Spokane Valley-Rathdrum Prairie Aquifer Optimized Recharge for Summer Flow Augmentation of the Columbia River

Submitted to:

Washington State Department of Ecology
Office of Columbia River
Yakima, Washington

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April 1, 2011

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EXECUTIVE SUMMARY

The Spokane Valley-Rathdrum Prairie (SVRP) aquifer and the Spokane River interact continuously from northwestern Idaho into northeastern Washington. Saturated hydraulic conductivities are extremely high; ranging from 5 ft/d to 22,100 ft/d. This allows water to almost flow freely between the river and aquifer. Large amounts of aquifer pumping and climate change impacts have decreased summer low flows in the Spokane River causing concern and sparking an interest in determining a plan to recharge both the river and the aquifer.

Using Visual MODFLOW with the regionally-approved 1990-2005 MODFLOW-2000 model data, a comprehensive aquifer recharge and natural recovery feasibility study involving two water sources, multiple injection sites, and timing considerations was conducted with withdrawals occurring during periods of excess river flows in the Spokane and Pend Oreille watersheds. One of the primary project constraints involved the influence of injection on flows in the Spokane River. The optimized artificial recharge was designed to improve low flows in the months of August, September, and October. To determine the well injection location that produced returns in the Spokane River during this time frame wells where placed on lines running nearly parallel to flow in the aquifer at various distances from the Spokane River. Three existing right-of-ways (2 railroads and 1 power line) were used as potential routes for the distribution system (extraction to injection). Locations of the sites are shown in Figure A. These routes were selected based on minimizing disruption to land use but other routes would likely be acceptable. Injection rates were varied between 25 ft^3/s up to 300 ft^3/s.

Using the US EPA pipe network model, head losses and pumping requirements were calculated for various configurations of the distribution network. Primarily injection well location, extraction point, well diameter, and pipe diameters were selected as the design variables. Trade-offs between pipe and well diameters and head requirements were documented for economic consideration. Price estimates were obtained using a combination of published information, local quotes, and bid documents from the region.

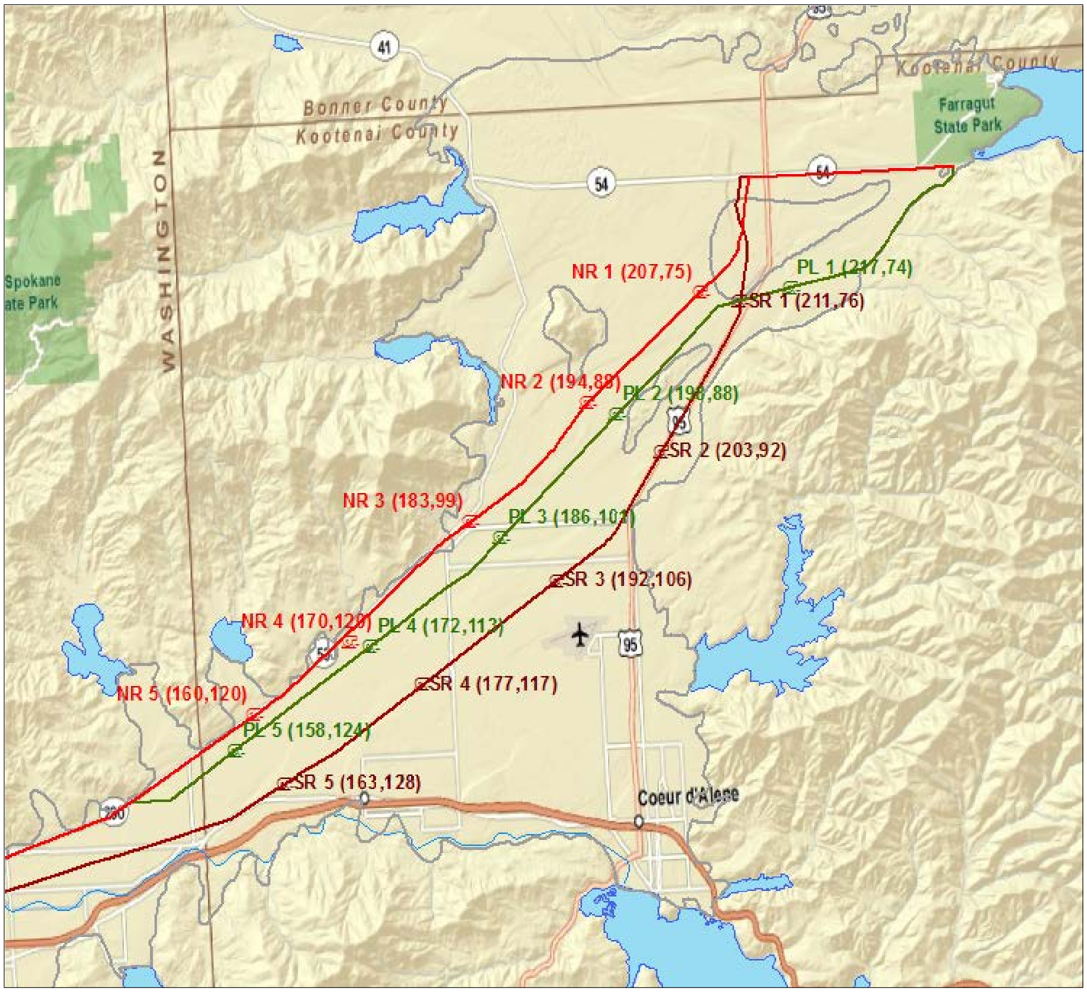

Figure A. Potential injection locations.

MODFLOW modeling results showed increases in head by artificial recharge produce increased flows into gaining reaches and decreased flow out of losing reaches. Timing was impacted by distance from the stream. Figure B illustrates a typical example of the responses provided by the different injection rates. Surface water diversions from the Spokane River proved to be problematic due to excessive treatment costs and groundwater extractions from the Washington side of the aquifer to the injection sites created large depressions that had to fill prior to any river benefit. Therefore, the optimum solution was to take water from the Lake Pend Oreille area during high flow periods. This increases the net recharge already occurring from that area. Indications are that improved stream flows would also enable Avista to generate additional hydropower during low flow months to offset some of the cost of pumping during high flow periods.

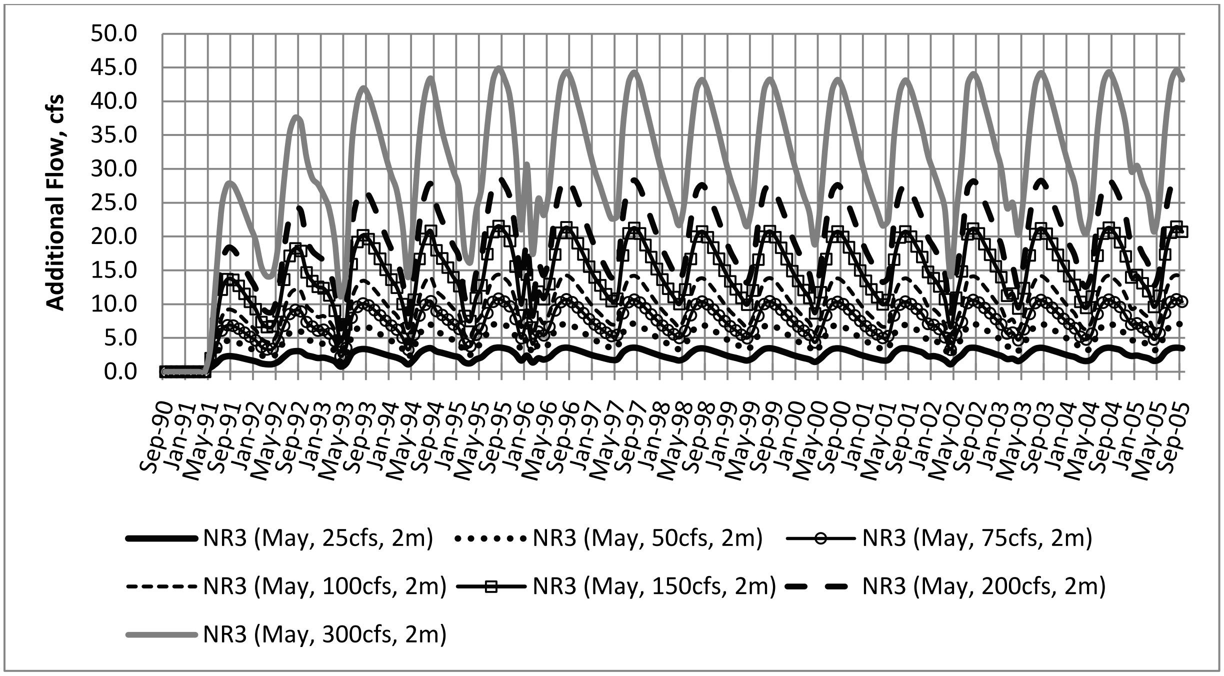

Figure B. Comparison of rate increases caused by injection at NR3.

Model results and economic analysis indicated that favorable river flow results can be achieved, although the price of water will be above existing retail costs. The two best alternatives involve 300 ft^3/s of extraction/injection via a 72-inch pipeline for four months (April-July) originating from near Lake Pend Oreille and terminate near the intersection of N. Ramsey Road and E. Diagonal Road (NR2) (approximately 2.5 miles west of Garwood, ID) or at Rathdrum, ID (NR3). Termed LPO-NR2-72-18-300 in the report to represent the source, injection location, pipeline diameter, well diameter, and flow rate, this injection location resulted in a shorter distribution line and a somewhat longer travel distance to the stream. The increased travel distance allowed water to spread out more over the year than did sites closer to the river so the peak is less than injection sites closer to the river but the base is broader.

Because of the acute demand for water in the watershed and the economy of scale demonstrated by the study results, longer pumping with high discharge rates are more cost effective than short pumping periods or low flows. Consequently, there are two alternatives that stand out above other options. Scenarios LPO-NR2-72-18-300 and LPO-NR3-72-18-300 both appear to the best of the options considered in terms of cost per acre-ft delivered to the Spokane

River. On average, LPO-NR2-72-18-300 provides 4,573 acre-feet of flow during August and 13,473 acre-feet during August-October for costs of $3,461 and $1,175 per acre-foot, respectively. Likewise, LPO-NR3-72-18-300 provides 5,062 acre-feet of flow during August and 14,527 acre-feet during August-October for costs of $4,132 and $1,440 per acre-foot, respectively. So, while NR3 provides 10.7% more August flow and 7.8% more August-October flow, the costs are 19.4% and 22.6% higher, respectively. While, it is tempting to convert the flow volumes into ft^3/s (e.g., 4,573 AF = 74.4 ft^3/s), the monthly time step makes such conversion somewhat problematic. These values represent average conditions not instantaneous quantities on a specific day.

The region is projected to need this much water and perhaps more to offset growth and climate change issues so the question regarding preferred alternative boils down of the publics' willingness to pay for water. Overall it appears that the LPO-NR2-72-18-300 provides a considerable amount of water (nearly 90% of the NR3 site) at a construction cost of nearly $90 million compared to NR3 site where construction costs are estimated at $122 million. For this reason, it appears that the NR2 site would be the preferred alternative.

Given the SVRP's designation as a sole source aquifer, there may be few alternatives for increasing water supply to the region. Should conservation efforts fall short of future demand, this project appears to provide a technically viable option for enhancing summer flow conditions in the Spokane River and points further downstream.

ACKNOWLEDGEMENTS

The research team at Washington State University would like to thank John Covert, Hydrologist, and Guy Gregory, Senior Hydrogeologist and Technical Unit Supervisor, in the Water Resources Program at the Washington State Department of Ecology in Spokane, WA for their review comments aimed at making this a more thorough investigation. In addition we would like to acknowledge the contributions of Ms. Suzanne Cox in developing cost estimates related to water treatment and Dr. Barry Green for his review of entire report.

Funding for this study was provided by the Washington State Department of Ecology, Office of Columbia River, Yakima, WA through a grant to the State of Washington Water Research Center at Washington State University.

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1.0 Introduction

Demand for water continues to rise throughout the Columbia River Basin with population and economic growth as well as increased awareness of ecosystem requirements primarily related to endangered fish species. At the same time, global climate change is reducing the effectiveness of snowpack storage by producing more winter runoff when demands are low and less summer flow when demands are high. The combined impacts of growth and climate change will require improved mechanisms for storage and management of water resources. Storage options include on-channel and off-channel dams as well as temporarily recharging groundwater aquifers for subsequent withdrawal (referred to as Aquifer Storage and Recovery or ASR). While traditionally on-channel reservoirs have been used to provide storage, concerns over fish passage and other environmental impacts has recently led to consideration of off-channel options including dams on dry gulches and ASR.

ASR typically involves injecting water into an aquifer through wells or by surface spreading and infiltration and then pumping it out when needed. ASR has been successfully used around the world as a technique for storing water during high discharge periods for use in later periods. Brown et al. (2006) studied fifty ASR projects from Africa, Australia, England, India and the United States and found that most were able to successfully meet their design objectives although problems with injection well clogging and geochemical reactions were occasionally noted. Mirecki et al. (1998) reported dissolution of a limestone/clastic aquifer that increased permeability over time and caused elevated chloride levels. WAC 173-157 establishes standards for review of ASR proposals and mitigation of any adverse impacts in the following areas: aquifer vulnerability and hydraulic continuity, potential impairment of existing water rights, geotechnical impacts and aquifer boundaries and characteristics, chemical compatibility of surface and ground waters, recharge and recovery treatment requirements, system operation, water rights and ownership of water stored for recovery, and environmental impacts (Ensenat 2003).

Another approach is to provide recharge to aquifers in places where passive discharge will optimize stream flow at critical periods. This concept of optimized recharge does not require physical capture an production of stored water, rather, additional recharge is placed at a location optimized so natural discharge processes will be augmented at a specified time and rate. Such a system avoids high capital and societal costs of surface reservoirs or impoundments and associated treatment systems.

The Spokane Valley–Rathdrum Prairie (SVRP) aquifer is an ideal potential candidate for aquifer storage and recovery (ASR) since the aquifer naturally drains back into the Spokane and Little Spokane Rivers as long as the groundwater levels are above the bottom of the streambed. Unlike typical ASR projects that require energy to retrieve the water, the SVRP ASR Project would rely on gravity drainage into the stream.

The proposed development of large co-generation facilities in 2001 along Idaho-Washington state-line border threatened critical groundwater supplies coming into Washington and focused discussion on the implications and responsibilities of sharing an important joint resource. Consequently, a regional MODFLOW model of the aquifer was developed cooperatively between the States of Idaho and Washington, the US Geological Survey (USGS), the State of Washington Water Research Center (SWWRC), and the Idaho Water Resources Research Institute (IWRRI) (Hseih et al. 2007). The model used a combination of newly collected data, historical information, refined analysis, and group consensus to create a calibrated, transient MODFLOW model of historical conditions on the SVRP. Model results indicate future summer groundwater withdrawals would impact the return flows into the Spokane River and, to a lesser extent, the Little Spokane River. Potentially, groundwater withdrawals could lead to flows less than those proposed as minimum instream flows in Washington and subsequently reduce the amount of flow downstream in the Columbia River.

Using the bi-state groundwater model, Barber et al. (2009) demonstrated the concept of using the aquifer's natural attenuation characteristics to mitigate summer low flows in the Spokane River without specifically identifying the source of the additional water. USGS gages located on the Spokane River in Spokane and on the Pend Oreille River downstream of the Lake Pend Oreille Dam (Albeni Falls) indicated that the average discharges during the peak runoff and winter months were above instream flow requirements, so high-flow diversions were likely possible. Based on the promising results of the preliminary study and the possibility of winter/spring diversions, it was concluded that a more comprehensive analysis of the potential for ASR should be conducted.

The overall objectives of this project were to investigate the technical and the economical feasibilities of developing an optimized recharge project on the SVRP. Such a project could benefit regional needs for additional water resources by mitigating the effects that well pumping of new or currently authorized withdrawals have on late summer stream flow in the Spokane River. This involved using existing information to evaluate potential diversion amounts, using the bi-state model to examine various combinations of location and pumping period for injection of water to maximize summer low flow benefits, and determine approximate costs for the alternatives, including the infrastructure needed for extraction wells, the distribution pipeline, and injection facilities. Rather than direct surface water diversions, potential extraction well fields near the Spokane River and Lake Pend Oreille were investigated. Well water typically requires less treatment before it is discharged and may alleviate any concerns regarding potential aquifer contamination as well as minimize the treatment costs associated with meeting anti-degradation criteria. However, the costs of surface water treatment were also estimated in order to cover the range of potentially viable solutions. Pipeline routes for the distribution system were chosen to follow existing right-of-ways to the maximum extent possible. Although funded through the Office of the Columbia River, this report assumes any benefit to the Columbia River itself is subordinate to the benefits of flow mitigation for the SVRP region.

To facilitate analysis and evaluation of the SVRP ASR Project, this report examined the following key technical aspects:

- Chapter 2: Needs Assessment (Task 1)
- Chapter 3: Water Availability Assessment (Task 2)
- Chapter 4: System Limitations (Task 3)
- Chapter 5: Target Design Objective (Task 4)
- Chapter 6: Alternative Evaluation (Task 5)
- Chapter 7: Cost Analysis (Task 6)
- Chapter 8: Benefits Analysis (Task 7)
- Chapter 9: Conclusions and Recommendations (Task 8)

1.1 Study Area

The aerial extent and impact of the SVRP ASR project can be seen in Figure 1. It includes sections of the Spokane, Pend Oreille, and Columbia Rivers. While the primary study focus is on the storage potential of the SVRP aquifer, diversions and return flows have the potential to impact streamflows downstream of the system. While technically this would include the mainstem Columbia all the way to the Pacific Ocean, most of the analysis performed in this study conclude at Grant County PUD’s Priest Rapids Reservoir.

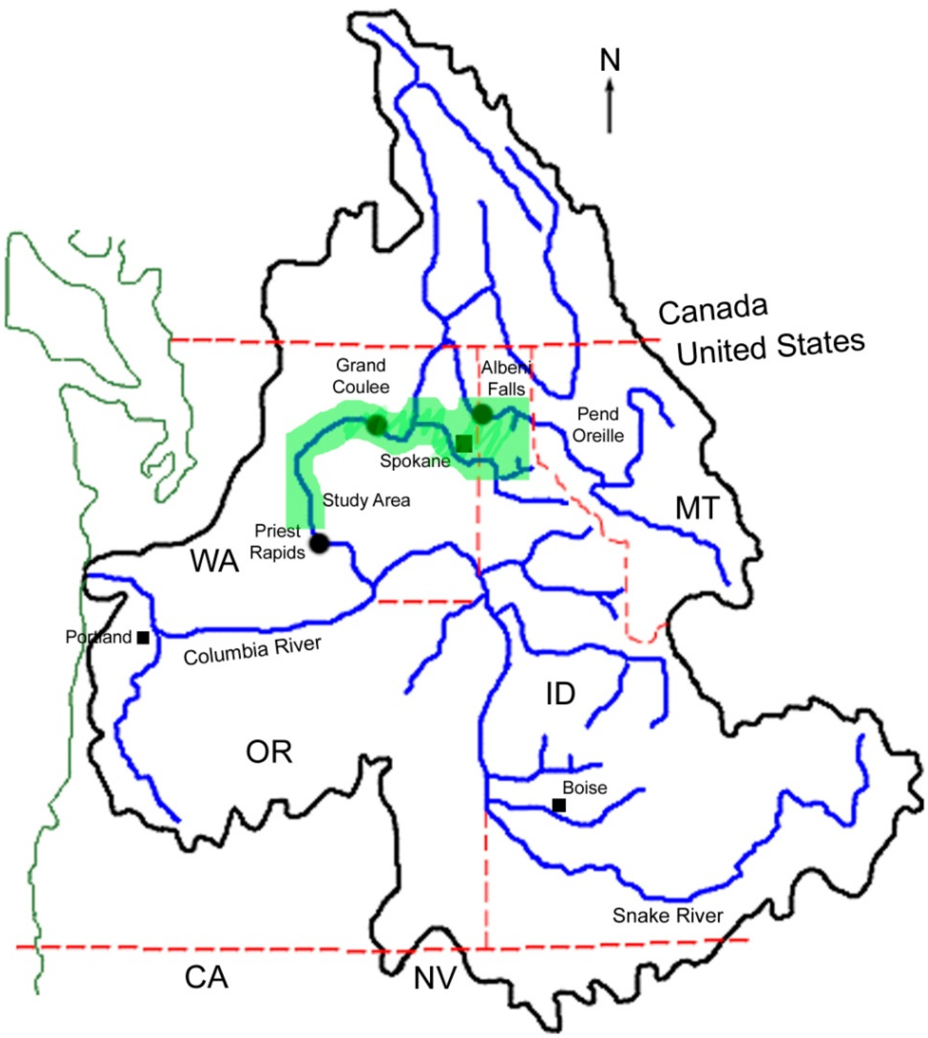

Figure 1. Study Area and Aerial Extent of SVRP ASR Project

2.0 Needs Assessment

This chapter examines factors associated with the purpose, the demand, and the operational criteria such as the time of arrival at several locations along the Columbia River and thus establishes the basis for the study. By examining the broader questions surrounding existing and future water requirements in the Columbia Basin and addressing the economic value gained by satisfying existing constraints, the critical nature of developing new water supplies for the State of Washington is exposed.

The essence of Washington’s water resources quandary can be seen by examining flow trends and predictions at critical locations throughout the region. For example, Figure 2 illustrates the range of high flows that occur during peak runoff events in the Spokane River compared to the low flow trends. In looking at over 100 years of data, water diversions and climate change impacts do not appear to be having significant impacts on high flows compared to low flow conditions where impacts are clearly noticeable.

