point of substitution. Unfortunately, all of these variables come into play when we try to predict the solubility of carbonate minerals and related sulfate minerals in groundwater.

8.2.3.5.14 Geochemical Models

Rock core analysis combined with recharge water and aquifer water chemistry must be used together to assess the foregoing types of factors in a feasibility field investigation and test program to predict how the Madison aquifer groundwater may react with the mineral matrix of

the Fort Union aquifer and/or the Fort Union groundwater before a substantial financial commitment is made to an ASR project for the City of Gillette. Likewise, the same types of analyses are required if treated CBM well water is to be the source or one of the sources for ASR recharge water, particularly if treatment of CBM groundwater is likely to increase dissolved oxygen concentrations in the recharge water. For example, storage of CBM water in a surface impoundment prior to its treatment and/or discharge into a pipeline to Gillette would introduce significant DO into the water, a fact that would greatly affect geochemical reactions between the injected water, the native groundwater, and the aquifer mineral matrix. Mixing of recharge water from combined Madison aquifer and CBM well sources will add another level of complexity to predicting potential geochemical reactions in the aquifer that might result in aquifer plugging.

One way to identify potential geochemical reactions that could present fatal flaws to an ASR program is use of one or more geochemical models designed to evaluate mixing of recharge water and native aquifer water. These models will indicate the tendency for precipitation or solution reactions to occur. The available geochemical models do not model biologic activity and do not provide insight into the kinetics (rate) of the geochemical reactions. Moreover, they do not take the mineralogy of the aquifer matrix directly into account. Despite these limitations, the models can be used to attempt to understand the prevailing equilibrium between soluble minerals in the aquifer matrix and the existing groundwater quality in the aquifer. Likewise, they can then be applied to making predictions of the effects of changing those conditions by introducing recharge water of known chemistry to the aquifer.

The models are based on water chemistry inputs and the assumption that the native groundwater in the aquifer is in or near equilibrium with the surrounding rock matrix of the aquifer. Therefore, the geochemical models do not require rock core data as input and can be operated based on the chemistry of water samples collected from the native water in the aquifer and the chemistry of the recharge water. This means that geochemical modeling can be performed without collection of rock cores.

The limitations of geochemical models must be recognized. The models contain simplifications of potentially complex relationships, particularly with regard to reaction mechanisms that may not be understood, such as bacterial activity. Therefore, considerable uncertainty may underlie the geochemical analyses provided by the models. In addition, the available data may impose some limitations on the model results. Considering the foregoing limitations of the geochemical

models, there may be a general perception that if the geochemical model results indicate there are no potential problems due to geochemical reactions, it may be reasonable to proceed with an ASR pilot test in the field where as geochemical model results predicting geochemical reaction issues may be the basis to collect rock cores for further evaluation of potential geochemical reactions that might result in ASR well plugging.

The latter logic is faulty when applied to the known factors for an ASR recharge project at Gillette, particularly in view of the previously discussed potential for calcium substitution for sodium cations in clay minerals in the aquifer to cause aquifer plugging. The difference between the calcium and sodium concentrations in the Fort Union aquifer water and the recharge water from the Madison aquifer presents a significant potential for problems with clay in the Fort Union aquifer matrix, if sodium-sensitive clay is present in the aquifer. If the aquifer is hydraulically fractured to stimulate production during pumping out of the aquifer, fractures crossing the confining beds between the productive sandstone units might offer even more opportunity for unfavorable geochemical reactions with clay in the aquifer system.

Nothing is presently known about the amount of clay in the aquifer matrix, how clay may be distributed with respect to the aquifer porosity, or the type of clay minerals (sodium-sensitive or sodium-resistant) that may be present in the aquifer matrix or the confining units. If sodium-sensitive clay is present in the aquifer, it could offer the one single fatal flaw that would cause ASR wells to fail after a period of operation. Therefore, there is no question that rock core samples from the Fort Union aquifer water-bearing strata and intervening shale and mudstone strata should be collected and analyzed at the earliest possible time and considered in the preliminary feasibility study and conceptual design.

However, the rock core effort should be started only after decisions have been made about regulatory and economic factors so that it is clear the project makes good economic sense and there are no insurmountable regulatory obstacles.