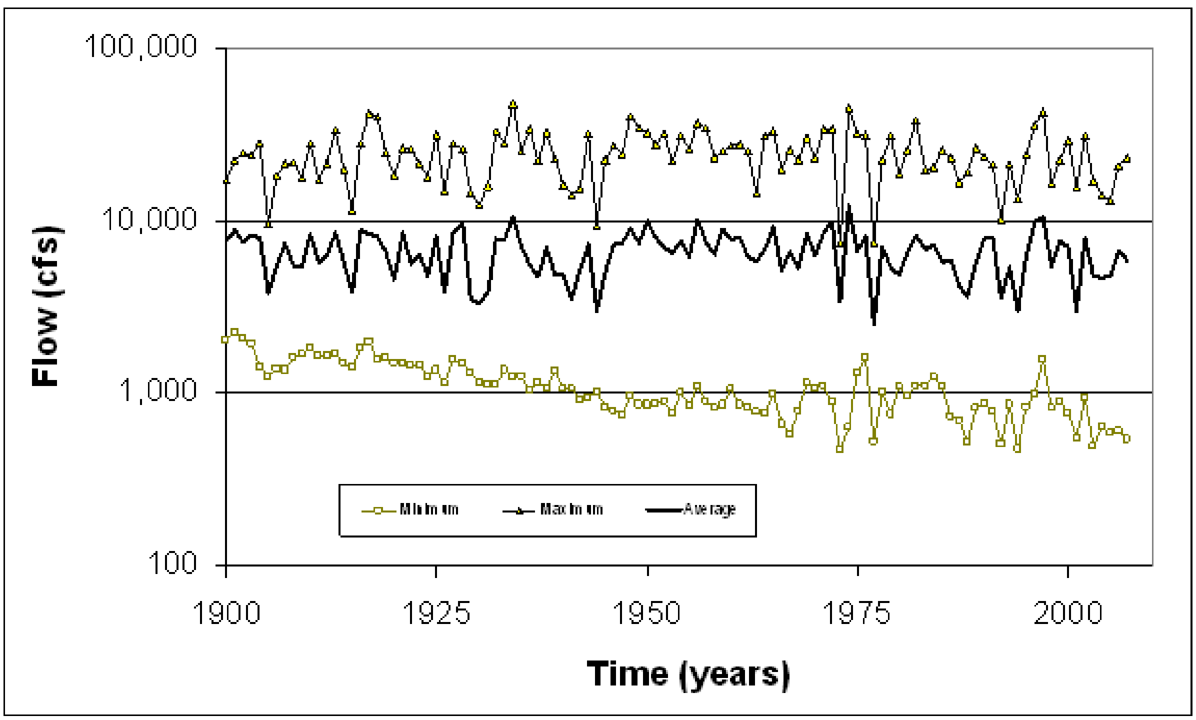

Figure 2. Average monthly stream flows at Spokane River gauge (USGS 12422500).

As illustrated in Figure 3, summer low flows at the USGS gage near downtown Spokane, WA (USGS gage 12422500) are often less than 1,000 ft^3/s, particularly in the last 40 years. It is this disturbing trend in low flows that raises concerns among water resources agencies, environmental groups, and water right holders. A regression analysis of the minimum annual daily flow data indicates a statistically significantly ($p < 0.0001$) decrease in low flow between 1900 and 2007. While the rate of decline was steepest from 1900 through 1950 (with the slope of the regression line equal to -20.477 $ft^3/s/yr$), the downward trend has still continued since that time (with the slope of 1951-2007 regression line equal to -3.315 $ft^3/s/yr$). The combined effects of changes in reservoir operations associated with the Post Falls Dam, changes in water use patterns (from irrigation of orchards and row crops to suburban residential uses), increases in municipal pumping as the regions' population has grown and changes in runoff patterns due to climate change (Fu et al, 2007) are creating severe low flow conditions that threaten water users and the environment. Prior to 1940, low flows recorded at the Spokane gage were always greater than 1,000 ft^3/s. However, since 1970, numerous occurrences of flows less than this have been observed with flows less than 600 ft^3/s becoming more frequent. This trend caused the Washington State Department of Ecology (Ecology) to essentially stop issuing new water rights in the basin; a trend that extends through much of the state. According to the University of Washington's Climate Impact Group (CIG), summer flows (June through September) in the Columbia River at The Dalles will be 30-50% less than the present flow. Although the predicted increases in winter and spring flows may compensate for annual flow totals, summer use of the water will likely require additional storage and better management. Consequently, it is important to determine whether high winter-spring flows can be used to augment lower flows. Hamlet (2001) concludes that the lack of reservoir storage currently limits options for water resources managers.

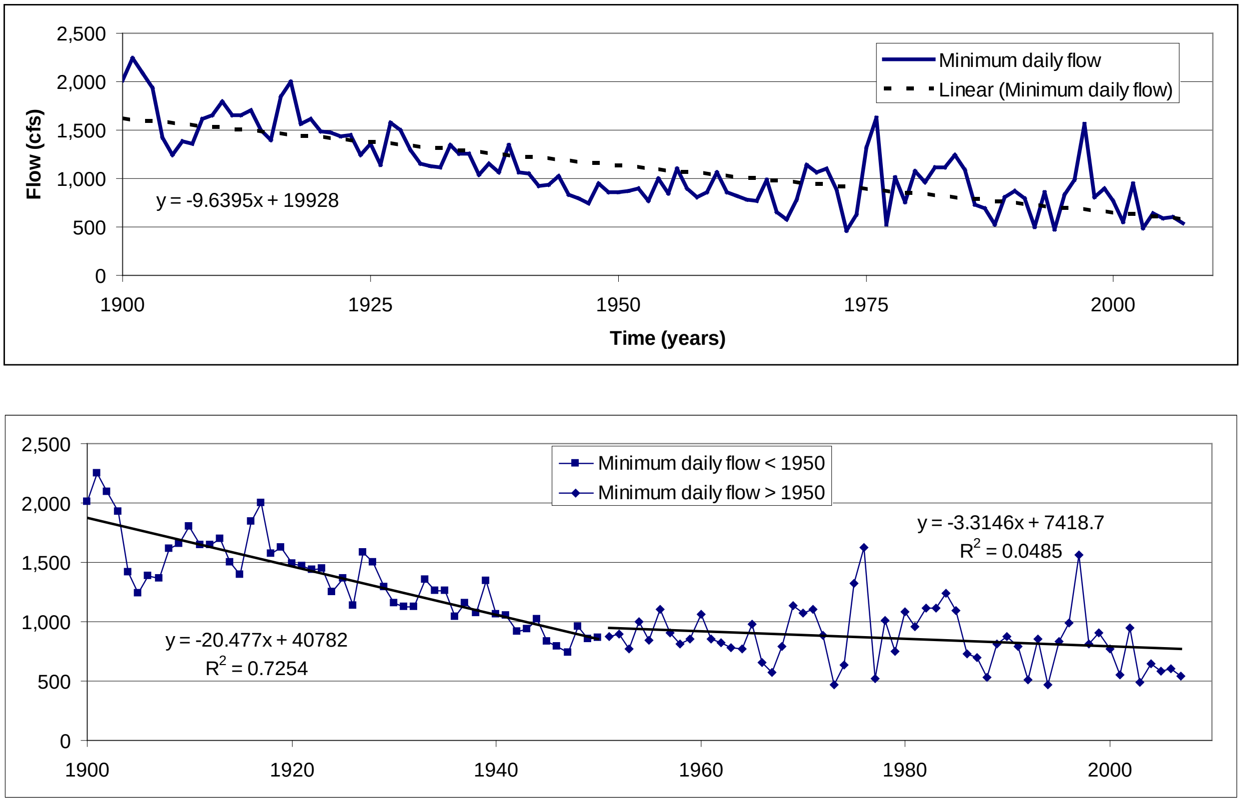

Figure 3. Long-term daily flow trend for Spokane River at Spokane gage.

2.1 Regional Demands

Water from the SVRP ASR Project would initially flow by gravity into the Spokane River at various reaches from Green Acres (Sullivan Road Bridge) to Nine Mile Dam. Simulation results indicate diminutive additional returns along the Little Spokane River. As such, the SVRP ASR project would have the potential to mitigate declining low flow conditions on the Spokane River (Figure 3 and Figure 5) arising from regional Idaho and Washington issues and future water demands.

As part of the Rathdrum Prairie Comprehensive Aquifer Management Planning (CAMP) effort, SFP Water Engineering et al. (2010) provided water demand forecasts for the Idaho portion of the SVRP aquifer over the next 50 years (to the year 2060). In a coarse attempt to factor in climate change impacts, the consulting team estimated a 10% increase in the precipitation deficit among other assumptions. This approach led to wide predicted range of water demands ranging from 77,600 acre-feet to 223,000 acre-feet (compared to the 74,000 acre-feet currently used) depending on assumed growth rates and conservation efforts. The median growth and climate impacts scenarios predict 101,000 to 163,000 acre-feet will be needed by 2060 with approximately 59,000 to 76,000 acre-feet being used consumptively (compared to the 40,000 acre-feet currently used).

The average of the moderate growth scenarios (132,000 acre-ft) is 58,000 acre-ft more than the current use. This is an annual average increase in demand across the region of 80 ft^3/s with a disproportionate amount likely to occur during low flow periods. Even assuming the historic 54% consumptive use ratio, the amount is 43 ft^3/s. A brief technical memo by Wylie (2010) shaped the annual 25,385 acre-foot consumptive use portion of the CAMP study to match historic Idaho use patterns. Using the 2005 regional MODFLOW groundwater model developed by Hseih et al. (2007), Wylie predicted the summer impact on the Spokane River would be an additional 31 ft^3/s decrease in low flow. Without knowing exactly where the increased demands would take place it is difficult to predict with certainty the precise influence on flow in the Spokane River. However, the Wylie study seems reasonable and regardless of where within the Rathdrum Prairie aquifer these withdrawals occurred, they would undoubtedly have a significant negative impact on flows into the Spokane River.

Growth on the Washington side of the SVRP may also require additional new supplies or rely on existing municipal permits not currently being fully implemented. Either way, additional aquifer withdrawals will have nearly immediate reduction in stream flow. Under a medium growth scenario the State of Washington Office of Financial Management (OFM) predicts the Spokane County population to increase by approximately 123,000 people between the 2010 estimated population and the year 2030 (http://www.ofm.wa.gov/pop/gma/projections07.asp). While some of this 26% population increase will likely occur outside the bounds of the SVRP aquifer delivery area, if even 67 percent of the population increase is served by the aquifer at a rate of 300 gallons/person/day then the additional drain on the river could be as much as 38.3 ft^3/s. While not all municipal diversions are consumptive, the WWTP discharge is downstream of the compliance point and thus the gage will likely show most of the diversion. Given the relative proximity of Spokane's drinking water wells to the Spokane River, little if any lag time would be provided by serving the increased demand, thus the impacts to the river would be nearly immediate. Moreover, it is conceivable that more than two-thirds of the growth would be within the service district of the SVRP-Washington providers which would increase drawdown of the Spokane River even more than this preliminary future demand estimate.

Climate Change Impacts

The University of Washington's Climate Impact Group (CIG) predicted streamflows at 297 locations throughout the Columbia River region based on the A1B and B1 global climate change scenarios (Hamlet, http://www.hydro.washington.edu/2860/). The A1B scenario typically predicts higher impacts because it assumes higher greenhouse gas emissions in the future than does the B1 scenario. Both have been widely cited in the literature. Bias corrected flow estimates using the Variable Infiltration Capacity (VIC) hydrologic model were generated for historic, year 2045, and year 2085 time frames. While results from both the delta change and hybrid-delta change methods are included in the CIG database, we elected to use the hybrid-delta change method as it represents composite views of 9-10 A1B and B1 emission scenarios rather than a single view. Because the A1B scenario is more conservative, we elected to focus on this information. Similar, although generally less severe, variation applies to the B1 scenario. The simulation period for the Spokane River gage in downtown Spokane station was October 1915 through September 2006. However, in order to base our analysis on more recent trends, we selected the most recent 20 year period which included flood and drought years (Water Years 1987-2006).

As illustrated in Table 1, the A1B climate change impacts to the average 1987-2006 discharges during low flow periods are significant. The 2045 estimates show decreases from July through October ranging from approximately 33 to 6 percent. The large July decreases are caused primarily by earlier spring runoff. The 2085 data show generally worsening conditions in July, August, and September but slightly improving conditions in October due to an increase in late fall-early winter precipitation. Needless to say, these streamflow decreases represent significant increases in demand if aquatic habitat is to be maintained in the watershed.

Table 1. Climate change impacts at Spokane River gage during low flow periods

Time	2045 Hybrid Delta		2085 Hybrid Delta	
	Δ flow (ft^3/s)	% change	Δ flow (ft^3/s)	% change
July	-1,029.7	-33.1	-1,270.6	-40.8
August	-368.5	-24.7	-515.2	-34.5
September	-250.3	-24.5	-325.2	-31.8
October	-84.2	-5.6	-45.1	-3.0

2.2 Columbia River Demands

Population growth, economic development, unmet agricultural demands, and wider recognition of instream flow needs compete for water from an already overtaxed Columbia River. Currently, during times of drought approximately 380 interruptible water right holders face risk of crop losses when their water use from the Columbia River is curtailed. Such losses can add up to billions of dollars in lost revenue. The Columbia-Snake River Irrigators Association (CSRIA) and Ecology entered into a voluntary regional agreement (VRA) in 2008 as authorized under ESSHB 2860 (Columbia River Bill) that would enable existing interruptible water right holders and new permanent water rights on the Columbia River and Lower Snake River to obtain drought permits. Under terms of the VRA, drought permits could be obtained provided water conservation, acquisition, storage and other appropriate actions would provide new water in a quantity sufficient to fully offset any and all new water uses during summer months (July and August on the Columbia River). However, more water needs to be obtained.

According to a National Research Council (2004) study, there are currently pending water withdrawal permit applications along the Columbia River in Washington State representing a volume of water ranging from 250,000 to 1.3 million acre-feet per year. The Department of Ecology confirms this assessment stating that over 500 applications totaling approximately one million acre-feet of water are pending within one mile of the Columbia River. Some applicants have been waiting 20 years to receive water rights because of litigation over the quantities of water necessary for instream and out-of-stream uses (Sandison, 2009). The cumulative effects and the risks to survival of listed fish species of potential future water withdrawals of between approximately 250,000 acre-feet and 1.3 million acre-feet per year were also evaluated. The NRC (2004) study concluded that allowing additional withdrawals during the critical periods of low flows and comparatively high water temperatures would increase risks of survivability to ESA listed salmon stocks and would reduce management flexibility during these periods.

Recognizing the need for additional water in the region, the Columbia River Basin water management act in 2006 directed Ecology to aggressively pursue development of new water supplies. While several promising avenues are currently being explored, additional water resources are still needed for the region to prosper.

2.3 Overview Assessment of Need

There appears to be a critical need for water throughout the Columbia River watershed during low flow months in order to address current and future demand forecasts. It is doubtful if this one project (or any one project) would be able to come close to meeting all of the existing demand let alone the additional quantities needed for growth. In the Spokane-Coeur d'Alene region, decreases in streamflow due to climate change may actually exceed future municipal demands in the SVRP aquifer system. As a result, water demand from this ASR project is likely limited by overall economics and water availability rather than demand.

3.0 Water Availability Assessment

A critical component of the SVRP aquifer storage and recovery (ASR) project is the availability of surface water for diversion during high flow periods. This chapter describes the analyses performed in order to examine potential quantities of surface and ground water supplies that might be viable for low-flow augmentation. In examining the region, it was determined that the potential sources of high-flow diversions are: 1) the Spokane River system and 2) the Lake Pend Oreille system. Therefore, both the Spokane/Coeur d'Alene and the Lake Pend Oreille watersheds were evaluated on a monthly basis to determine if potential sources of surface water physically existed and, if so, what were the potential quantities of water and when was it likely available without posing hardships on the existing ecosystem demands. Diversions from these two sources could be either direct surface water withdrawals or pumping from nearby well fields looking to exploit surface-groundwater interaction characteristics as well as river bank (lake bank) filtration for water quality treatment. The Spokane River runs through the SVRP area so pumping costs would likely be minimized. Similarly, Lake Pend Oreille is located on the northeastern boundary of the aquifer so it is geographically positioned well for a pipeline. This section of the report discusses the physical availability of surface water from the two river systems. The final selection will depend on the benefit/cost ratio provided by each alternative provided in future sections of the feasibility study.

3.1 The Spokane River System

One obvious potential source of water for the SVRP ASR Project is excess flows in the Spokane River during storm and snowmelt runoff events. Preliminary conversations with Ecology staff in Spokane indicate that winter diversions from the Spokane River are likely feasible. Although fed by upstream sources such as the Coeur d'Alene, St. Joe, and St. Maries Rivers, the Spokane River officially begins at the outfall of Lake Coeur d'Alene and flows westward into the Columbia River. The river is approximately 111 miles long and runs through the SVRP aquifer as shown in Figure 4. The distance from the river to potential injection sites is less than the Lake Pend Oreille option. Consequently, pumping and right-of-way acquisition costs are likely to be favorable compared to other potential sources. Furthermore, runoff contributions from the mountains along the Idaho/Montana border include significant snowmelt in most years creating a highly seasonal river stage. Figure 5 illustrates the variability in monthly average flow rates recorded at the USGS gage (12422500) located in Spokane. While the average and high flows appear adequate to allow additional diversion in many months, the extreme low flow conditions must also be examined. Furthermore, although Avista operates six hydroelectric power facilities on the Spokane River (Post Falls, Upper Falls, Monroe Street, Nine Mile, Long Lake and Little Falls) and the City of Spokane Water Department operates one facility (Upriver Dam), their ability to dramatically change the hydrograph timing is relatively limited due to the lack of significant storage capacity and operational goals for Lake Coeur d'Alene. Consequently, the ability to change runoff timing through reservoir operation was considered to be negligible.

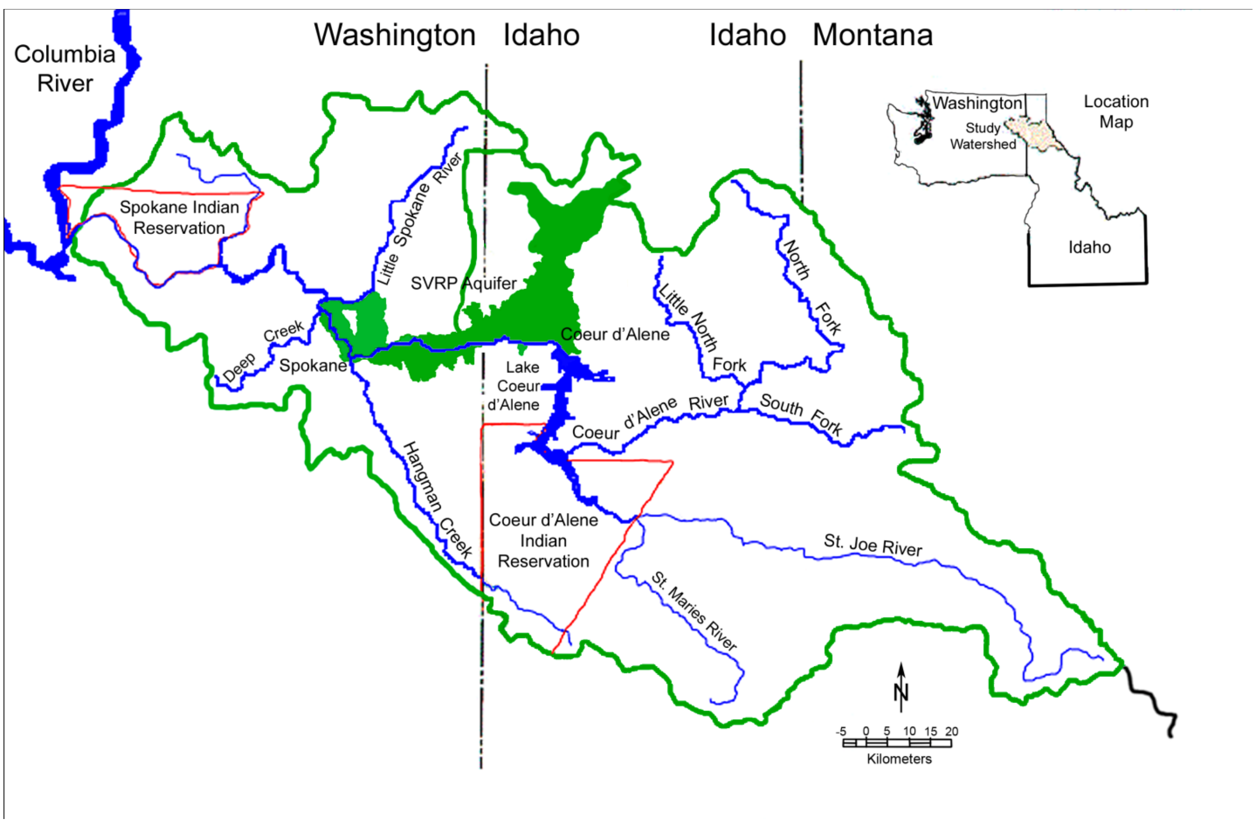

Figure 4. Spokane River watershed with Spokane Valley-Rathdrum Prairie Aquifer

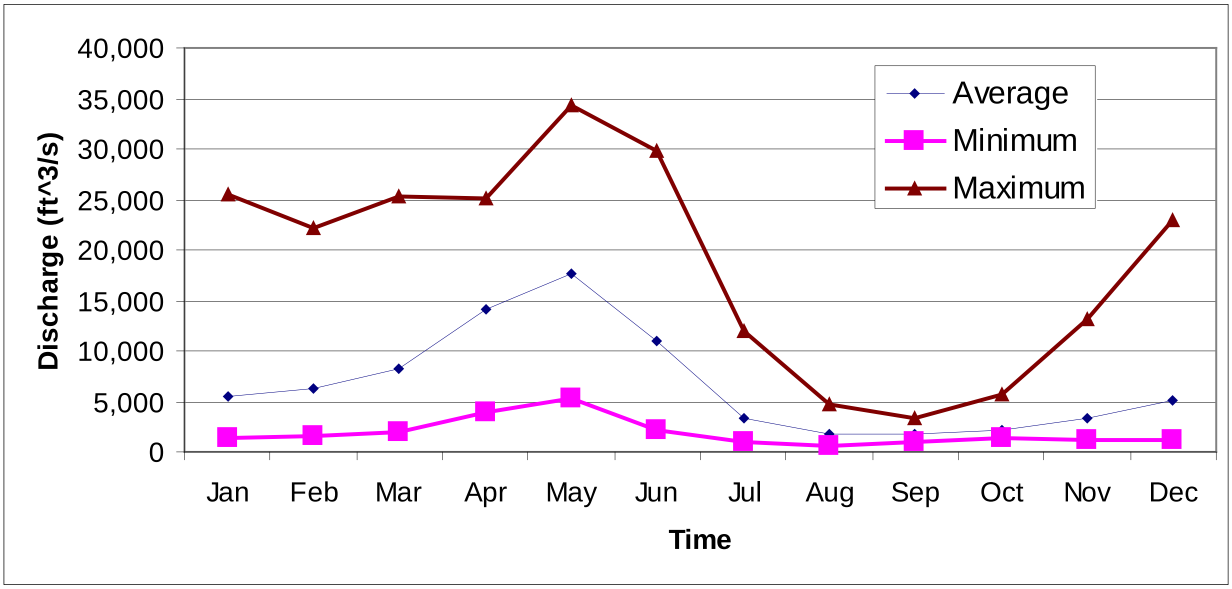

Figure 5. Average monthly discharge at the USGS Spokane gage from 1891 to 2009 (1 $ft^3/s = 0.0283\ m^3/s$).

In examining the historic flow records, we elected to investigate whether the period of record would result in any significant changes in values. Our goal was to base predictions on current operations encompassing reservoir operation, population growth, and hydrology while maintaining a significantly long record to allow meaningful statistical analysis. In discussions with Ecology, we selected a 40 year time frame (1969-2009) as the basis for comparison.

Figure 6 illustrates the discrepancies between the average monthly flows based on the entire 1891-2009 data set compared to the 1969-2009 period. Average monthly runoff in the watershed decreased April through September as depicted in the lower panel. Monthly decreases during this time span were 5.8, 10.6, 8.9, 18.1, 23.7, and 4.7 percent, respectively.

Inspection of the minimum values yielded similar but lower results in the April through September time frame although average October through December flows were nearly 37% higher (see Figure 7). Because the flows are low, large percent changes do not necessarily represent lots of volume but in this case the differences are 200, 500, and 670 ft^3/s, respectively.

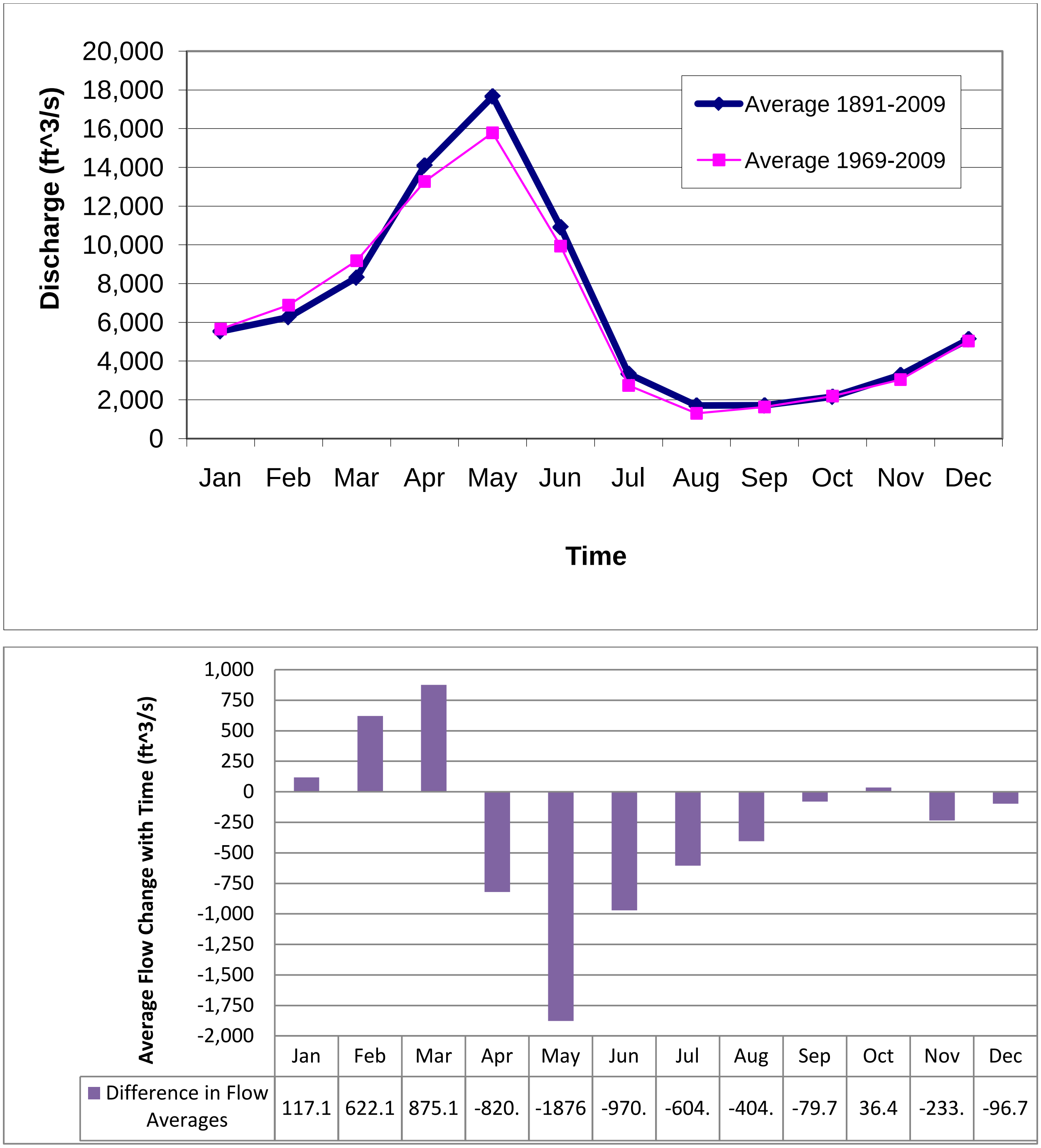

Figure 6. Comparison of average monthly flows versus period of record.

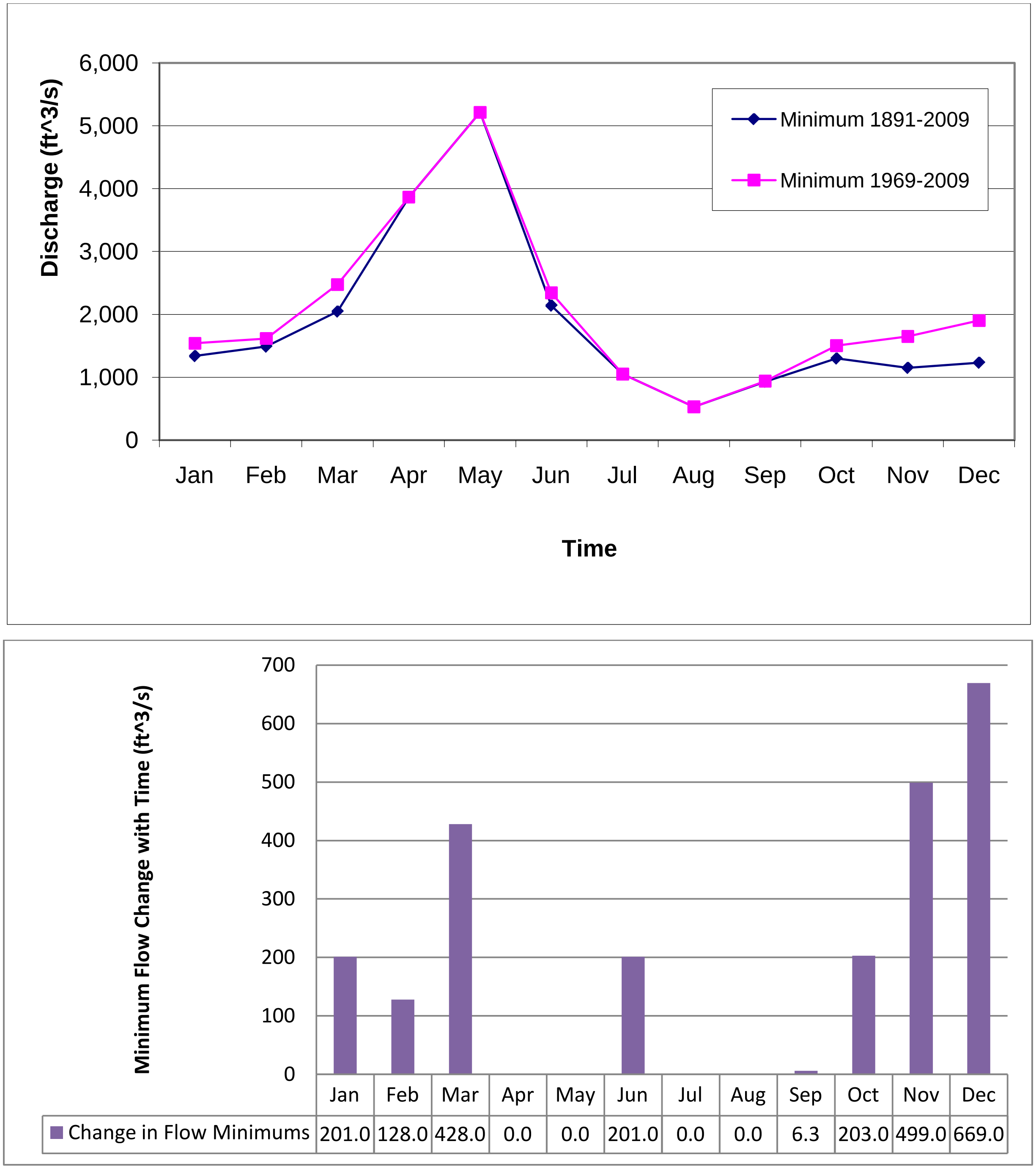

Figure 7. Minimum monthly average flows.

Figure 8 examines the shift in maximum average monthly flows. The maximum values are generally lower since 1968. The difficulty in analyzing minimum and maximum values, however, is that only a single dry or wet month is needed to produce a seemingly large difference. Nevertheless, because the 1969-2009 water year monthly average flow values shown in Figure 6 were consistently below the long term (1891-2009) average, we chose to conduct our Spokane River analysis based on the most recent 40 year period.

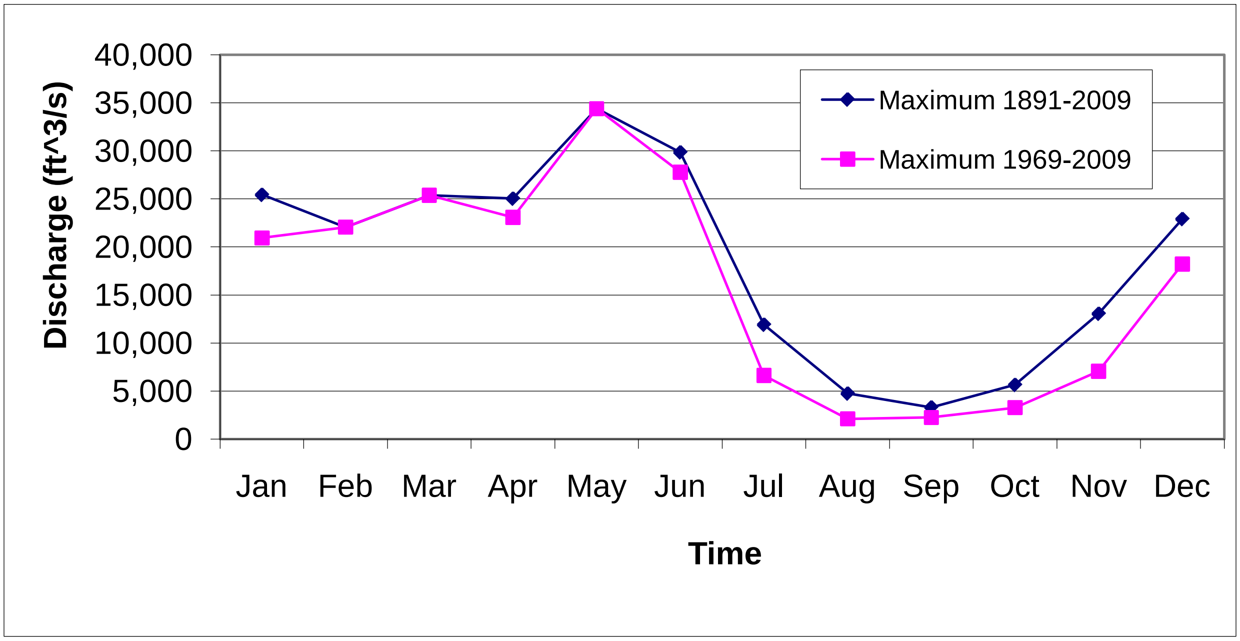

Figure 8. Maximum monthly average flows.

3.1.1 Diversion Constraints

Minimum instream flow requirements for the Spokane River are currently under discussion among impacted stakeholders and regulatory agencies. There will likely be additional debate before all parties agree to the final solution as there are still significant differences that exist between various ideas being proposed as indicated in Table 2. In addition, these are not the only flows that have been discussed; other groups would prefer higher flow levels in certain months due to access to spawning gravels and other considerations. Nevertheless, for the purpose of this analysis we elected to adopt the Ecology recommend instream flow requirement.

In addition to instream flow requirement, there are a number of existing hydroelectric facilities that have considerable capacities. According to Avista (2005), Post Falls Dam is located at River Mile (RM) 102 and has a hydraulic capacity of 5,600 ft^3/s; their Upper Falls facility is at RM 74.2 and has a turbine capacity of 2,500 ft^3/s; their Monroe Street facility is at RM 74 and has a capacity of 2,800 ft^3/s; their Nine Mile Dam (RM 58) has a generating capacity of 6,500 ft^3/s, and Long Lake (RM 34) can pass 6,300 ft^3/s through its turbines. Little Falls Dam is located at RM 29 and has a capacity of 7,000 ft^3/s (Picket, 2005). The largest turbine capacity is at the City of Spokane's Upriver Falls Dam (RM 80.2) which can handle 7,500 ft^3/s. While none of these rights have historically placed calls on junior rights, energy prices and climatic change impacts on the hydrologic cycles may force a somewhat different tact in the future.

Table 2. Proposed instream flow requirements at USGS Spokane gage (1 ft^3/s = 0.0283 m^3/s). (Spokane River Instream Flow Work Group, 2008)

Month	Ecology Recommended Instream Flows (ft^3/s)	City of Spokane Environmental Programs Recommended Instream Flows (ft^3/s)
January	1,100	1,100
February	1,100	1,100
March	1,100	1,100
April	< 115 ft^3/s for future diversions	2,700
May 1-14	< 115 ft^3/s for future diversions	2,700
May 15-31	< 115 ft^3/s for future diversions	2,300
June 1-15	< 115 ft^3/s for future diversions	2,300
June 16-30	850	565
July	850	565
August	850	565
September	850	565
October	1,100	780
November	1,100	780
December	1,100	780

Looking just at meeting the instream flow requirements at the Spokane River gage as the controlling factor for additional water withdrawals, Figure 9 demonstrates that considerable amounts of flow (> 2,000 ft^3/s) would be available for diversion on average during the November through March time frame. Flow availability for April, May, and June were set at 115 ft^3/s to be consistent with the proposed policy even though significantly more flow is present in these months. Figure 10 presents the 1969-2009 averages illustrating similar, although generally somewhat reduced, patterns and quantities of potentially available flow.

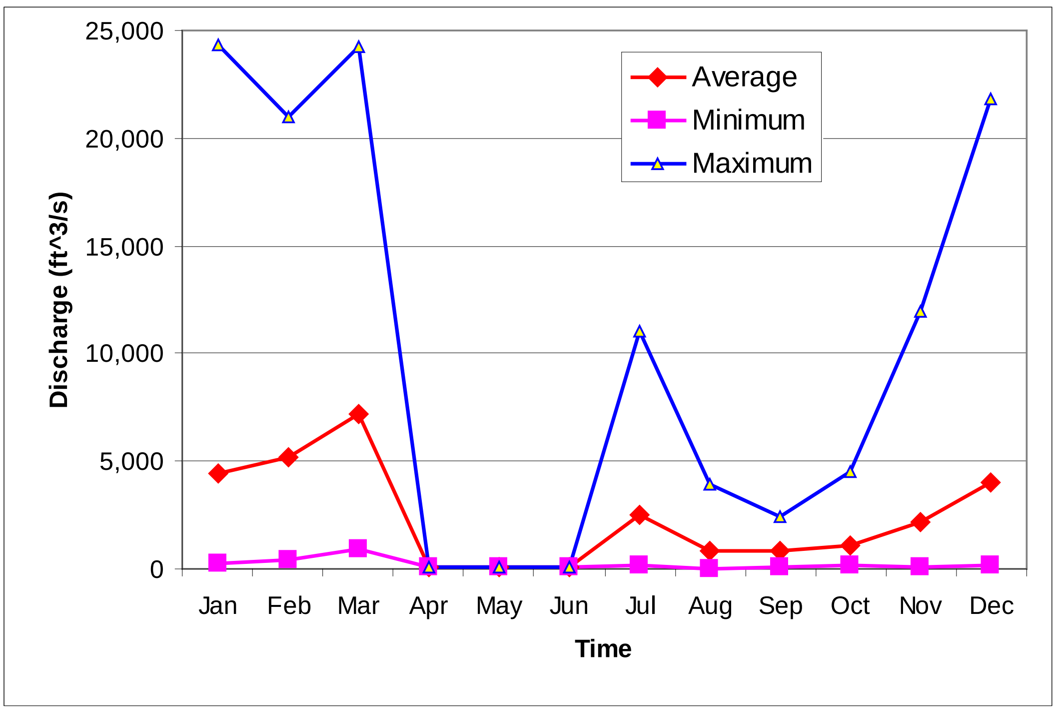

Figure 9. Average monthly 1891-2009 available flow at USGS Spokane gage after meeting Ecology's instream flow requirements (1 $ft^3/s = 0.0283\ m^3/s$).

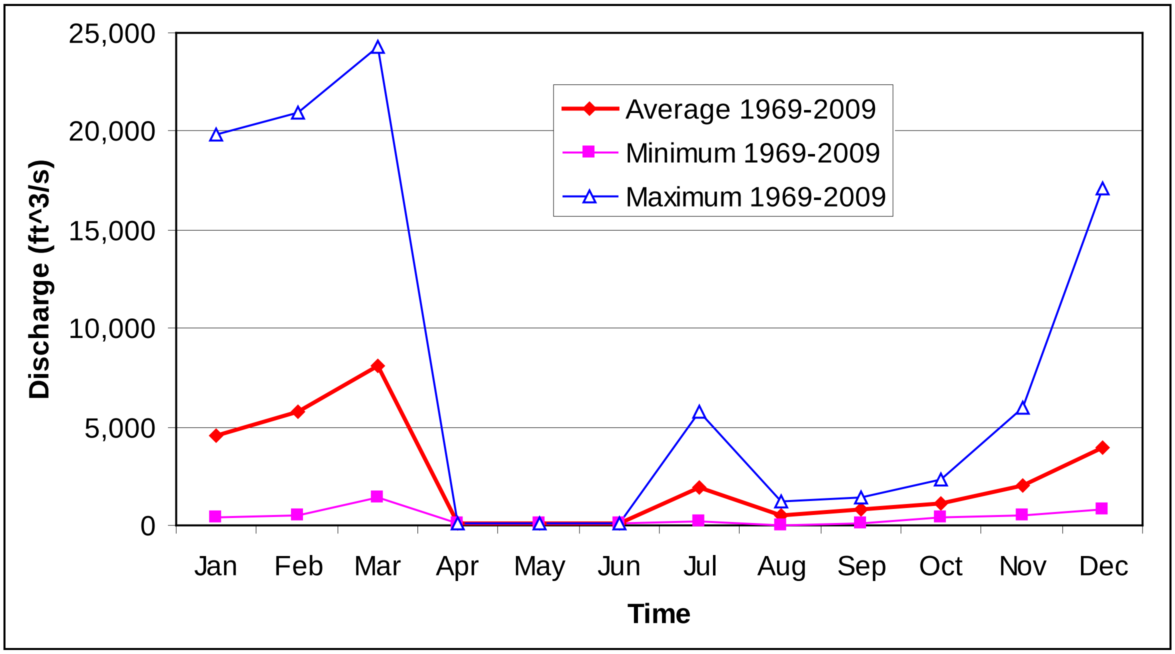

Figure 10. Average monthly 1969-2009 available flow at USGS Spokane gage after meeting Ecology's instream flow requirements (1 $ft^3/s = 0.0283\ m^3/s$).

Some caution must be exercised when looking at average flows as these are often influenced by large flood events, unseasonally late runoff, or rapidly falling flows at the end of the snowmelt runoff season. As illustrated in Figure 11, steadily decreasing (June and July) or steadily increasing (November) flows can make averages appear better or worse than actual conditions. Moreover, long-term averages can mask variability or recent trends. For example, in 14 out of 117 years of record, the instream flow requirements would prevent or severely limit diversions in August. This may not seem unreasonable in an overall assessment of feasibility until closer examination reveals that 10 of these occurrences have been since 1986. Presumably, a combination of increased pumping and climate change are altering low flows in the Spokane River (see Figure 12) such that additional diversions in the future would not likely be allowed. Similarly, the available flow rates for July and September are trending downward toward levels that make suspect the possibility of additional future diversions during these periods.

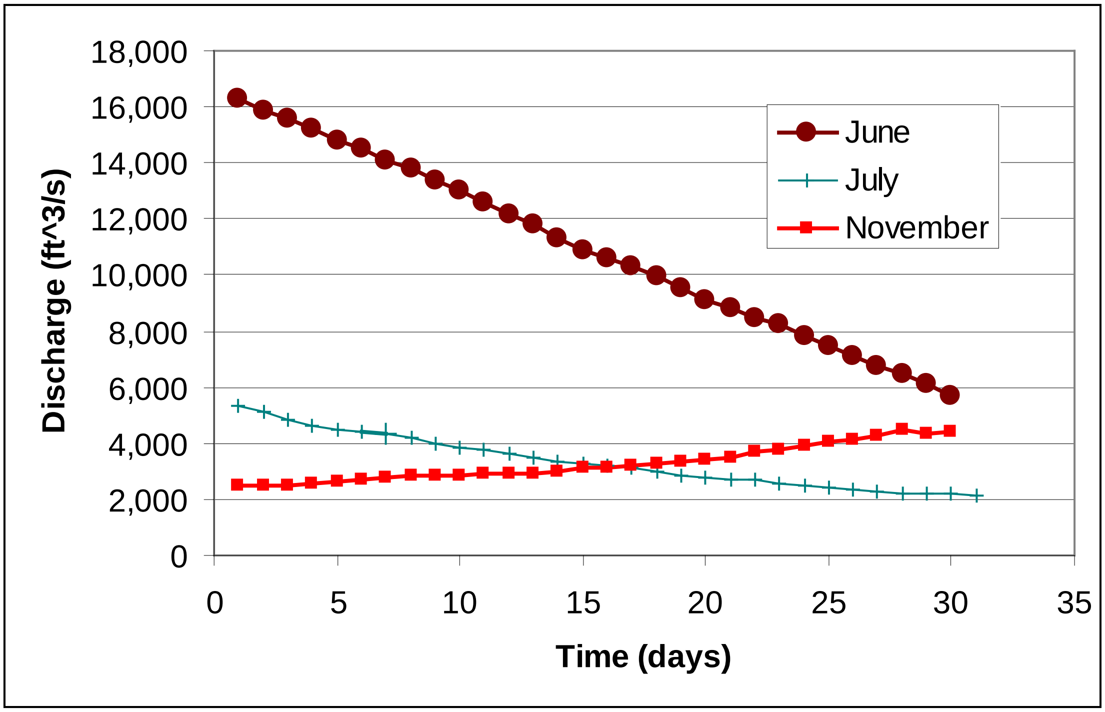

Figure 11. Average daily flow at USGS Spokane gage from 1891 to 2008 (1 ft^3/s = 0.0283 m^3/s).