8.2.3.5.15 Geochemical Kinetics

Some geochemical models do not provide any information about geochemical kinetics. Other models provide some indication of rates at which geochemical reactions will occur and the geochemical paths the reactions may follow.

The rates of some chemical reactions in aquifers are faster than others. In general, surface reactions such as ion exchange, oxidation and development of flocculants, and adsorption of flocculants will occur much faster than dissolution reactions. Therefore, the changes in water chemistry resulting from rapid reactions may change the ultimate equilibrium reaction path for slower dissolution reactions with the aquifer matrix minerals. Likewise, carbonate reactions in response to sudden changes in temperature and pressure can be rapid whereas carbonate equilibrium reactions between the groundwater and the rock matrix may take months to years and are affected by flow velocity, particle grain size, and other factors as discussed above. Some reactions may take place at such slow rates that they will not affect ASR operations.

Therefore, the results of geochemical modeling, while important for understanding possible reactions that will occur in the aquifer, are not conclusive on several levels. One level is the inherent uncertainty in the modeling assumptions and data quality. A second level is the differences between the times for different types of geochemical reactions to occur. Some types of reactions predicted by the models, if they occur in reality, will occur very quickly. These include formation of colloidal iron floc and precipitation of calcium carbonate due to rapid change of temperature or pressure or mixing of different types of groundwater. However, other reactions predicted by the models may require months or years to occur. It is therefore very important that the results of geochemical modeling be followed by field tests involving pilot ASR operation cycles over a period of months, as part of the feasibility study. Pilot ASR operations will reveal the surface reactions relatively quickly, but determination of the equilibrium between recharge water and the aquifer mineralogy may require a longer period of pilot testing before a commitment is made to a full fledged ASR program.

8.2.3.6 Field Investigations and Test Program

When rock core analysis, geochemical modeling, and other preliminary feasibility analyses and conceptual designs are completed, it will be appropriate to move into the next step of the feasibility analysis, assuming the foregoing effort does not indicate a potentially fatal technical problem in adding recharge to the target aquifer. The next step is an ASR pilot test.

The potential for geochemical reaction issues in the Fort Union aquifer when recharged with water from the Madison aquifer indicates that a short-term pilot test program is inappropriate because some of the critical equilibrium reactions with the aquifer mineral matrix may take more

than 12 months to occur. Therefore, a pilot program of at least 12 months duration is needed. Continued monitoring of water quality for subtle changes should continue from 12 to 24 months, or until it is apparent that the aquifer has reached a geochemical equilibrium. Accordingly, one aspect of the pilot test is investigation of both short-term and long-term geochemical reactions and their effects on aquifer plugging and long-term water quality changes.

8.2.3.6.1 Baseline Tests

Initial testing will include baseline aquifer tests to establish the well and formation loss coefficients, based on conventional step tests. Longer constant rate tests should follow to determine aquifer hydraulic characteristics. These tests are typically followed by a stepped-rate injection test. The baseline tests will provide a standard against which future performance of the well can be compared to determine plugging and loss of efficiency.

8.2.3.6.2 First Recharge and Recovery Cycle

A crucial step in the pilot test will be evaluation of the water quality changes that take place during the initial ASR cycle. The initial ASR cycle is when the recharge water will react with the native groundwater in the formation. Therefore, this cycle should be planned for a specific amount of storage time followed by recovery of stored water so that the amount of mixing between injected water and groundwater can be determined as well as the aquifer response to geochemical and biological changes. The differences in water quality between the injected water, the native groundwater, and the recovered water will give an indication of the amount of mixing and the types of short-term geochemical reactions that occurred. The aquifer hydraulic response during this cycle will be the initial basis for assessing recovery efficiency and/or identifying changes in the baseline hydraulic performance of the aquifer that indicates some type of response to geochemical and biologic change after injection of recharge water.

Because potential geochemical reactions are an issue in all of the potential aquifers for use in ASR operations at Gillette, the initial ASR cycle and subsequent ASR cycles should not recover 100 percent of the injected recharge water. The intention of leaving a small amount of recharge water in the aquifer at the end of the initial ASR cycle and subsequent ASR cycles, is to build up a buffer zone around the well which will increase in size with subsequent injections.