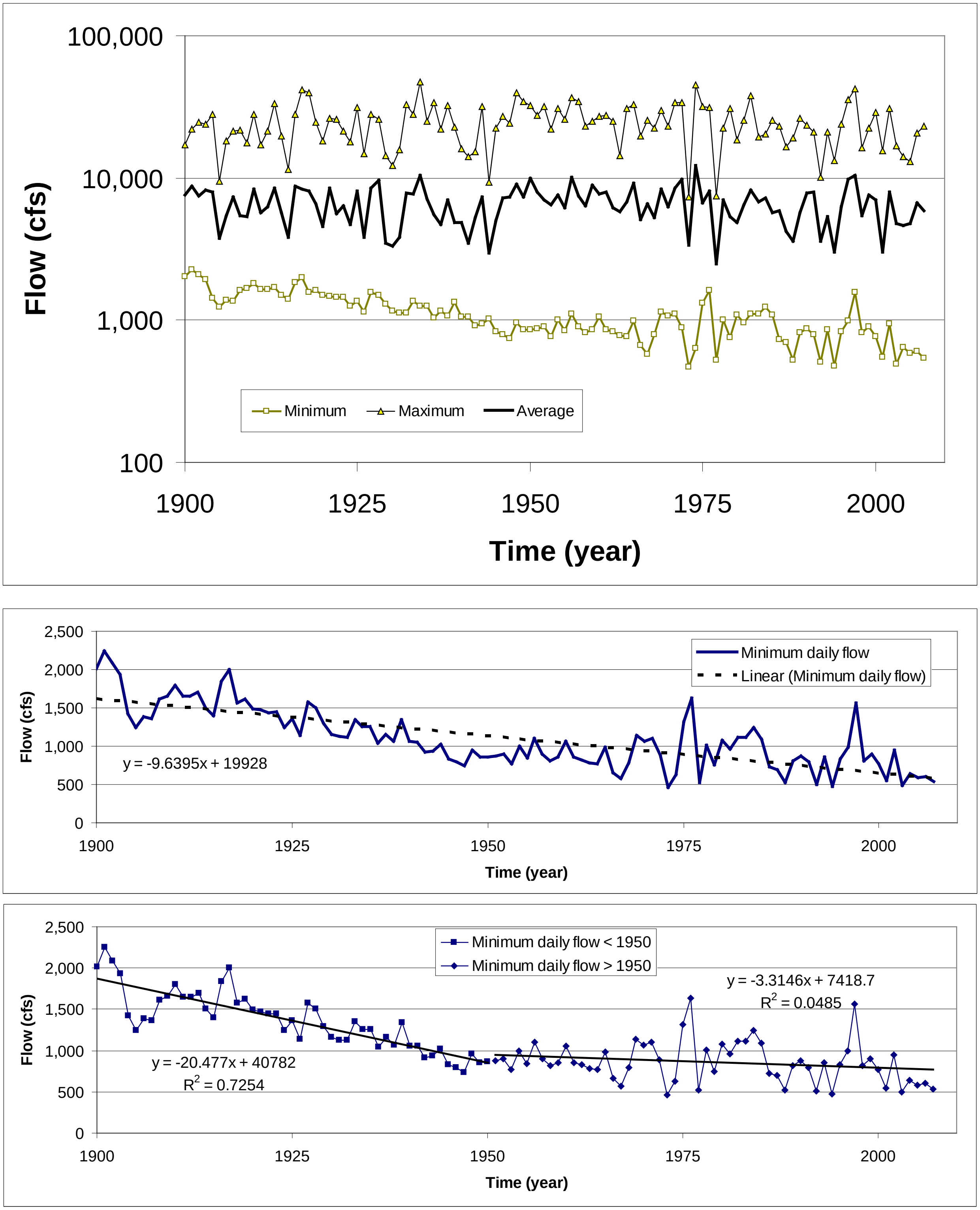

Figure 12. Long-term daily flow trend for Spokane River at Spokane gage
($1\ ft^3/s = 0.0283\ m^3/s$).

Assuming that April, May, and June diversions are limited by agreement and July, August, and September diversions are not permissible due to flow concerns, future diversions for the ASR Project would be limited to the six fall and winter (Oct-Mar) months. Looking at percent exceedance values for individual months yields the results shown in Figure 13 for the entire period of record and Figure 14 for the 1969-2009 time frame. These values are after adjustment for Ecology's proposed minimum instream flow amounts and thus reflect divertable quantities. Looking at the 90th percentile for the entire period of record (flow available in 9 out of 10 years) suggests 992, 992, 2340, 395, 583, and 669 ft^3/s are available Jan, Feb, Mar, Oct, Nov, and Dec, respectively. For the 1969-2009 period, the 90th percentile flows for the corresponding Jan, Feb, Mar, Oct, Nov, Dec periods are 1,033, 1,092, 2,728, 592, 873, and 1,218 ft^3/s, respectively. We have to be somewhat careful with these numbers; withdrawal of surface water may change both the surface- and ground-water flow regimes.

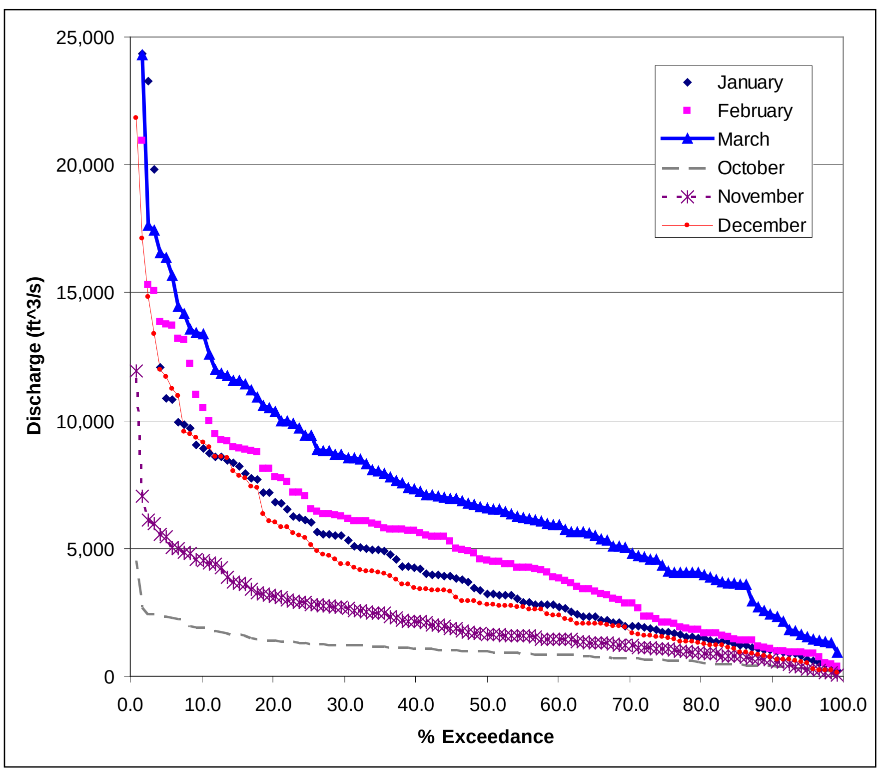

Figure 13. Exceedance values for 1891-2009 flows at USGS Spokane gage after adjusting for Ecology instream flow requirements (1 ft^3/s = 0.0283 m^3/s).

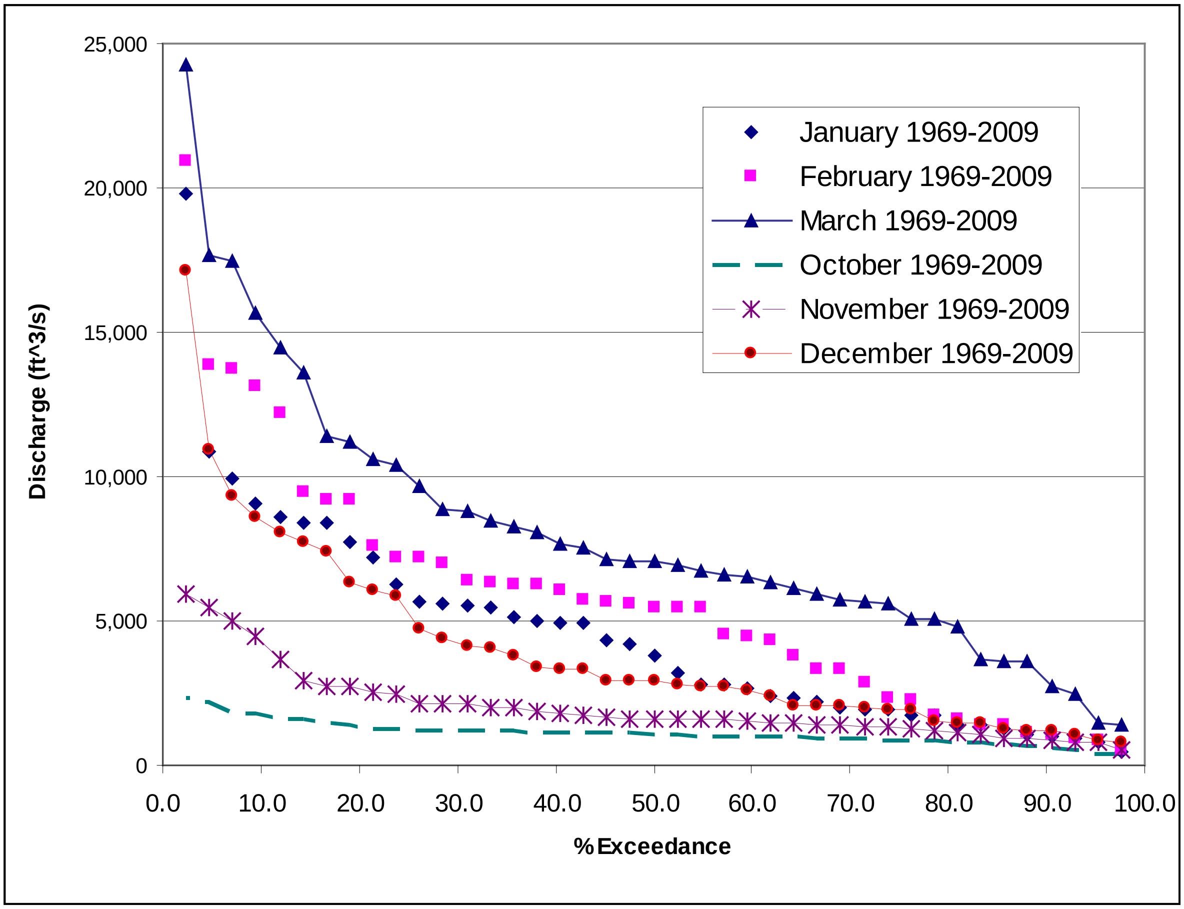

Figure 14. Exceedance values for 1969-2009 flows at USGS Spokane gage after adjusting for Ecology instream flow requirements (1 $ft^3/s = 0.0283\ m^3/s$).

As pointed out in a study by Fu et al. (2007), flows in the Spokane River watershed are best correlated by water year (October through September) because of the dependence of spring runoff on winter snowpack that begins in November and December. Consequently, the aggregate impacts of seasonal flows were also examined. In this analysis, average monthly flows from October through March were summed and ranked from low to high. Overall, seasonal values tended to be a bit higher than individual months although the differences were generally small except for March (see Table 3). There were also some discrepancies in the shorter 1969-2009 time period data due to monthly variations overshadowing the sum of the flow records. For example, the October 99th versus 90th percentile decreases from 1,612 ft^3/s to 1,085 ft^3/s because the December through March flows were higher.

Table 3. Available flows after accounting for Ecology's proposed instream flow requirements based on water year rankings (1 ft^3/s = 0.0283 m^3/s).

	Available Flows (ft^3/s)					
Exceedance	October	November	December	January	February	March
99 %	589	658	654	459	389	947
90 %	403	550	1,478	1,083	1,092	3,579
75%	721	1,062	1,130	3,150	3,735	4,562
	Available Flows for 1969-2009 (ft^3/s)					
Exceedance	October	November	December	January	February	March
99 %[1]	1,612	1,529	1,082	440	517	1,375
90 %	1,085	784	1,279	1,944	1,117	3,596
75%	542	1,670	2,036	1,716	1,380	6,115

[1] The lowest ranked water year values were used (97.6 %) as statistically extrapolation would be needed to determine 99 % values.

The average 1969-2009 water availability in December, January, February and March in this scenario is 3,993, 4,557, 5,790 and 8,081 ft^3/s, respectively. Looking beyond these relatively high averages, there is considerable additional flows even in low flow periods. Over the past 41 years, winter diversions above instream flow requirements could have been as low as 440 ft^3/s in January and 517 ft^3/s in February. However, this still represents a considerable amount of potential water (over 27,000 acre-ft in January and 28,800 acre-ft in February).

Ignoring the 115 ft^3/s proposed agreement for additional diversions, the month of May has the highest average available flow (14,846 ft^3/s) even after allowing up to 2,800 ft^3/s for instream flows. This far exceeds any likely development scenario.

Based on this review, it appears that up to 400 ft^3/s of water would be available for diversion in the October through March time frame. An additional 115 ft^3/s would be available in April and May. Our feasibility analysis will use these values as upper limits from the Spokane River. It is important to emphasize that these flows do not reflect existing unexercised water withdrawal authorizations in Washington or Idaho, values not quantified in this study.

3.2 Pend Oreille River and Lake System

Another potential source of water for the ASR project is the Pend Oreille watershed located north of the SVRP domain. While the river drainage system is by and large to the north of the SVRP aquifer, Lake Pend Oreille abuts the aquifer along the northeastern corner (see Figure 15). Modeling results (Hseih et al. 2007) and groundwater elevation data already indicate that the Pend Oreille River watershed contributes flow to the SVRP via subsurface connection from the lake to the aquifer. Although increasing the quantity of water, either through direct surface water withdrawals or a well field, would likely be seen as a basin transfer, there remains a high potential for excess water during some parts of the year. It is also possible that storage agreements could be configured such that reservoir operation would increase the potential window for diversions.

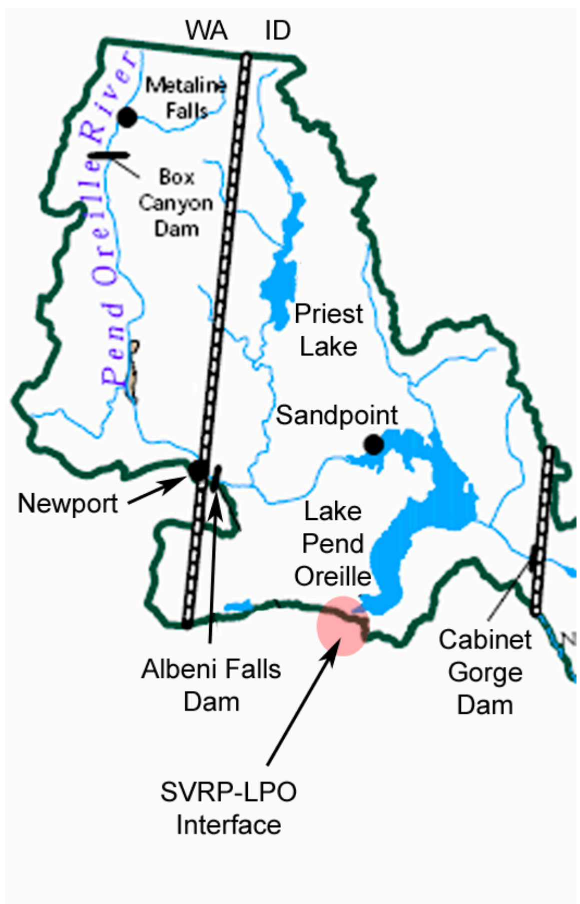

Figure 15. SVRP interaction with Pend Oreille watershed (SVRP-LPO Interface).

The USGS has operated the Pend Oreille River at Newport, Washington gage (station number 12395500) since 1903 although water years 1913-1928 and 1942-1952 are missing. The USGS gage is located at latitude 48°10'56" N and longitude 117°02'00" W on the left bank of the river near Newport, Washington. It is 0.2 miles upstream from the U.S. Highway 2 bridge, 0.2 miles east of Idaho-Washington State line, 1.6 mi downstream from Albeni Falls Dam, and at river mile 88.5. The drainage area is approximately 24,200 square miles. Since the early 1950's, discharge measurements have been regulated by operation of the dam. Albeni Falls Dam is a 90 foot high concrete dam that was built over a 5-year period (January 1951 to December 1955) for flood control, hydropower, navigation, recreation, water quality and fish & wildlife. As illustrated in Figure 16, the impact of the dam has been to reduce high flows during the historic snowmelt periods and increase low flows from September through March by releasing water from storage. Because of this significant change, we elected to use only the 56 year period of record after 1952 in the water availability analysis.

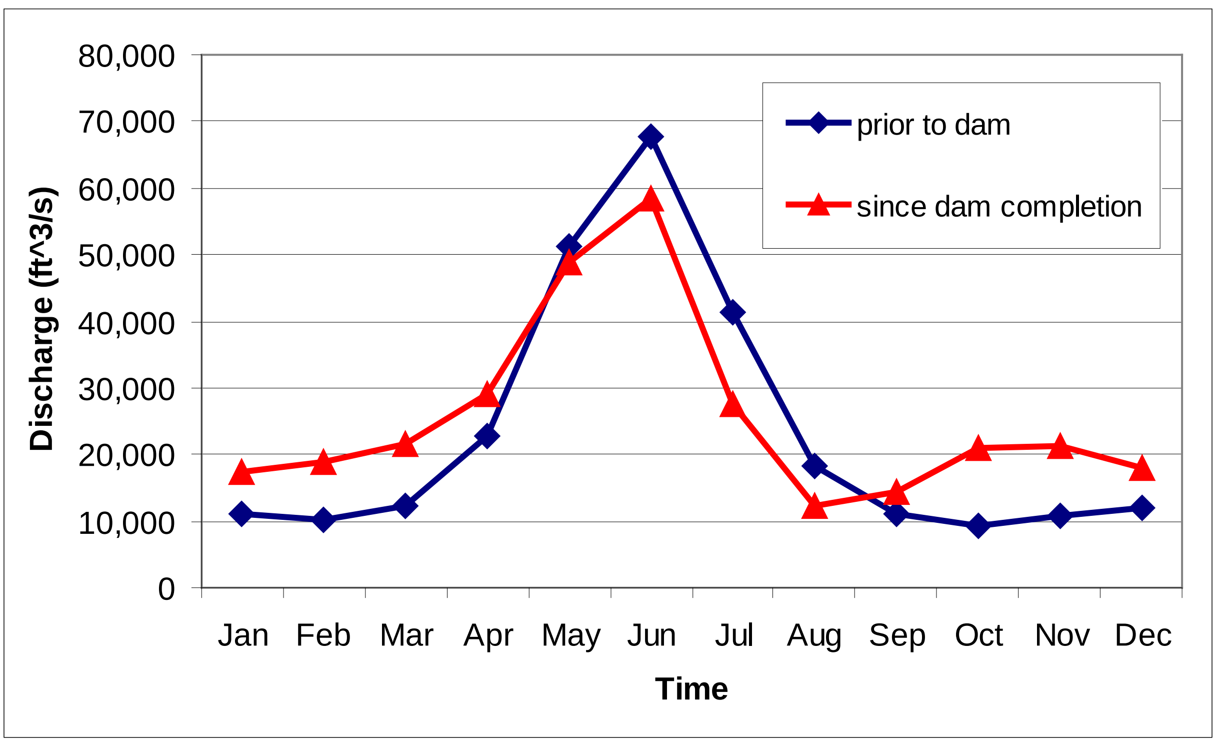

Figure 16. Impact of Albeni Falls Dam on Pend Oreille River flows (USGS 12395500).

The U.S. Army Corps of Engineers Reservoir (USACE) Control Center in Seattle, Washington operates Albeni Falls Dam, which regulates the level of Lake Pend Oreille and thus the flow downstream in the river. Summer lake elevation levels are typically between 2062.0 and 2062.5 feet. Fall drawdown begins in mid-September continuing until the winter flood storage target level (2051 feet) is reached during the first week of November. Figure 17 illustrates operating limits during a normal year. Winter lake elevation recommendations are made by the U.S. Fish and Wildlife Service and Idaho Department of Fish and Game. These agencies, along with the National Marine Fisheries Service, the USACE, Bonneville Power Administration and other stakeholders, review results of the annual female kokanee (*Oncorhynchus nerka*) spawner survey, recent winter lake elevation history, the previous year's lower Columbia River chum salmon spawning success, and the forecasted precipitation trend in the Columbia River system to determine the specific storage target. Reproduction of the lake's shore-spawning kokanee is considered important because these fish serve as a major food source for Lake Pend Oreille's threatened native bull trout (*Salvelinus confluentus*). While the project has experimented with several winter pool depths, NOAA Fisheries recommended operation is 2,051 feet to provide additional water for chum flow in the fall.