8.2.3.6.3 Subsequent Recharge and Recovery Cycles

Because of the concern about geochemical reactions, the first ASR cycle should be followed by two or three additional cycles of identical injection volume and storage time as the first cycle. These cycles should demonstrate improvement of recovery efficiency with successive cycles. ASR cycles with larger volumes can then follow with recovery of water limited to a target water quality concentration in each cycle so as to increase the size of the buffer zone. These latter cycles should include an increased storage period between injection and recovery; for example, 5 to 7 days, to allow slower oxidation and adsorption reactions to take place and to allow biologic activity to develop. If complex geochemical reactions are detected, it will be desirable to run a larger number of smaller cycles to demonstrate control over geochemical conditions near the well, before advancing to larger operational cycles that extend those reactions further away from the well.

Continued operation of the ASR pilot test will support collection of information about the hydraulic performance of the well. The type of information to be collected should include:

1. Flow rates during injection and recovery,
2. Cumulative volume of water injected into storage,
3. Injection pressures in the well (or depth to water under gravity injection), and
4. The depth to water in the well during pumping to recover recharge water,
5. Membrane filter and/or bypass filter tests to estimate plugging potential of injection water.

The tests based on ASR cycles over a period of months will address a number of practical issues in addition to providing an assessment of well plugging. Some of the issues are as follows:

1. Geochemical reactions and their effect on well plugging,
2. Frequency and duration of backflushing required to control well plugging and maintain injection rates,
3. Results of mixing between injection water and native aquifer water,
4. Water quality changes in constituents and/or properties of concern such as hardness, dissolved iron and manganese, dissolved silica, colloids, etc.,

5. Changes in water quality with successive ASR cycles,
6. Response of injected water quality to increased storage time,
7. Recovery efficiency (this issue will differ depending if native water quality is suitable for use without treatment),
8. Maintenance of disinfectant residuals in well to suppress biological activity by use of trickle injection flow rate of treated water, and
9. Changes in water levels at other wells in the Gillette area in response to ASR operations, including assessment of how much injected water may be recovered by others, a fact that may decrease recovery efficiency.

The foregoing information may be used to support various types of simulation modeling, after the ASR pilot test is completed. ASR modeling may include modeling of the water system operation with ASR wells at specific locations, numerical simulation of the aquifer hydraulic response to ASR operations, modeling of solute transport during injection and recovery, and modeling of geochemical reactions. Some or all of the foregoing types of models may be useful in planning the final design and location of the ASR wells.

8.2.3.7 ASR Wellfield Expansion

After completion of the feasibility investigations followed by the pilot ASR test, sufficient information should be available to proceed with design work to for an ASR well field. Depending upon availability of land, suitability of existing wells, hydraulic characteristics of the aquifer, and other related factors, the design must determine a well spacing and layout. A preliminary review of the Fort Union aquifer hydraulic properties indicates the “bubble” of injected water around a well in the Fort Union Formation will not normally extend more than a few hundred feet from the injection well. It is anticipated that the hydraulic characteristics of the Wasatch and Lance/Fox Hills strata will similarly result in the extent of injected water around each ASR being limited to a few hundred feet.

If existing wells are to be used, each ASR well may therefore operate on its own “bubble” of injected water. If a number of ASR wells can be sited fairly close together, such that the bubble of injected water around each well coalesces with those around other ASR wells, the recovery efficiency of the project may be improved. This may not be as important in the Fort Union aquifer where the native groundwater quality is acceptable in much of the aquifer.

The final design must also take into consideration how operation of the ASR and non-ASR wells operated by the City of Gillette and by other non-incorporated communities in the same area may affect recovery of the injected recharge water around the ASR wells. For example, operation of a nearby non-ASR production well may draw the bubble of water around an ASR well off to one side, thus decreasing the recovery efficiency of the ASR well. Similarly, unbalanced injection and recovery operations between ASR wells may move the bubble of stored water away from one well, resulting in high recovery efficiency at one well and low recovery efficiency at another well. Accordingly, the design for final expansion of an ASR well field must take into account a number of site-specific factors in order to achieve the best possible results.