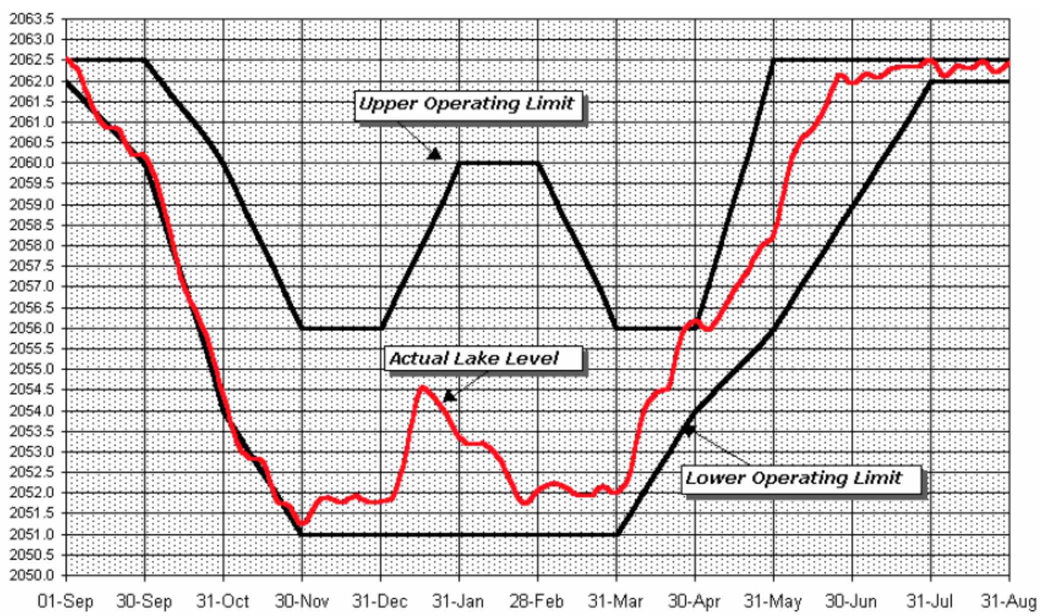

Figure 17. Water regulation curve for a typical year (1989-90) for Lake Pend Oreille (http://www.nws.usace.army.mil/PublicMenu/documents/ALBENI/rule_c2.pdf).

3.2.1 Diversion Constraints

State established flow targets

There are currently no state-mandated instream flow requirements for the Pend Oreille River downstream of Albeni Falls Dam on the Washington side of the border. However, in 1992 the Idaho Department of Water Resources issued the Idaho Water Resources Board an instream flow right of 10,655 ft^3/s immediately downstream of the dam. The water right (Permit No. 96-8730) established a constant year-round flow for the 2.4 mile river reach in Idaho to protect fish and wildlife habitat, aquatic life, and recreational values.

Columbia River system needs

At the federal level, considerable discussions continue to occur on how to operate the Albeni Falls reservoir in conjunction with other projects in order to achieve the minimum weekly flow objectives downstream at McNary Dam (Litchfield and Marotz, 2008). Flow releases are negotiated by a Technical Management Team comprised of a number of federal, tribal and state agencies including NOAA, USACE, Bureau of Reclamation, US Fish and Wildlife Service, BPA, the Colville Tribe, and others. This section examines how the Pend Oreille flows are constrained by mainstem Columbia needs recognizing the evolving changes and system-wide complications and ramifications of storage operations.

For example, under a 2008 request to the Technical Management Team, water managers would bypass inflows at Albeni Falls until the 2008 freshet begins. This requires a fairly complex solution because Lake Pend Oreille has 1.15 million acre-feet of useable storage capacity and ultimately the operation of the Grand Coulee and Libby projects are involved and trade-offs between resident fish populations (sturgeon, bull trout, kokanee, etc.) versus downstream salmonid species as well as hydropower, irrigation, flood control and recreation interests.

Further complicating the issue is that the State of Idaho has not historically issued state water rights with provisions tied to federal targets. Thus, the consequence of these targets on state operations is uncertain and beyond the scope of this project to analyze. While no immediate change in policy is anticipated, the legal system in association with endangered and threatened species is complex and constantly evolving.

The State of Washington (Washington Administrative Code 173-563-040) established the weekly average instream flow requirements for the main stem of the Columbia River shown in

Table 4. The flow at Priest Rapids is the overall control in the Columbia River above its confluence with the Snake River. In other words, the greater flow requirement between the project site or Priest Rapids governs the requirement. However, Ecology also reserves the right to decrease these flows in times of drought. According to the WAC, the minimum average weekly flows shown in Table 4 are subject to a reduction of up to 25 percent during low flow years, except that in no case shall the outflow from Priest Rapids Dam be less than 36,000 ft^3/s. This rule is no longer applied to new applications (decisions made after 07/27/1997 – WAC 173-563-020(4)), as the rule was amended to require consultation on a case-by-case basis after this date.

Table 4. Average weekly instream flow requirements along the Columbia River.

Date	Chief Joseph	Wells and Rocky Reach	Rock Island and Wanapum	Priest Rapids	McNary	John Day	The Dalles
January	30,000	30,000	30,000	70,000	60,000	60,000	60,000
February	30,000	30,000	30,000	70,000	60,000	60,000	60,000
March	30,000	30,000	30,000	70,000	60,000	60,000	60,000
April							
1–15	50,000	50,000	60,000	70,000	100,000	100,000	120,000
16-25	60,000	60,000	60,000	70,000	150,000	150,000	160,000
26-30	90,000	100,000	110,000	110,000	200,000	200,000	200,000
May	100,000	115,000	130,000	130,000	220,000	220,000	220,000
June							
1-15	80,000	110,000	110,000	110,000	200,000	200,000	200,000
16-30	60,000	80,000	80,000	80,000	120,000	120,000	120,000
July							
1-15	60,000	80,000	80,000	80,000	120,000	120,000	120,000
16-31	90,000	100,000	110,000	110,000	140,000	140,000	140,000
August	85,000	90,000	95,000	95,000	120,000	120,000	120,000
September	40,000	40,000	40,000	40,000	60,000	85,000	90,000
October							
1-15	30,000	35,000	40,000	40,000	60,000	85,000	90,000
16-31	30,000	35,000	40,000	70,000	60,000	85,000	90,000
November	30,000	30,000	30,000	70,000	60,000	60,000	60,000
December	30,000	30,000	30,000	70,000	60,000	60,000	60,000

All of these requirements are downstream of Lake Roosevelt and thus subject to intense manipulation via dam releases. It is therefore difficult to transfer these requirements upstream to the Pend Oreille watershed. While the WRIA 62 Planning Unit and Ecology appear to be working towards determining flow requirements, none have been established. According to the WRIA plan, once the rule is established, the planning unit would like Ecology to petition the governor to ask Congress to raise the minimum discharge from Albeni Falls Dam from 4,000 ft^3/s to the discharge necessary to follow the new flow rule (Golder 2005). Although Idaho Fish and Game mention rainbow and brown trout spawning below Albeni Falls Dam, the primary ecological concern for this reach appears to be bull trout (*Salvelinus confluentus*). According to the US Fish and Wildlife (2002), bull trout historically used the river primarily to seasonally migrate from Lake Pend Oreille downstream into the Pend Oreille River tributaries to spawn and rear as well as for feeding and overwintering. Without fish passage at the dam, the connection between the lake and river may be lost but bull trout are likely still present (Geist et al. 2004).

<u>Summary of administrative requirements</u>

It is unclear how these federal flow targets might impact the availability of water for the SVRP ASR Project. First, the BiOp values appear to be considerably different than the recommendation set forth by WAC 173-563-040 although the latest 2008 BiOP may change the recommended 135,000 ft^3/s mid-April through June Priest Rapids target. Second, the river flows are highly regulated by upstream reservoir operation. The Technical Management Team (TMT) appears to meet regularly during the flow periods to examine system-wide options for possible reservoir drawdowns to increase flows when targets are not being met. However, there is a degree of understanding as to the feasibility (or infeasibility) of meeting the flow targets under variable hydrologic conditions. In addition, non-federal projects such as the Hells Canyon Project owned by Idaho Power appear to be exempt from the BiOp. Moreover, it is uncertain as to how the TMT would view the trade-off between reducing winter flows and increasing summer flows.

As a consequence of the uncertainty associated with the system constraints posed by the BiOp flow and state flow requirements, this study assumed no water availability thus no potential for diversions from the Pend Oreille watershed during the months of July or August. In fact, in a preliminary telephone conversation with a NMFS TMT member, it was suggested that the November through March time frame would likely be more acceptable to the parties except when April and May targets at McNary (220,000 ft^3/s from April 10 through June) were being met. Since the lag time would need to be fairly long for water to show up in the following July-August period, it was decided to also exclude September and October from consideration.

The USGS gage (12395500) on the Pend Oreille River represents approximately 81 years of data (1903-2009) although as previously mentioned there are discontinuities in the period of record. Data from the periods between 1912-1928 and 1942-1952 are missing. The system is heavily managed due to construction of dams so it was decided to focus on the period of record after dam construction (1952-2009). Looking strictly at average, maximum, and minimum average monthly discharges of the entire period of record regardless of water year produces the graph shown in Figure 18. Typical of northwest snowmelt-dominated streams, the peak discharge occurs in June and the low flow occurs in late summer (August-September). It is important to note that not all the minimum and maximum average monthly flows occur in the same water year because of upstream storage and release.

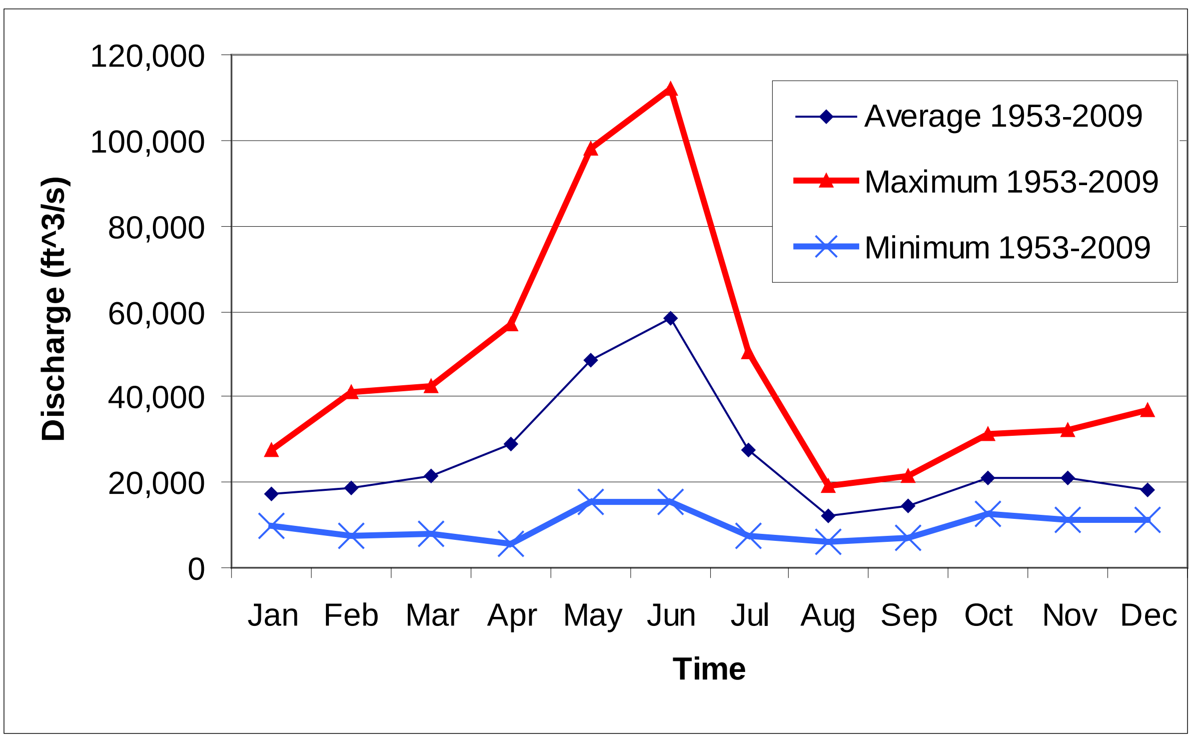

Figure 18. Monthly stream flows at USGS Albeni Falls gage (1953-2009).

Flood flows in excess of turbine capacity (hydraulic capacity is 33,000 ft^3/s) are passed through the spillway structure. As a result, these flows do not generate hydropower and upstream withdrawal would not impact electricity generation at Albeni Falls. Actually, since spills can lead to total dissolved gas (TDG) problems downstream, reducing spills may have a positive environmental benefit. Daily spill values averaged over the past 10 years (1999 to 2008) are shown in Figure 19. The figure indicates that significant flow (e.g., greater than 1,000 ft^3/s) is spilled from April 1 through July 15. Between April 1 and June 30, the average daily spill rates exceed 18,000 ft^3/s. The average 10 year monthly spill rates for April, May and June are 5,086, 20,227, and 28,525 ft^3/s, respectively. Furthermore, the lowest average daily spill during that period was 850 ft^3/s in early April. Because the flows are highly impacted by upstream reservoir operation, shorter duration fluctuations in spill rates are difficult to interpret.

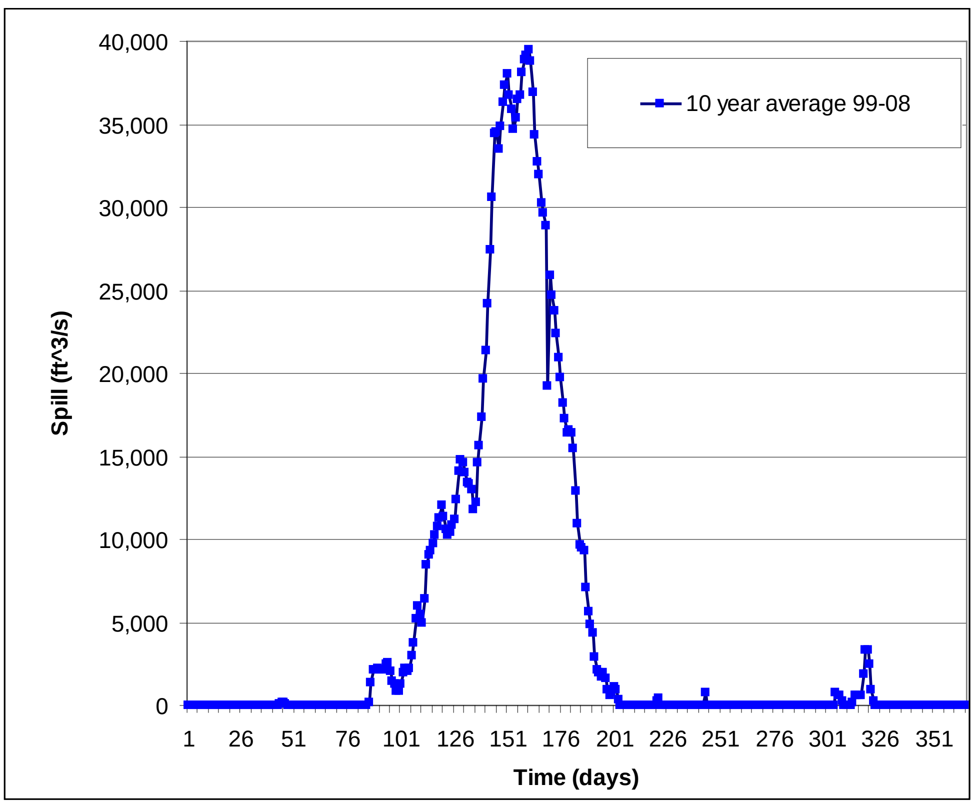

Figure 19. Average 10-year Spill from Albeni Falls Dam from 1999 to 2008.

Stream flows in the Pend Oreille watershed vary considerably from year to year due to both snow pack differences and the operation of upstream reservoirs. Figure 20 illustrates the high (1997), average (1955), low (2001), and median (average 1962 and 1970) water years. As with many snowmelt dominated streams, peak flows occur in the May-June time frame. A low average monthly flow of 7,495 ft^3/s occurred in February 2001 although the lowest monthly flow on record (5,507 ft^3/s) occurred in February 1977. The 1977 water year was only slightly wetter (less dry) than the 2001 year. The next driest year (1988) is nearly 20% wetter than these two extremely dry years.

In addition to natural climate-driven variability, the Pend Oreille watershed is now a fairly managed system due to the construction of dams and reservoirs in Idaho and Montana built in the mid 1900's. Figure 21 demonstrates the significance of this impact. Prior to 1942, discharges were considerably higher in June, July and August than they have been since 1953. Conversely, flows at other times of the year are now higher since the stored water can be used to manipulate stream flows. The low flows prior to 1942 appear to be very close to the minimum instream flow requirement imposed on the system by the State of Idaho in 1992.

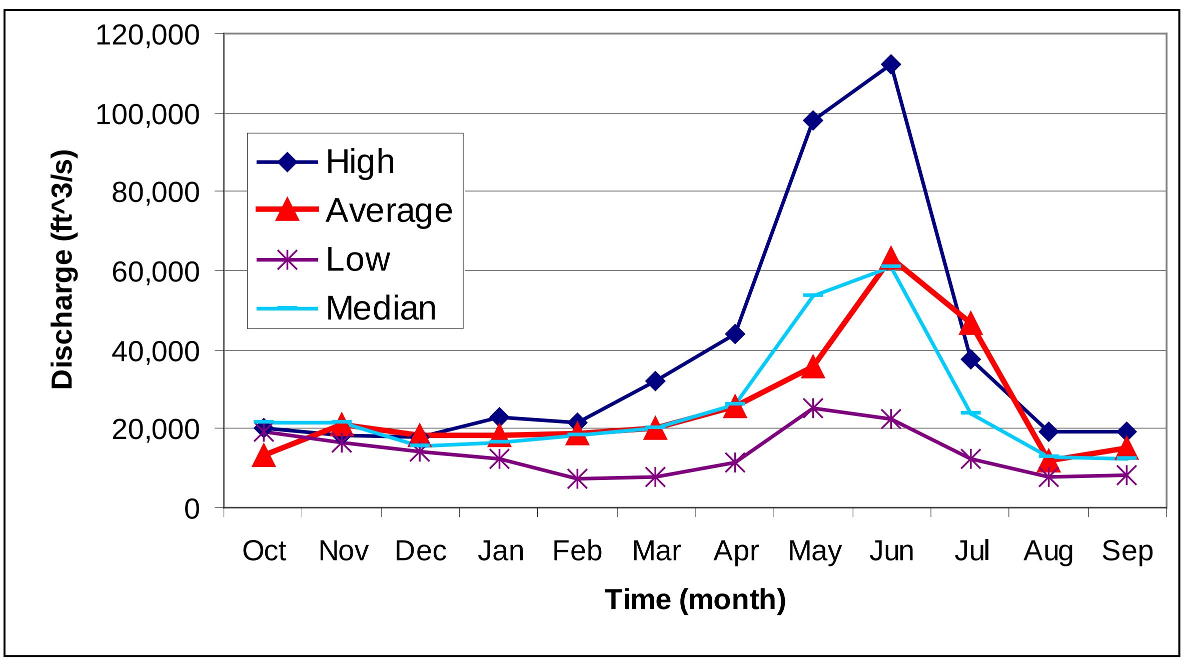

Figure 20. Monthly average discharge in Pend Oreille River for range of water year flows

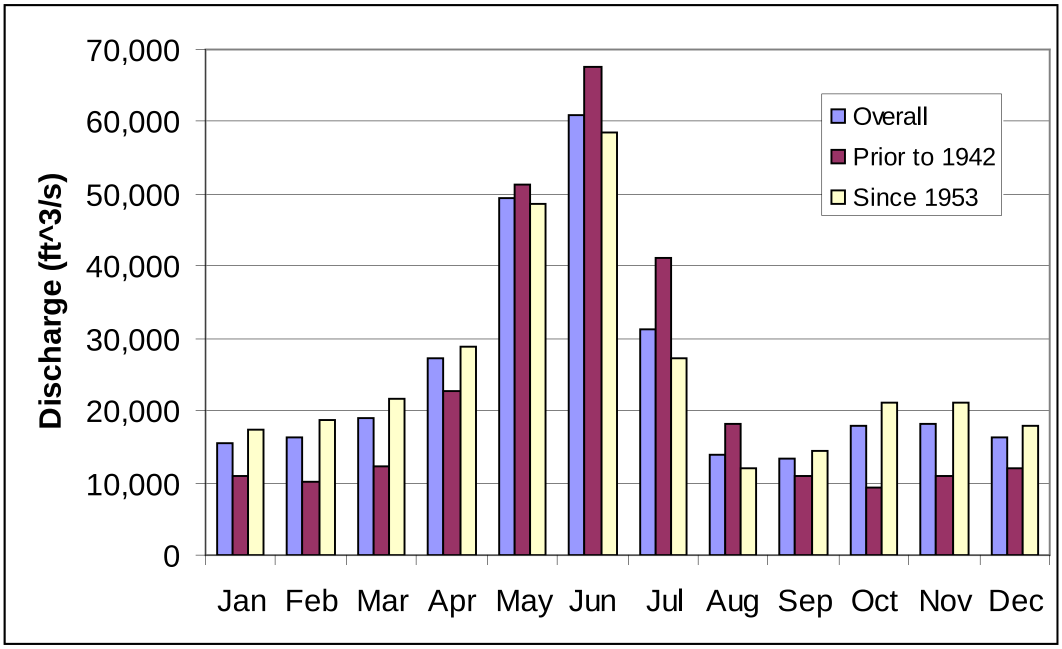

Figure 21. Average monthly flow below Albeni Falls Dam location.

Using the 10,655 ft^3/s instream flow set in Idaho as the constraint against upstream diversions, the frequency and quantity of flow available were determined. Table 5 indicates the frequency where flows for diversion were not available (i.e., the instream flow would not be met) over the water year 1953-2009 period (57 years). For example, in January, 100 ft^3/s could be diverted 55 of the 57 years compared to August where 100 ft^3/s could be diverted only 39 of the 57 years. The data indicate that up to 4,500 ft^3/s would be available in May and June in all 57 years. Likewise, on average, considerable amounts of flow would be available in most months (August and September being the obvious exceptions). The risk of not being able to operate the SVRP ASR Project below 500 ft^3/s for periods other than August and September would be minimal based on the historic records.

There is one point of caution that must be made regarding the use of monthly averages in the transition month (July) especially at the higher discharge amounts. There is a precipitous drop-off in discharge between July 1 and July 31 (43,200 to 15,500 ft^3/s) as shown in Figure 22. Although both of these values are well above the 10,655 ft^3/s instream flow, using an average monthly value may be misleading.

Table 5. Frequency of occurrences where average monthly flow (1953-2009) after instream flow requirement did not exceed diversion target.

Flow (ft^3/s)	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
< 100	2	2	1	1	0	0	2	18	12	0	0	0
< 250	3	2	1	1	0	0	2	18	12	0	0	0
< 500	3	3	1	1	0	0	2	18	13	0	0	0
<1000	4	4	1	2	0	0	3	24	13	0	1	1
< 2500	8	11	2	2	0	0	5	35	20	2	1	3
< 5000	19	20	11	3	1	1	8	51	35	8	4	18

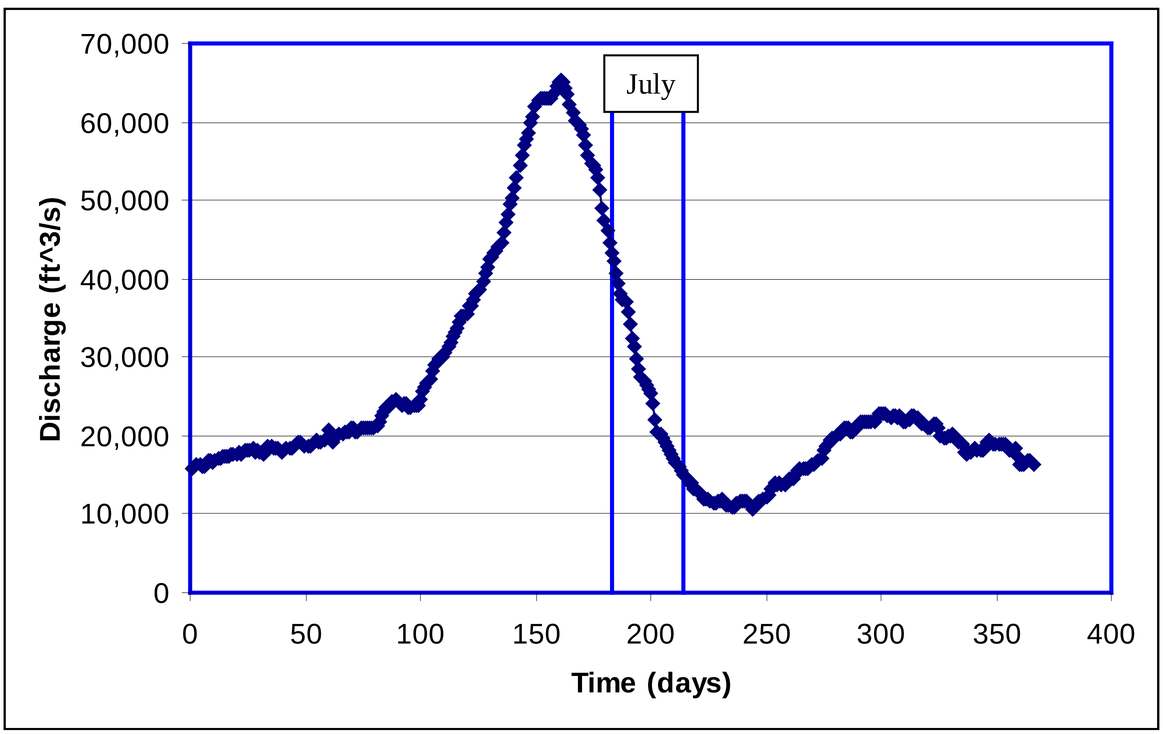

Figure 22. Average daily flows at Albeni Falls gage from 1952 through 2009 (1 $ft^3/s = 0.0283\ m^3/s$).

Nevertheless, it appears that 500 ft^3/s could be diverted any time between October and July without adversely impacting the minimum instream flow. A 5% chance exists that this quantity of flow would not be available in January or February. However, during the March through June time frame, the likelihood that the water would be available is fairly large. Furthermore, the data indicate that there is a realistic expectation of long-term diversions up to 1,250 ft^3/s in most years or short-term (May-June) diversions up to 4,500 ft^3/s.

The scenario involving satisfying the hydraulic capacity of the dam is less encouraging. As indicated in Figure 23, the hydraulic capacity (hydropower capacity) is rarely exceeded except during the months of May and June. High spill rates occur infrequently in other months. Spills in March occurred only 3 times during the 57 years of record with an average rate of 332 ft^3/s, and spills in April occurred only 18 times during the 57 years of record with an average spill rate of 2924 ft^3/s. However, operating a facility that is capable of diverting water infrequently is not generally cost effective unless the May and June diversions make up the majority of water and the other diversions are seen more as bonuses.

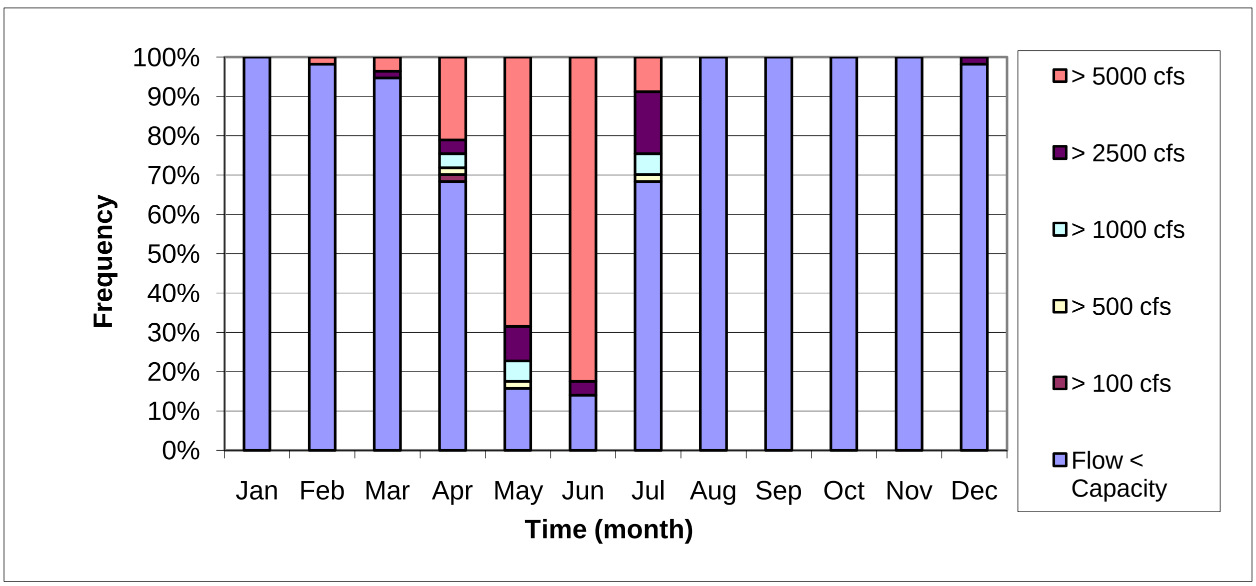

Figure 23. Frequency that hydraulic capacity is exceeded by diversion target.

Hydraulic capacity alone does not dictate when out-of-stream diversions are allowed. Because this is a heavily managed system for flood control, fisheries, recreation, and hydropower, stream flow records are distorted from natural flow regimes and different upstream reservoir operations could make up the flow deficiencies. Furthermore, by shifting water from high flow to low flow periods, additional hydropower can be generated along the Spokane River-Grand Coulee pathway during summer peak demand periods so water may be worth more in August than April. Arguably, it is a question of who benefits from this shift, and some amount of mitigation may be necessary. In a constrained water environment, municipal water is almost always worth more than the hydropower loss making economic compensation feasible, if required.

A possible long-term constraint to water availability that bears discussing is the impact of climate change. Figure 24 illustrates the overall average annual flow on a water year basis (e.g., the sum of October through September average monthly flow divided by twelve) from 1953 to 2008. A trend line through the data reveals a slightly negative decrease in flow since 1953. In spite of the fact that 1996 and 1997 represented very high flow levels, the average values have consistently been below the 25,000 ft^3/s level since 1980. However, we feel that this still represents a significant amount of water.

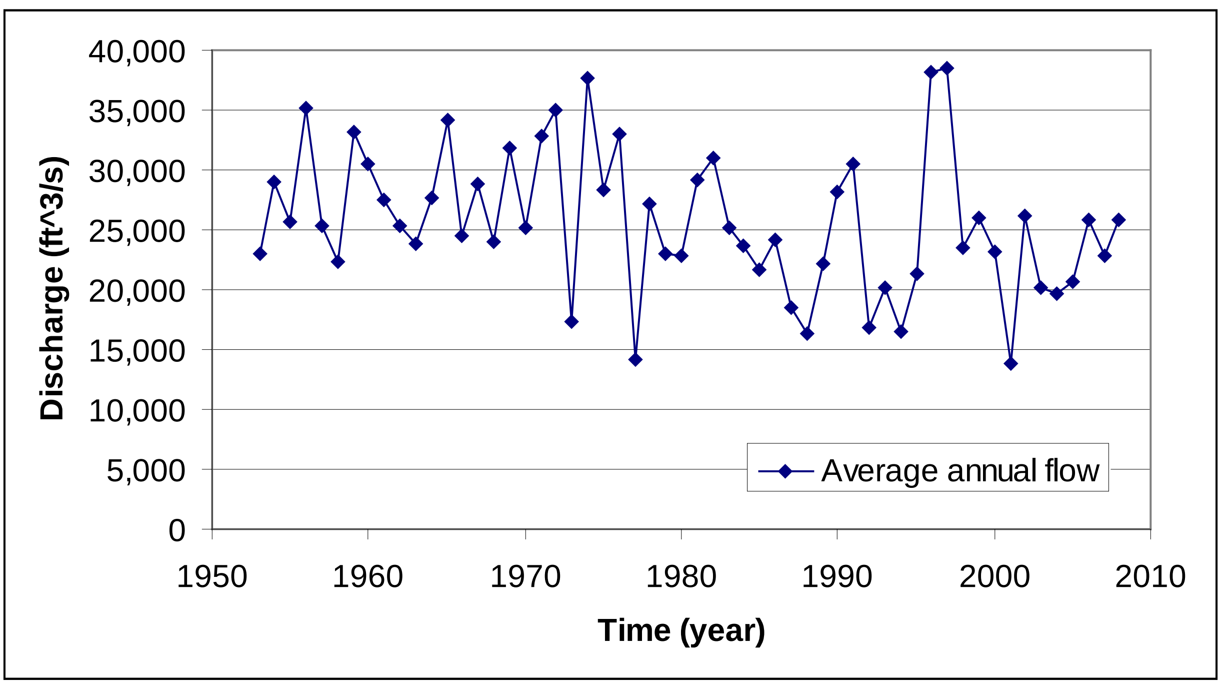

Figure 24. Average annual monthly flow by water year (1 ft^3/s = 0.0283 m^3/s).

Overall, there appears to be more than enough flow in the Pend Oreille watershed to support the SVRP ASR project. Physically, in most years the project could take up to 2,500 ft^3/s in peak runoff months (May and June) without significantly adversely affecting operation of the lake or hydropower facility. In most years, diversions between 500 and 1,250 ft^3/s from November through April are also physically viable based on conservative instream flow requirements.

3.3 Summary of Maximum Reliable Diversion Quantities

In conclusion, we elected to use the flows shown in Table 6 as potential upper limits to quantities and monthly timing of allowable diversions. To be clear, these are not the flows we anticipate modeling in the feasibility study, they are simply estimates of the total amount of flow that are likely to be physically available.

Table 6. Maximum diveratable quantities (ft^3/s).

Month	Spokane	Pend Oreille
January	400	500
February	400	500
March	400	500
April	115	750
May	115	2,500
June	0	2,500
July	0	1,000
August	0	0
September	0	0
October	0	0
November	400	500
December	400	500

4.0 System Limitations

It is important to identify and examine the system limitations associated with the SVRP aquifer serving as a potential ASR project. This is particularly true given the relatively large downstream demand for water that this project could ultimately be used to help supply. Consequently, this chapter includes an investigation of aquifer properties such as depth, hydraulic conductivity, and storage potential as well as discharge limitations in the Spokane and Little Spokane Rivers, the possible impacts on dams and impoundments, and administrative issues such as closures and future growth in the Coeur d'Alene and Spokane areas. In addition, environmental considerations were included in the evaluation. For example, a potential discharge location subjected to excessive injection may result in a significant rise in the groundwater table that may threaten local basements or quarries.

4.1 Direct Injection – Aquifer Properties

The SVRP aquifer is characterized by sands, gravels, cobbles, and boulders which result in the SVRP having very high permeability values (Hsieh et.al, 2007) and consequently relatively high hydraulic conductivities (5 to 22,100 ft/day) throughout most of the aquifer. As such, injection or pumping often results in more horizontal flow with less vertical mounding than in many typical aquifer storage and recovery (ASR) situations. This fact aids the potential for the SVRP as an ASR project by increasing the ability of the system to react to intermittent high intensity changes in inflows and outflows to the aquifer. The aquifer properties such as depth, hydraulic conductivity, storage potential, discharge limitations to the Spokane and Little Spokane Rivers, and mounding from injection are analyzed in this section.

The area defined by the SVRP aquifer ranges in depth to the bed rock from approximately 850 feet in the Northeastern part to as little as 8 feet at the end of the western arm of the Spokane River before it joins the Little Spokane River. Figure 25 shows how the depth of the SVRP aquifer changes throughout its extent.

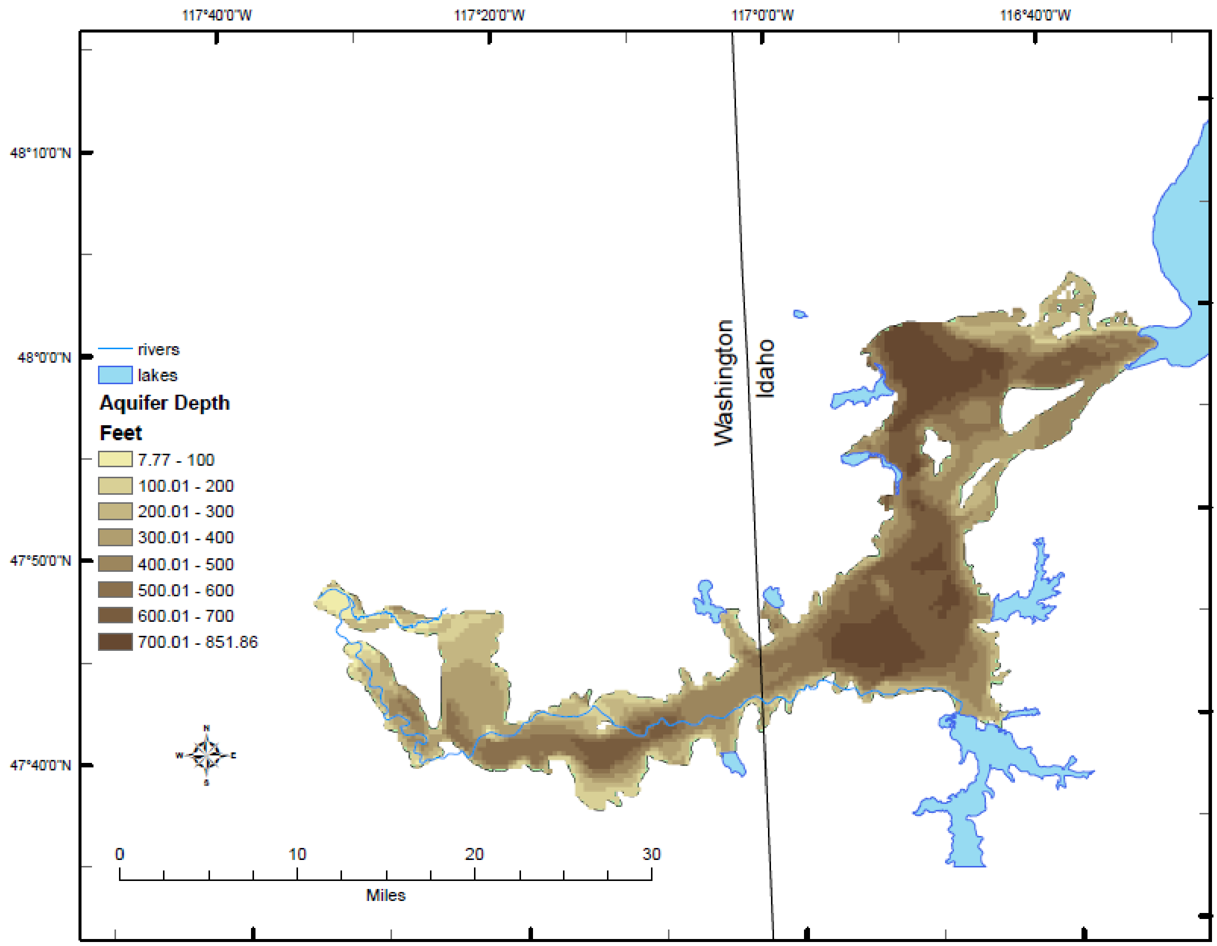

Figure 25. Depth from ground surface to bedrock in area defined by the SVRP aquifer (Hsieh et al. 2007).

The depth of the saturated zone varies between wet and dry seasons in the region and also by longer-term variations in water year runoff volumes. To determine the naturally occurring range of saturation depths for the SVRP aquifer, the lowest and highest average stream flows per month recorded by the United States Geological Survey (USGS) on the Spokane River at Spokane, WA (gage number 12422500) between September of 1990-2005 were used to represent the driest and wettest periods, respectively. August 1994 was the lowest stream flow recorded during that period at 531 ft^3/s and May 1997 was the highest stream flow recorded during that period at 34,390 ft^3/s. The resulting water table elevations from Visual MODFLOW (the Alternatives Evaluation section discusses in more detail how Visual MODFLOW was used to determine these values) were used to establish ranges that represented the natural limits of the aquifer. Using the natural range of the aquifer allowed the affects of the ASR project to be understood within the context of seasonal fluctuations. Figure 26 and Figure 27 were developed using the respective month and year heads from the Visual MODFLOW model and subtracting these values from elevation of the bottom of Layer 1. This shows the response of the entire aquifer which allowed the natural range for multiple areas to be determined.

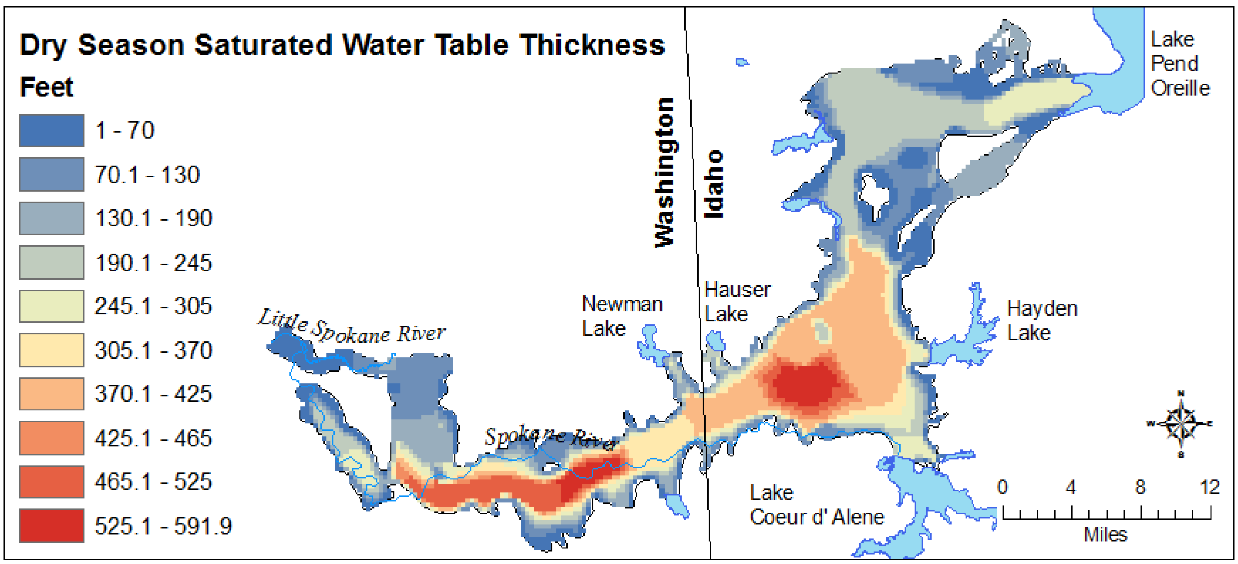

Figure 26. Low flow period saturated water table thicknesses.

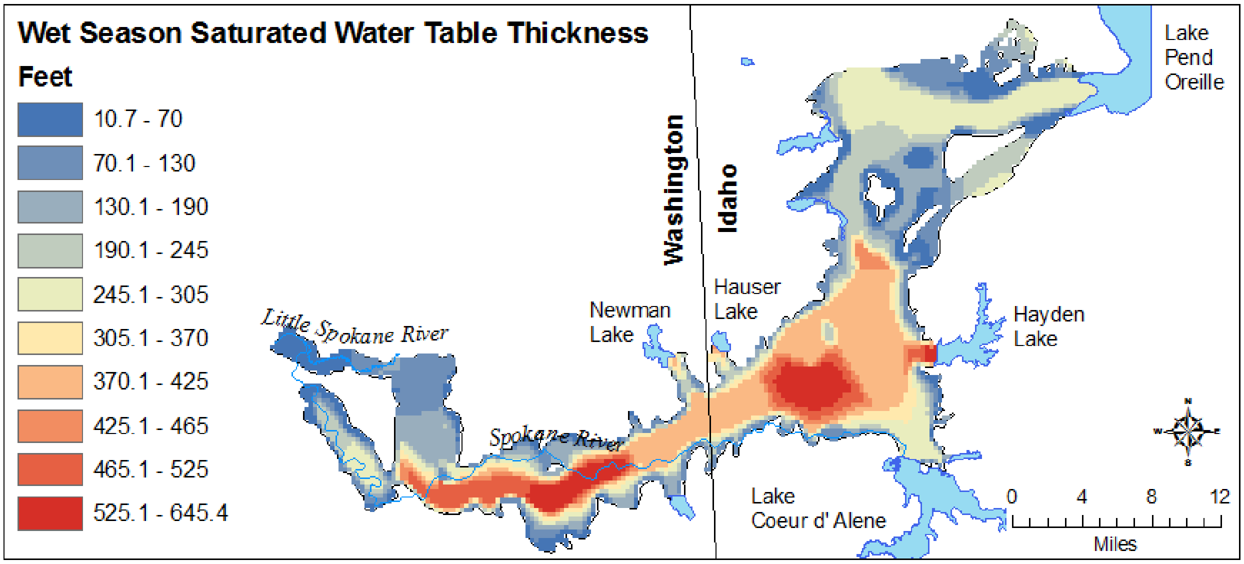

Figure 27. High flow period saturated water table thicknesses.

Differences between the water table elevations depicted on Figures 26 and 27 range from 451 feet near Hayden Lake, Idaho to less than 1 foot near the Little Spokane River just Northwest of Spokane, Washington. The large fluctuations were localized near the outlets for all the lakes and the edges of the SVRP aquifer. The reasons fluctuations were localized near the lakes and other boundary sources of runoff are due to seasonal snow melt and rain fall outflows from the boundary locations. The outflows rapidly infiltrate into the ground because of the high hydraulic conductivity of the SVRP aquifer. So when a wet year occurs, outflows from the lakes or tributary watersheds increase, which increases the seepage into the aquifer that subsequently causes the groundwater table to rise. This groundwater rise could flood basements and quarries. The implementation of an ASR project in the SVRP aquifer would result in further increases in the groundwater table elevation, which could be a limitation to the ASR project if the rise is above natural level, assuming that underground structures and quarries were designed above those naturally occurring levels.