8.2.3.8 Regulatory Requirements

To pump water from the Madison aquifer to the Fort Union aquifer, the WYDEQ requires a Class V injection well permit. Class V wells are injection wells or systems that are used to dispose of non-hazardous wastes directly into or above underground sources of drinking water (USDWs). The Safe Drinking Water Act (SDWA) is designed to protect the quality of drinking water in the Unites States, and Part C specifically mandates the regulation of underground injection of fluids to ensure that such injection does not endanger USDWs. The Underground Injection Control (UIC) Program was established in 1982 when Congress passed the Safe Drinking Water Act and the US EPA has promulgated a series of UIC regulations under this authority.

The minimum requirements for a Class V injection well have been set by Volume 40 of the Code of Federal Regulations (40 CFR) Sections 144-147 and in Chapter 16 of the WYDEQ Water Quality Division Rules and Regulations as promulgated within Wyoming Statute 35-11-101 through 1413. Although Wyoming is a primacy state under both Section 1422 and Section 1425 of the Safe Drinking Water Act, under Section 1422, the WYDEQ (rather than EPA) has been delegated the authority to regulate Class I, III, IV and V UIC facilities.

There are different types of Class V well permits. To inject water from the Madison well formation to the Fort Union well formation and use the injected water as drinking water is considered beneficial use the injected water and the type of permit required is a Class V, Type 5B2, “Aquifer Recharge Facilities” permit. Aquifer Storage/Recovery (ASR) is the process of

injecting water into an aquifer for storage and subsequent recovery for beneficial use, using the same well. Beneficial uses include, but are not limited to, public drinking water (as is the case under this alternative), agricultural uses, future recharge, and industrial uses.

Although Chapter 16 of the WYDEQ Water Rules and Regulations, Appendix B lists the type of permit for a 5B2 permit as "Permit by Rule", per discussions with John Passehl, Program Principal, WYDEQ's Underground Injection Control Program, he indicated the permit WYDEQ would require would be an "Individual Permit" based on the probable magnitude of the area that would be effected and that WYDEQ would want more control during the permit process, such as public notice, etc.

8.2.3.8.1 Permit Submittal Requirements

John Passehl indicated that to obtain the permit, a lot of work would be required up front. John Passehl indicated that the WYDEQ would require the submittal of detailed chemical analysis showing how the two waters (from the Madison and the Fort Union formations) would react/blend, for review and approval as part of the permit application process. John Passehl indicated that the difference in some of the water quality parameters, such as the Madison formation being much higher in hardness than the Fort Union formation, would not likely stop the WYDEQ from approving the application, as long as the discharge into the aquifer does not result in the range of any parameter or concentrations of constituents in excess of the maximum allowable concentrations of the Primary Standards of the SDWA, at any place or places of withdrawal or natural flow to the surface.

An "Individual Permit" requires the following:

- 30 days public notice;
- The class of the use of the receiving aquifer's water (in this alternative, the Fort Union water) and the water to be injected (in this alternative, the Madison water) must be fully characterized;
- A calculation of the area of review be submitted which should not be less than the area of potentially impacted groundwater. All areas of review shall be legally described by the township, range and section to the nearest 10 acres as described by township as described under the general land survey system. The calculation of the area of review determines the total amount of void space around and down

gradient from the point of injection and uses accepted ground water theory to determine the extent of any affected groundwater around the facility;

- Construction and engineering details in accordance with Section 10, Chapter 16 and Chapter 11, WYDEQ's Water Quality Rules and Regulations;
- Information, including the name, description, depth, geologic structure, faulting, fracturing, lithology, hydrology, and fluid pressure of the receiver and any relevant confining zones. The fracture pressure of the receiver shall be submitted only if the injection is under pressure into a confined aquifer. John Passehl indicated the WYDEQ does not allow injection at a volume or pressure that fractions the formation. He indicated that typically only a surface injection pressure is allowed which could limit the volume injected.
- The volume of water injected and the maximum daily injection volume must be reported when applying for coverage under a permit. Class V may inject any volume of water as long as the pressure of injection is controlled to prevent the receiving formation from fracturing;
- Topographic and other pertinent maps;
- The facility and each of its intake and discharge structures;
- Other wells, springs, surface water bodies, and drinking water wells listed in public records or otherwise known to the applicant within the area of review;
- Bedrock and surficial geology, geologic structure, and hydrogeology in the area;
- A list of other relevant permits, whether federal or state, that the facility has been required to obtain, such as construction permits;
- And detailed plans for monitoring the volume and chemistry of the discharge, and water quality of the selected water wells within the area of review.

Within 60 days of submission of the application to WYDEQ, the administrator is required to make an initial determination of completeness. During any 60 day review when an application is deemed complete, the administrator can issue a draft permit of issuance or denial. If the application is denied, the application is appealable by the application to the Environmental Quality Council.

John Passehl indicated there are no permits in the State of Wyoming similar to the alternative being evaluated (to pump water from the Madison formation to the Fort Union formation). He indicated a permit like this is unusual because it usually does not make economic sense to

inject/pump water from aquifer to another and then pump the water again (essentially twice) for treatment. John Passehl indicated the only time he has seen this permit used before is for water produced from the coal bed methane gas wells. He indicated that since the WYDEQ has created a separate classification for water produced from coal bed methane gas wells, he is no longer aware of any permit used for this purpose.

8.2.3.8.2 Permit Monitoring and Operation Requirements

The injection well(s) would be required to be operated and maintained in a manner that is compliant with the standards for the permit. If for any reason, injection activities violate groundwater standards, injection will not allowed under the "Individual permit."

John Passehl indicated that WYDEQ would require quarterly grab samples of the injected well water be taken and tested for parameters that characterize the suitability of groundwater as a drinking water supply. These parameters are the same parameters the City of Gillette has been testing their existing water supply for. The well water would need to be tested for the following parameters, as listed under the SDWA: inorganic contaminants, synthetic organic contaminants, radioactive contaminants, volatile organic contaminants, microbiological contaminants (coliform and bacteria), fluoride, sodium, nitrate (as nitrogen), aluminum, arsenic, barium, cadmium, chromium, copper, iron, lead, manganese, mercury, selenium, and zinc.

As per Chapter 16, all permits shall be for no more than ten (10) years duration. Each permit shall be reviewed by the department at least once every five (5) years for continued validity of all permit conditions and contents. John Passehl indicated there is no guarantee any permit will be renewed or denied and this could happen at any given time if WYDEQ desires.

Feasibility of underground injection as a tool for the City of Gillette's drinking water involves several technical considerations including geologic, economic, and engineering questions. These may vary significantly by operator and location. There are however, a common set of questions that must be answered for any proposed injection well, including:

- Formation Suitability: Selection of a suitable injection zone which may potentially include evaluating depth, relative location to producing wells and locally producing

aquifers, significance of local fracturing and faulting, condition of active an abandoned wells within the area, as well as other artificial penetrations.

- Isolation: The receiving formation must be vertically and laterally separated or otherwise confined from other USDWs. The well must also be equipped to isolate the receiving zone from other porous zones in the well to avoid unauthorized fluid movement in zones that are not permitted for injection.
- Porosity: Porosity is the percentage of void spaces or openings in a consolidated or unconsolidated material. Reservoir rocks are typically high in porosity, while confining zone rocks range from high to very low porosity.
- Permeability: Permeability is defined as a measure of the relative ease with which a porous medium can transmit a liquid under a potential gradient. A reservoir rock will have sufficiently high permeability to allow fluid movement. Confining zone rocks will have very low permeability and act as barriers to flow.
- Reservoir Pressure: The reservoir pressure is the static pressure within the receiving formation expressed either as psi or fluid head. Reservoir pressure may limit the rate at which the water can be injected and/or may limit the total volume of water that can be injected.

8.2.3.8.3 Treatment Requirements

When injection is considered using Class V type wells for beneficial uses, pre-treatment of the produced water may be required before it is injected into an aquifer for either recharge or ASR. Per discussions with John Passehl, Program Principal, Underground Injection Control Program, WYDEQ, John Passehl indicated that at a minimum the WYDEQ would require bacteria testing of the water from the Madison formation and would require disinfection of the water injected if analysis shows that coliform bacteria, sulfate reducing bacteria, or iron fixing bacteria are present in the water. Since the water from the Madison formation would be brought to the surface and then would need to be conveyed through a pipeline for approximately 40 miles, John Passehl indicated it is likely bacteria would be present in the water and the WYDEQ would require chlorinating the water before it was injected into the Fort Union formation.