The same dry (August 1994) and wet (May 1997) months were chosen to assess this possible limitation. The depths below ground surface to the water table are shown in Figure 28 and Figure 29. Areas where excessive injections could affect residents and quarries above the SVRP aquifer are highlighted in tan and light red.

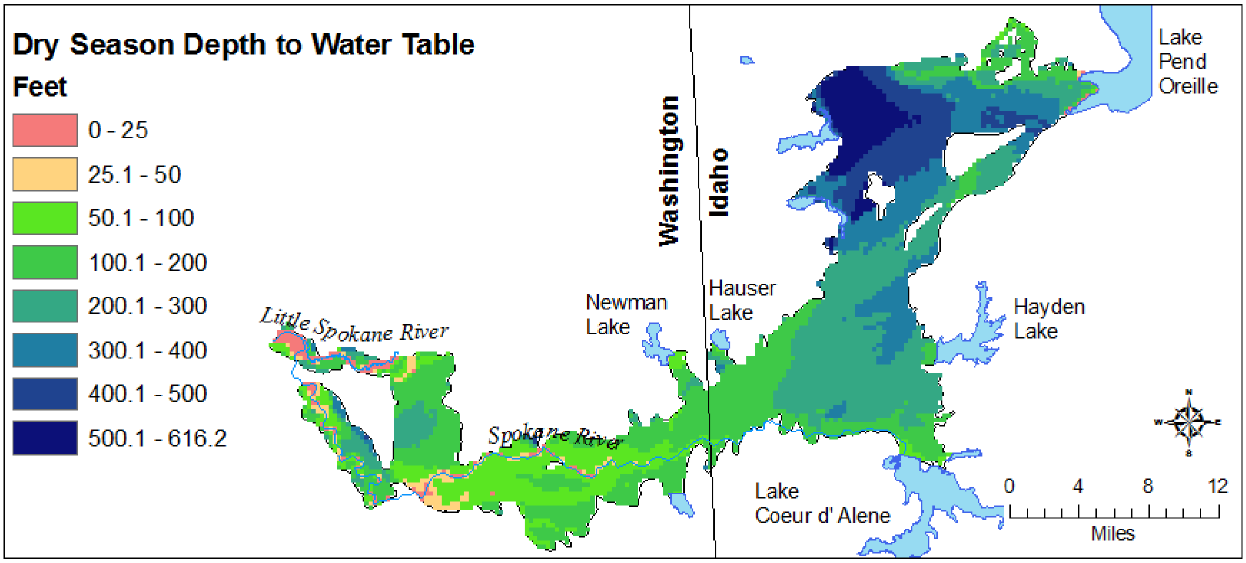

Figure 28. Depth to water table during low flow period.

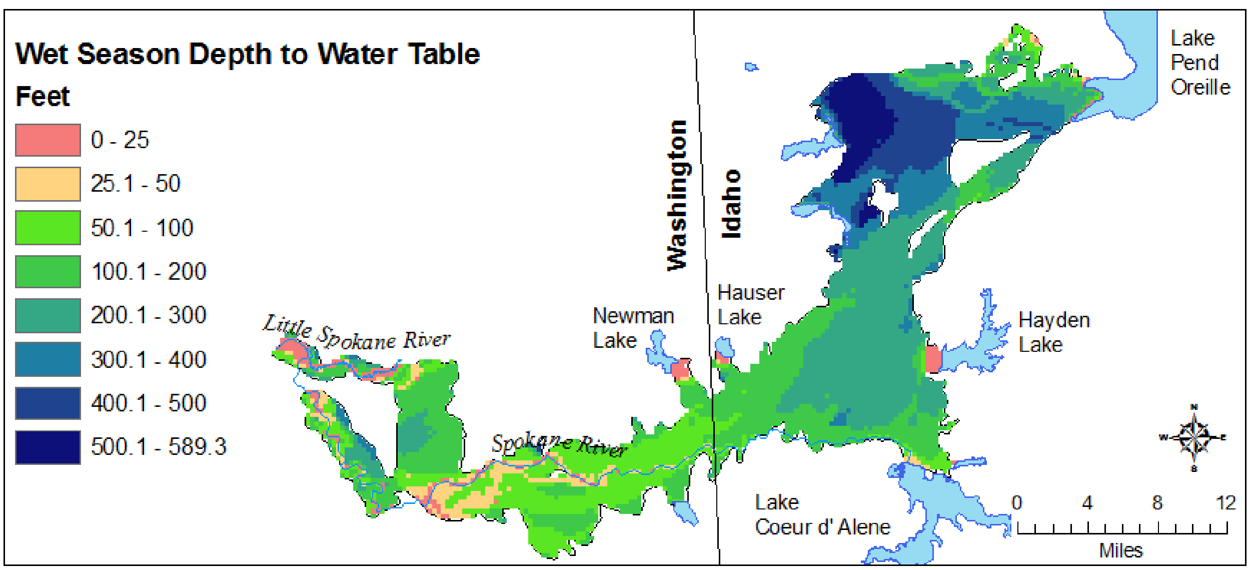

Figure 29. Depth to water table during high flow period.

Based on a shallow depth to groundwater during both low flow and high flow periods, the Little Spokane River area is one potential location where care is needed to ensure the groundwater table does not rise excessively. For the same reason, excessive groundwater mounding along the Spokane River could also be a problem during wet periods. During high flow periods, the regions near Hayden, Hauser, and Newman Lakes are potential areas of concern based on the localized rise in the groundwater levels. This threat, however, disappears fairly rapidly as flows dissipate through the aquifer and inflows cease or are severely reduced during low flow periods. Moreover, considering these lakes have groundwater table elevations about 300 ft above those in the valley floor where the injection sites would be located, excessive additional rises due to ASR injection are not expected near these locations.

Also, the goal of the project is to increase the seasonal low flows. High flows in May are during periods where additional flow from the ASR project to the river will be at a minimum. This means that while some additional flow may occur during the seasonal wet period, the additional peak flows from the ASR project will mostly augment the low flow periods thus limiting the impact on groundwater table rise above natural levels outside the immediate area of the injection well locations.

Potential injection locations in the SVRP aquifer were tested using fifteen different points located along three alternative right-of-way routes (five points per line) that ran from Lake Pend Oreille in Idaho to the Washington-Idaho state line. The scenarios available for each point were: 1) up to seven different injection flow rates ranging from 25 ft^3/s to 300 ft^3/s, 2) four different durations of injection period ranging from 1 month to 4 months, and 3) six potential different months for starting injection. The number of options for each injection location was reduced by determining which starting months did not provide a reasonable time period lag for water to reach the Spokane River in August. This was accomplished using the 25 ft^3/s injection rate to establish which starting month would be used for the 50-300 ft^3/s injection rate scenarios. Also, modeling results indicated that relative groupings of injection well locations (e.g., NR3, PL3, and SR3 or NR5, PL5, and SR5) produced similar return flow results on a monthly stress-period basis so the number of runs were restricted to the most potentially significant (high flow) rates. Overall, 275 possible scenarios were examined rather than the theoretical maximum of 2520. Considering the scenarios available, the most water injected is for the 4 month injection period at a rate of 300 ft^3/s (54,744 acre-ft) and the least amount of water injected is for the 1 month injection length at 25 ft^3/s (1,488 acre-ft). The starting month for each site changes, with sites closest to the Spokane River starting later in the year, a complete table of options and outputs is provided later in Table 8.

The naming convention for the potential injection well sites in the SVRP aquifer is demonstrated in Figure 30. There were three alternative routes; the northern railroad, the powerl line, and the southern railroad. The circled well just east of the Washington-Idaho state line in Figure 30 is located on the Northern Railroad (NR) pipeline. Since it is the fifth well as one moves east to west along the route, it is named NR5. All other wells on the three pipeline routes are numbered in the same manner. Figure 31 shows both the name of the well and the column and row of that well in the MODFLOW model. The column row designation is given for reference to graphs which were developed from the model coordinates.

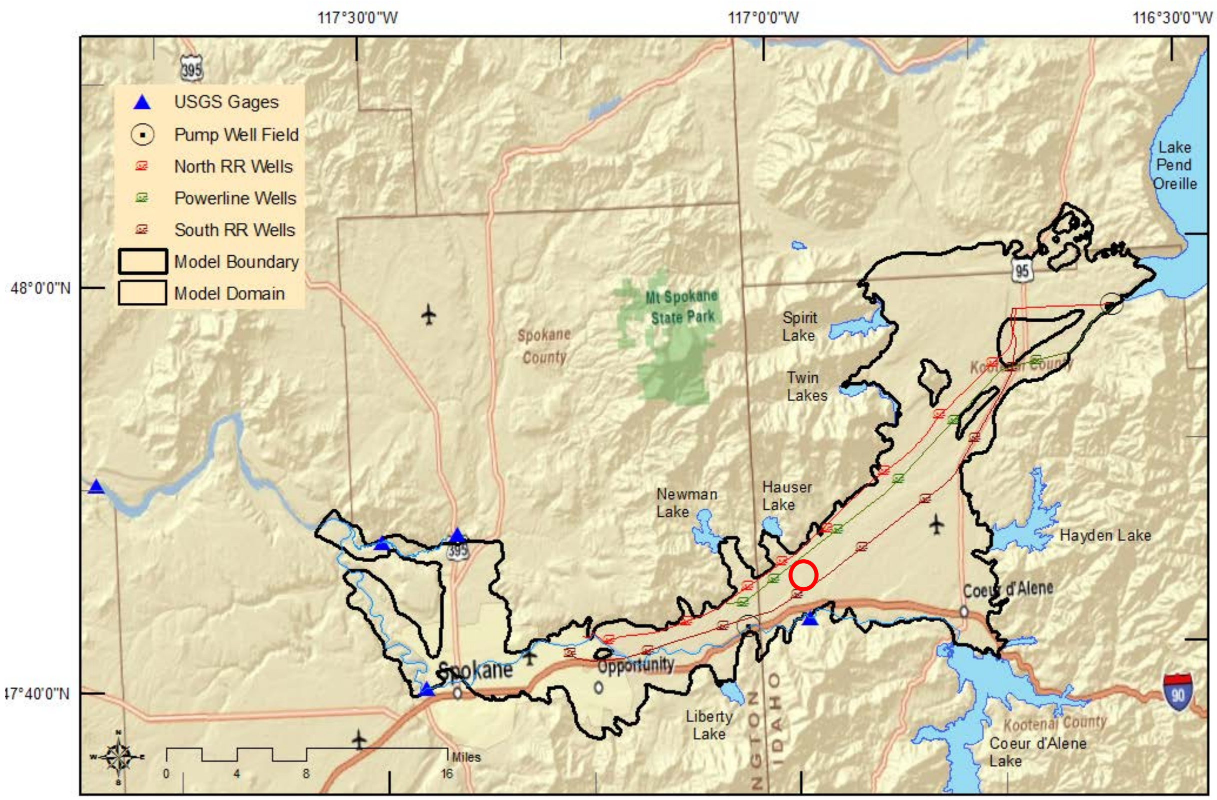

Figure 30. Model injection sites with circled site producing the most groundwater table rise.

Of the suite of options evaluated for this project, the scenario most likely to cause excessive groundwater table mounding is the four month injection period near the Washington-Idaho state line with a constant injection rate of 300 ft^3/s. This is the largest amount of flow tested in any of the scenarios. Well locations closer to the Spokane River return higher peak augmenting flows (discussed in detail in Alternative Evaluations section) with the location nearest the Spokane River yielding the highest peak return. Well locations on the Washington side were not used because of the short lag time for the injected water to reach the Spokane River defeated the goal of increasing August-October stream flows. Figure 30 shows the locations of the injection and pumping wells that were tested with the most likely well injection site to produce excess groundwater table rise circled in red.

To determine the groundwater table rise produced by NR5, the heads predicted by the four month 300 ft^3/s injection scenario were subtracted from the calibrated groundwater table from the MODFLOW-2000 model. Although there were 15 years of data available to use in the SVRP model, only the wettest year (May 1997, as discussed above) was chosen to check for excessive groundwater table rise.

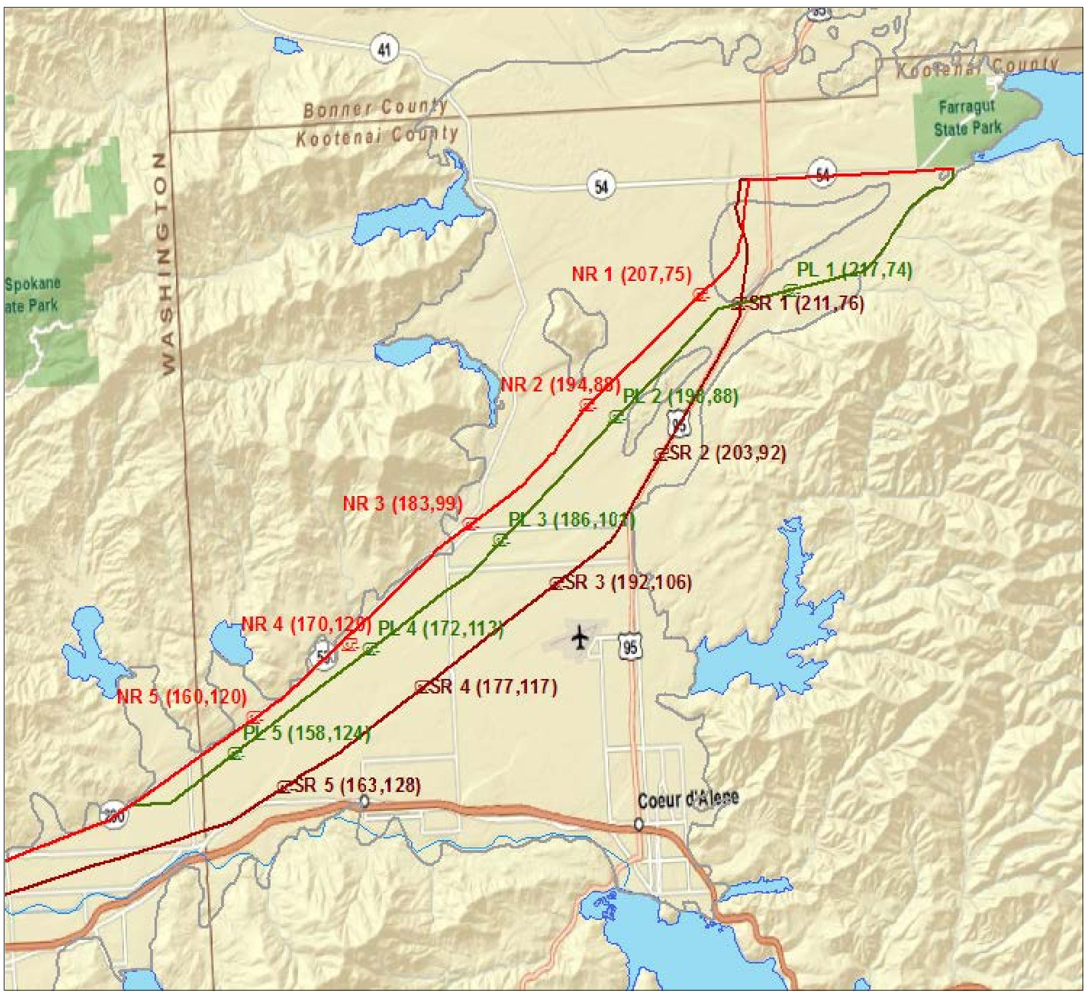

Figure 31. Well name, model coordinates, and right-of-way pipeline routes.

MODFLOW simulation results for August 1997 had high predicted return flows along the Spokane River reaches (see Figure 32). Also notice that the Little Spokane River change is negligible as the line is at or close to the x-axis. The NR5 4-month injection scenario shown in Figure 32 was run from April 1997 through March 1998 to encompass the May-August injection period (Figure 33). The comparisons show the difference in heads between the existing conditions and the 4 month pumping scenario.

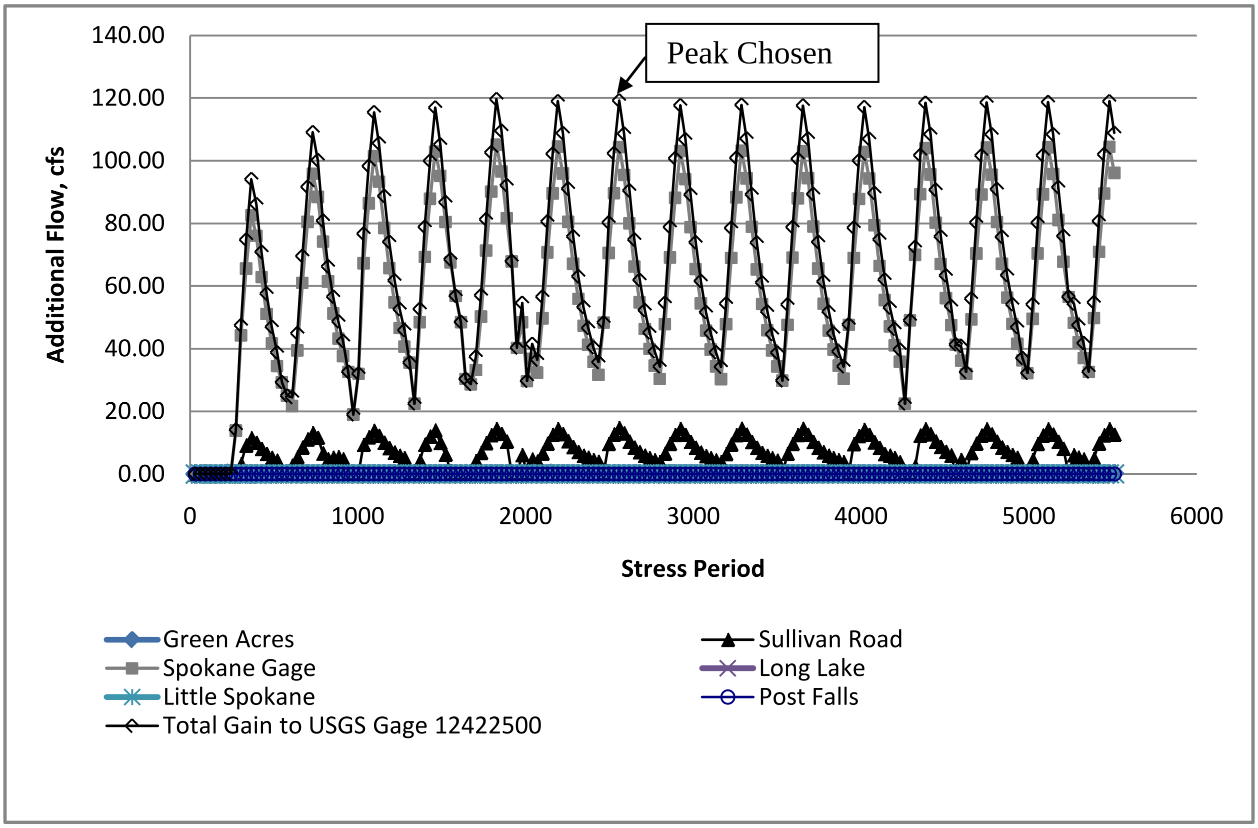

Figure 32. Additional augmentation flows in the Spokane and Little Spokane Rivers for NR5.

Figure 32 is used to identify how wet year 1997 impacted flow from the SVRP aquifer to the Spokane River. It also demonstrates how the different zones for the Spokane River react to additional flow. As is seen from Figure 32, the Green Acres, Little Spokane, Long Lake, and Post Falls zones do not show significant increases in flow. However, the Sullivan Road and Spokane Gage (USGS 12422500) zones show significant increase in flow. The total gain to the USGS Spokane Gage (12422500) is the sum of the zones upstream of that gage.

The progression of months shows the head produced by the injection well peaks considerably east of the areas of concern along the Spokane and Little Spokane Rivers and does not have a large affect near Newman, Hauser, or Hayden lakes. August 1997 shows the maximum extent of the lateral movement of the mound which is widespread and provides additional baseflow for the Spokane River throughout the fall months and into winter.

The large lateral extent of the groundwater mound caused from the injection period is a consequence of the high hydraulic conductivity. For typical ASR systems the movement of water would pose a challenge as the water could move away from the injection site and possibly out of the system before it could be recovered via pumping wells later in the season. For the SVRP ASR project, the high aquifer hydraulic conductivity allows the water to flow to the natural discharge areas in the Spokane and Little Spokane Rivers. The limitation imposed by the speed in which the water moves through the aquifer is offset by timing the injection such that the peak increase will occur in August. As Figure 30 shows, the injection sites vary in distance from the Spokane River. By varying the starting time of the injection, the peak return can be set to arrive at the Spokane River in August. However, wells located in Washington State (west of NR5) do not provide sufficient lag times for the injected flows and return flows to the rivers occur during the wet season thus defeating the primary purpose of the ASR project. Consequently, one limitation to the system is that injection wells will need to be located in Idaho.

Storage potential in the SVRP occurs on an almost yearly cycle. The water is not stored for long periods of time but rather moves through the aquifer and is discharged into the Spokane and Little Spokane Rivers in as little as a few days depending on where in the aquifer the water originated. The depth to the top of the aquifer (the unsaturated zone) in the northern areas would provide potential to temporally store considerable amounts of water as seen in Figure 28 and Figure 29. The permeable nature of the aquifer and the existing groundwater gradients however, would likely cause some of the water to drain towards Lake Pend Oreille or the Pend Oreille River.

Model-predicted head rise from injection of 300 ft^3/s over four months is less than 11 feet at its maximum, see Figure 33. At this injection location, NR5, depth to water is over 200 feet. Head rise in areas of the model that have shallow depth to water values are calculated to be minimal (< 1.5’) and should not cause problems for property owners.