8.2.3.9 Non-Technical Barriers to Implementation

Owners of local water wells and springs may protest permits. The WYDEQ provides public notice for all Class V permit applications and places notice in newspapers for general circulation. Any citizen is free to protest an application. If a protest is submitted a public hearing is then scheduled for all parties to present their technical merits. The WYDEQ will then decide what action to take relative to the permit based on the merits of the application. Opponents have the ability to delay applications by way of protests, appeals, and lawsuits. John Passehl, Program Principal, Underground Injection Control Program, WYDEQ indicated that people with domestic wells may have opposition to the water being injected into the Fort Union. If the opposition is great, this permit may not be approved and this alternative may not be an option. It is not known for sure what would happen until the application is submitted.

8.2.3.10 Capital Components

For the purpose of comparison to the other alternatives, a number of assumptions were made and a cost estimate was prepared. For supply under this alternative, the City of Gillette would have to build a pump station and supply line to pump the water transferred from the Madison formation to the Fort Union, when needed. Pumping 24 hours a day in 2037 requires a pipeline to carry approximately 1,500 gpm. This is the minimum design flowrate for the supply line. A 24” line was chosen to allow more pumping flexibility and some extra capacity. The water was assumed to be supplied from the intersection of 4J Road north of Southern Drive and Southern Drive. This point was chosen as it is already fed by three 12” mains, and it is proposed to be upgraded with the in-town improvements portion of this alternative. The pump station could be located anywhere that is convenient along Southern Drive.

Similar facilities would be needed to deliver CBM supply water to the proposed ASR wells. Given the unknowns surrounding CBM water at this time, a cost estimate was not prepared for this alternative; however, it is considered to be on the same order of magnitude as the Gillette connection and pump station.

The ASR well field was developed based on the general layout previously performed by Wester-Wetstein on the Southern Well Field project, with the field expanded to 45 wells. The collection system and transmission systems were expanded to provide the upsize in delivery capacity.

The pump station and clearwell design and layout from the surface water treatment alternative was used as the required pump station here has substantially the same storage and head requirements.

The transmission main into town was assumed follow a similar route to that laid out in the Southern Well Field concept. A 42" main was used to convey this flow similar to the other alternatives. The main would be connected to reservoirs Z1-R4 and Z1-R5.

In-town improvements consist of a new 24" backbone in 4J road north to the Z1-R3 reservoir. This would serve as an interconnect between the existing Madison source and the new ASR source. Future large diameter supply lines from reservoirs Z1-R4 and Z1-R5 are anticipated as development to the south and its associated demands grow. Due to the unknown timeframe and exact nature of this development, these improvements were not included in the cost estimate.

8.2.3.11 Cost Estimate

The following capital cost was developed for the ASR alternative. Detailed cost estimates can be found in Appendix A.

Capital cost for the ASR Alternative: $145,606,000

8.2.4 Coal Bed Methane Water

Due to the many unknowns surrounding CBM water, no alternatives have been developed exclusively for its use. However, surface water impoundment or ASR concepts with CBM as a source are very similar to the Madison Formation supplied versions of these alternatives explored above. For the purpose of developing a preferred alternative, relative costs between alternatives should be similar with CBM source water and Madison Formation source water.

The unknowns at this time with the CBM source water prevent it from being considered as a preferred alternative in this report. However, as discussed above, the Madison Formation supplied alternatives are very close in concept to what would be required with CBM as a long-term primary source. In fact, the storage alternatives were developed with the general concept

that they could be used with either Madison Formation or CBM water. If the issues surrounding CBM can be worked out, it could be used to simply replace, or partially replace the Madison Formation source water in either of these scenarios. This could potentially be a benefit to Gillette as they may be able to reduce their Madison pumping costs, and it could also be a positive use of CBM water and a benefit to local CBM operators.

Further, according to a CBM operator representative, this option ***is not*** a 30-year source. ***Any planning efforts relative to CBM water should be firmly based on the premise that it is a short-term or secondary source at best.***

The above discussion notwithstanding, the development of CBM should not be viewed as a alternate track to be pursued at the expense of neglecting the Madison Formation source. Gillette's water needs are significant and immediate, and any delays to implementing this project will result in severe penalties to the Gillette water situation. If such delays could possibly save a significant amount of money, they should be pursued. However, in this situation, the savings are minimal and are definitely outweighed by the pressing water demands now and in the future. Accordingly, the best course of action for implementing CBM water is to plan for the Madison Formation as the known and reliable source water with provisions to add CBM as a source when the conditions are right and the issues resolved.

8.2.5 Summary of Cost Estimates for Long-Term Alternatives

The following summary shows the capital costs of each of the long-term alternatives. Operation and Maintenance costs, non-economic factors, funding strategies, and rate structures are all important tools in determining a preferred alternative when used in conjunction with capital pricing. These items can be found in Chapters 9 and 10.

New Madison Well Field and Transmission Main: **$159,393,000**

Surface Water Impoundment of Madison Water with a Surface Water Treatment Plant and New Transmission Main: **$101,103,000** (9 MGD of Treatment)
$114,834,000 (18 MGD of Treatment)

Groundwater Storage and Recovery of Madison Water in the Fort Union Aquifer:
$145,606,000

8.2.6 Present Worth Analysis

A present worth analysis has been prepared for the Madison Well Field and Transmission Main and Surface Water Impoundment with Surface Water Treatment Plant. The Groundwater Storage and Recovery alternative was not included as its risks make it infeasible; consequently, it was not included in this level of study. The initial present worth analysis was based on a discount rate of 4.875% based on the current Federal Government project rate, and an annual construction inflation rate of 5%, which is based on RS Means historical cost data over the last 50 years. Salvage values were based on construction costs indexed to the time of replacement and useful lives were generally based on Bureau of Reclamation recommended useful lives. The only exception to this is the useful life of the pipelines on the project, which was set to 60 years to match the assumed useful life of the existing Madison infrastructure.

At these initial rates, the surface water impoundment and treatment plant had the lowest present worth (cost). However, once a sensitivity analysis was performed, it was noted that the outcome of this analysis is highly sensitive to changes in both the discount rate and the annual construction inflation. For instance, a 1% change in the discount rate creates a shift in present worth of $12.6 million, or approximately $1.6 million per 1/8 percent change. A similar sensitivity can be seen to construction inflation rates. A 1% change in construction inflation causes an $11.7 million shift in present worth, or about $1.5 million per 1/8 percent change. The initial present worth calculation was based on a 4.875% discount rate and 5% annual construction inflation. See Table 8-20 below for a summary of the results of this analysis. Complete calculations are included at the end of Appendix A of this report.

Table 8-20

Present Worth Sensitivity Analysis

Interest Rate	Surface Reservoir	New Well and Pipeline Alternative
4%	-$76,732,979	-$70,189,507
4.5%	-$80,395,864	-$80,624,952
4.875%	-$82,605,954	-$87,310,640
5%	-$83,251,466	-$89,343,678

*Construction Inflation Held at 5%/Year

Construction Inflation	Surface Reservoir	New Well and Pipeline Alternative
4%	-$88,068,411	-$101,670,196
5%	-$82,605,954	-$87,310,640
5.5%	-$78,794,906	-$78,031,968
6%	-$74,113,261	-$67,081,188

Discount Rate Held at 4.875%/Year

Less favorable present worth
More favorable present worth
Both alternatives comparable

Once this sensitivity analysis was completed, it can be seen that even small departures from these rates can cause a change in which alternative has the most favorable present worth. With the present worth results coming out so close, it was determined that in this respect the alternatives were substantially similar. Without a very definite funding package with amounts; borrow dates; and loan specifics including interest rate, closing costs, origination fees, and term the present worth of the alternatives is so close that it can not be used as a determining factor in selecting a preferred alternative.

8.2.7 Time Indexed Costs

Costs have been indexed with a 5% annual construction inflation factor to the anticipated time of their expenditure for each alternative. The timing and anticipated expenditure amount for each year can be seen in the present worth calculations discussed above and include in Appendix A.