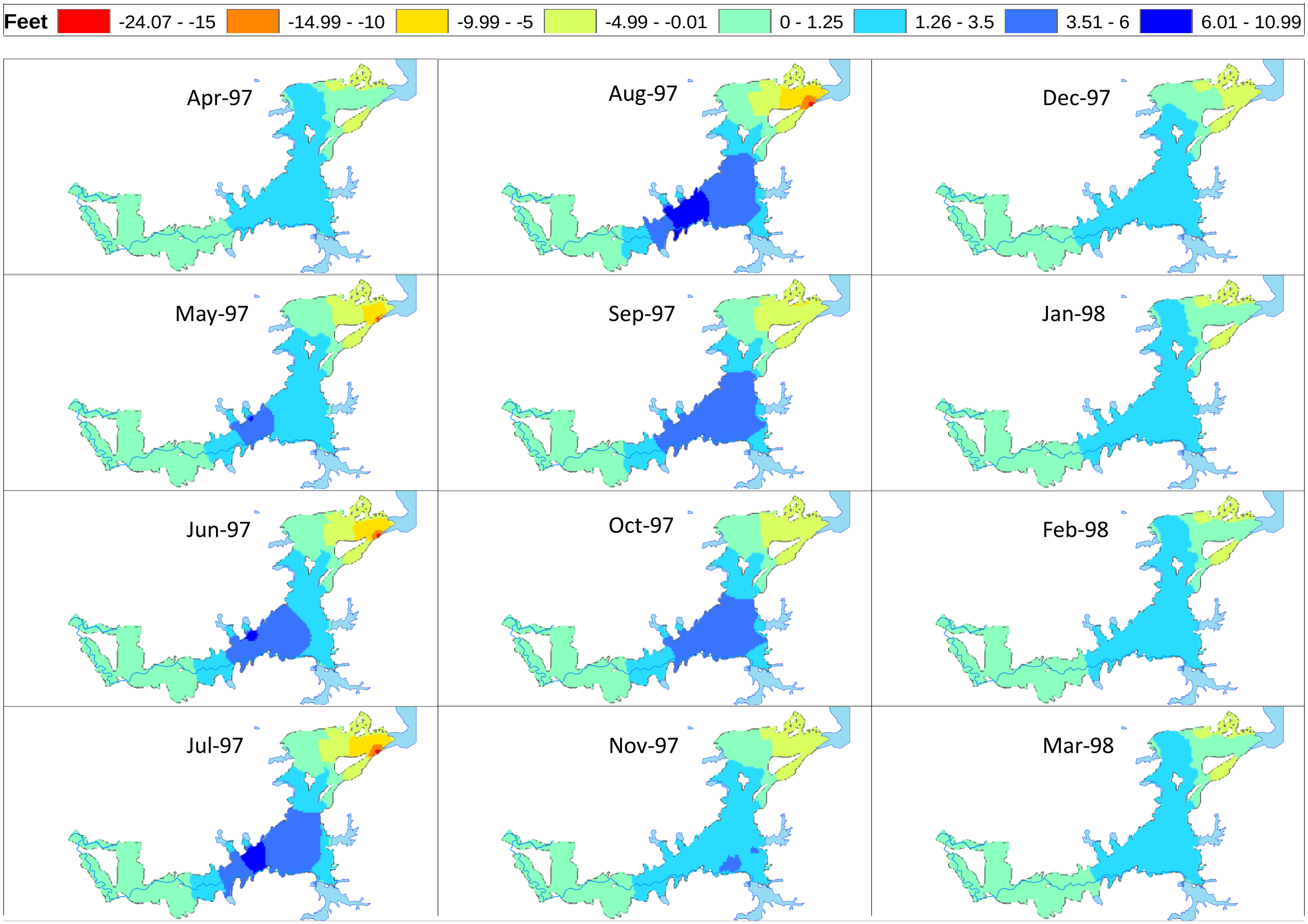

Figure 33. Maximum mound height from injection of 300 ft^3/s for 4 months at NR5.

4.2 Stream-Aquifer Interactions

Discharge limitations in the Spokane and Little Spokane Rivers are controlled by Darcy's Law and limited by the elevation difference between the surface of the groundwater table and river surface elevation. Darcy's Law in MODFLOW is expressed as:

$$Q = -K\ A\left(\frac{h_A - h_B}{L}\right) \quad = \quad -K\ A\ \left(\frac{h_A - h_B}{M}\right) \qquad (1)$$

where Q is the discharge [ft^3/day], K is the hydraulic conductivity [ft/day], A is the cross-sectional area [ft^2], M is the thickness of the streambed material [ft], h_A is the water surface elevation in the aquifer [ft], and h_B is the water surface elevation in the river [ft].

The negative sign in equation (1) indicates flows moves in the direction of decreasing hydraulic gradient. If the river elevation is higher than the aquifer (e.g., h_B is more than h_A), then water flows out of the river and into the aquifer. Conversely, if the river stage is below the aquifer, then water enters the stream from the aquifer. Figure 34 illustrates a physical representation of this latter concept. The limitations for the SVRP aquifer-river interaction are the head values in the river (h_B) and the aquifer (h_A) as well as the streambed/aquifer interaction parameters; hydraulic conductivity (K), contact surface area (A), and the streambed thickness (M). In MODFLOW the interaction parameters are combined into a calibration parameter referred to as the conductance (C). Hsieh et. al. (2007) determined eleven different zones of streambed conductance for the Spokane River and one zone for the Little Spokane through model calibration. For the scenarios tested in this study, no limitation to additional flow into or out of the Spokane and Little Spokane Rivers was identified.

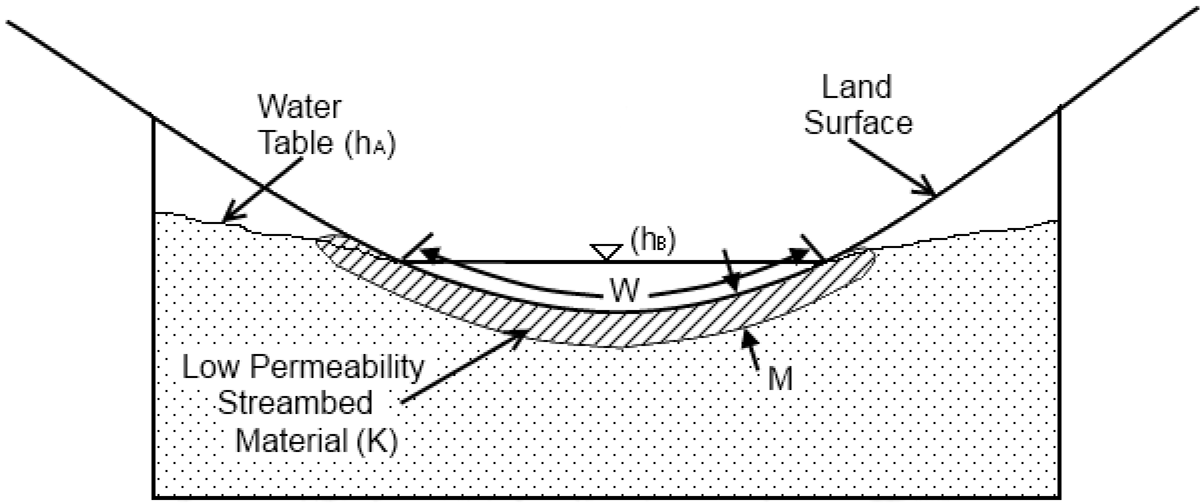

Figure 34. SVRP aquifer-streambed configuration

The interaction between the existing groundwater and the injected water was also considered. To help identify the differences been the two, scenarios focused on pumping the injected water from the aquifer itself verses pumping directly from a surface water body. This also reduces the cost of water treatment for the ASR project as well. The CE-QUAL-W2 model, originally developed for Ecology's 2001 TMDL study, was used to determine how the levels of important minerals and nutrients react to increased return flows and little to no negative impacts were observed. Details on water quality modeling are presented later in this discussion.

The timing of water availability is also a limiting factor for the SVRP ASR project. As stated above, even though direct diversion from surface water bodies was not the main scenario considered, pumping in the aquifer near a surface water body has nearly an immediate impact (little to no lag time) and therefore decreases the amount of water available for in-stream flows. Figure 35 shows how the cone of depression around a well can extend to the steam and start to draw water directly from both the surface water body and the groundwater that would have eventually reached the stream. Because of these relationships, the amount of freshwater available for diversion from the river/lake sources was used as a limiting factor. This and the reliability of having a full supply for extraction/injection were examined in Chapter 3.

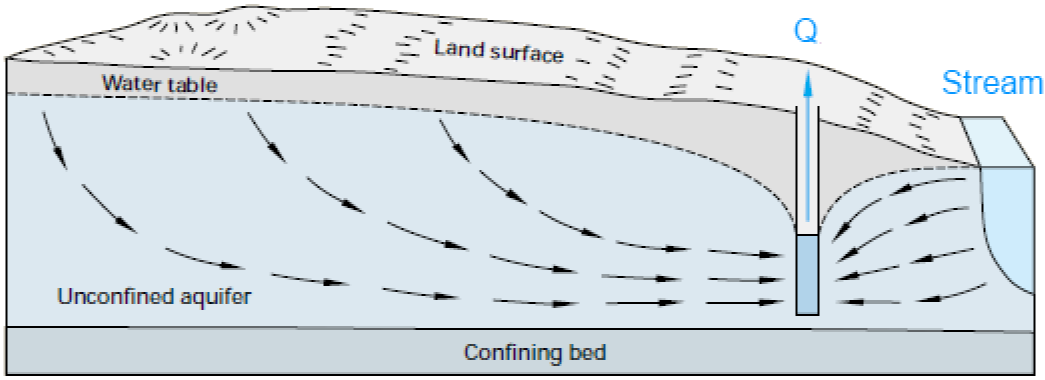

Figure 35. Pumping well-stream interaction

(Image from "Ground Water and Surface Water, A Single Resource" Winter et. al. (1998))

The impacts of well drawdown near Lake Pend Oreille on percentages of aquifer versus subsurface lake interaction were documented using a number of different scenarios. Figure 36 and Figure 37 show that injection period lengths (1 to 4 months) and injection rates (25 to 300 ft^3/s) do not cause any significant differences in percentages of water withdrawn from Lake Pend Oreille. The 300 ft^3/s scenario shown in Figure 37 draws slightly more water than the other scenarios but this could be due to the use of two pumping wells each withdrawing 150 ft^3/s instead of just a single pumping well. Also shown in Figure 37 is that the length of the injection period has no affect on the percent withdrawn from Lake Pend Oreille. This allows a three month injection/pumping scenario to be used at all five locations along the NR pipeline corridor. Since the NR1 location did not have a zone budget set up when the two month scenario was completed, the three month scenario was substituted instead. This was considered an acceptable approach since the location was deemed unacceptable due to the amount of water flowing back towards Lake Pend Oreille. Also, Figures 36-38 show sudden changes in 2005 percentages for all injection periods regardless of locations. These sudden changes occur because in 2005 the model was only designed to run until September so the additional flow that would have filled the depression made by the pumping well had not been given a fill pumping cycle to be replenished. In other words, the decreases are a modeling artifact rather than expected results.

The actual amount of water pumped from Lake Pend Oreille depends on the exact location of the injection well. Figure 38 shows that as injection sites move closer to the pumping well field near Lake Pend Oreille, the amount of induced flow drawn from Lake Pend Oreille is decreased. This is because a portion of the water injected near the extraction wells tends to form a loop, so less new water from Lake Pend Oreille is needed to fill the pump demand. This also decreases the amount of water that flows toward the Spokane River.

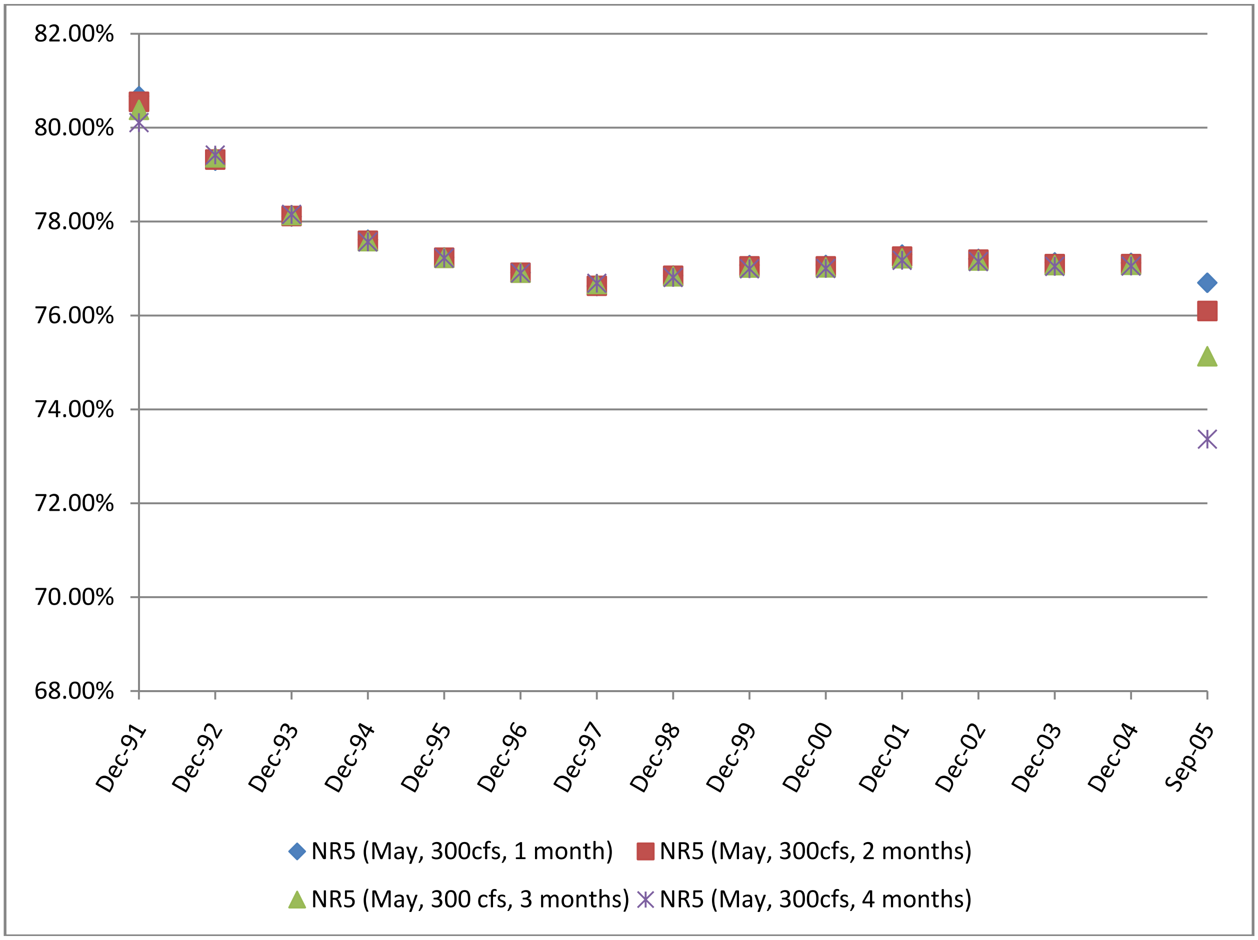

Figure 36. Effects of withdrawal period length on percentage of surface water used.

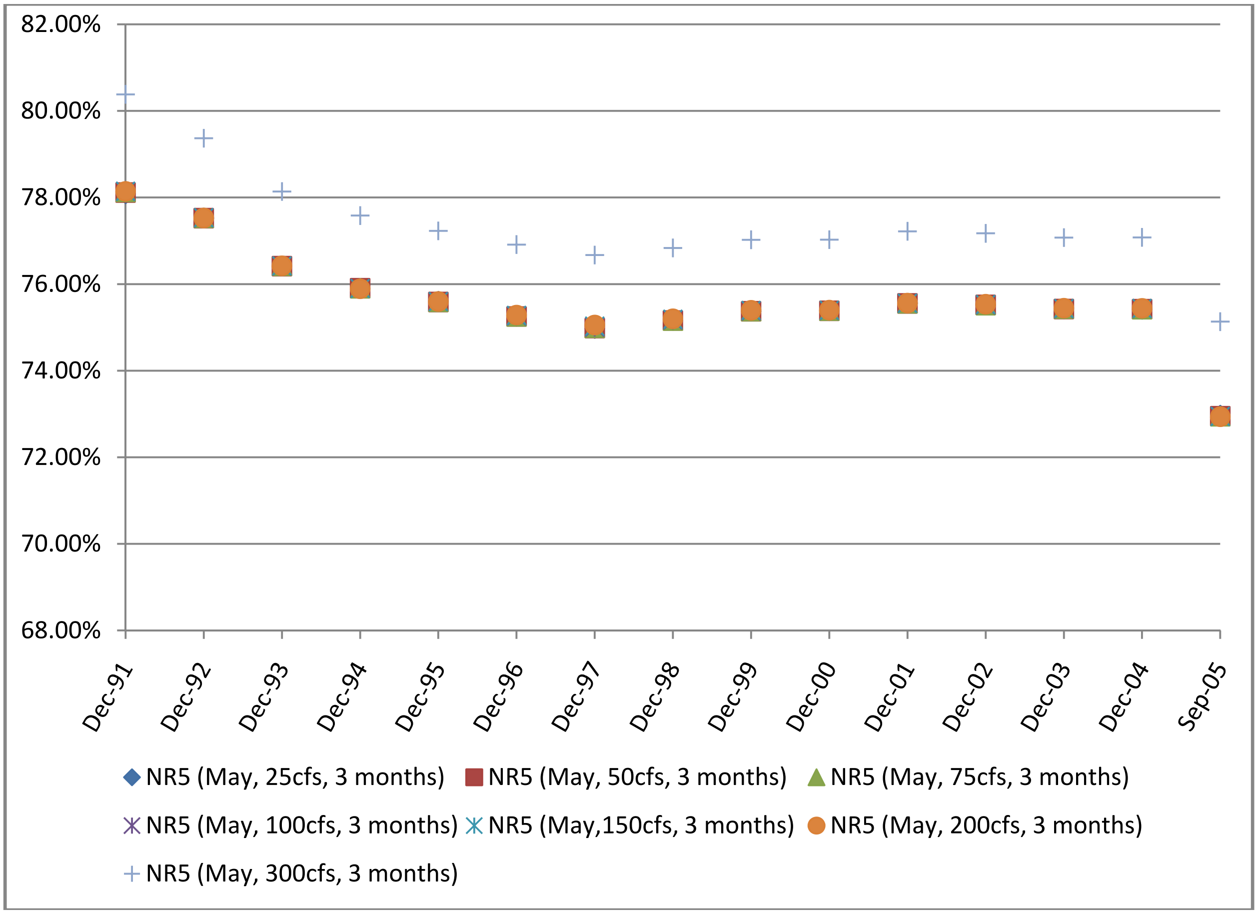

Figure 37. Total yearly percentage of induced flows captured from Lake Pend Oreille as a function of injection rate (ft^3/s).

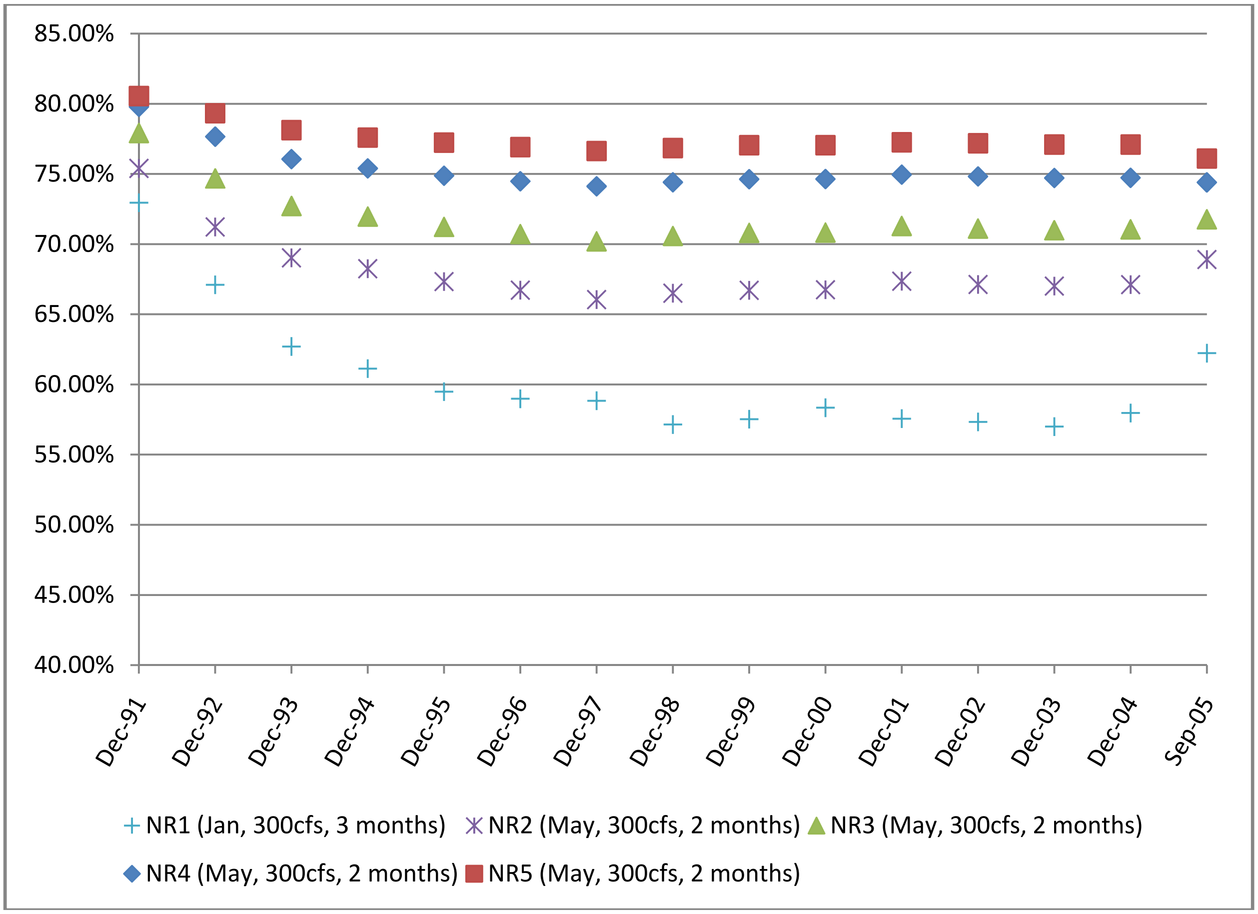

Figure 38. Total yearly percentage of induced flow captured from Lake Pend Oreille as a function of injection location.

4.3 Existing Public Water Systems

According to a document provided by Idaho Department of Environmental Quality (IDEQ 2010), there are 244 Public Water System (PWS) wells in the Idaho portion of the SVRP area acknowledged by the Idaho Department of Water Resources (IDWR) which need to be considered in the placement of the proposed injection wells. The "Public Drinking Water Well Site Evaluation" check list from the IDEQ defines a one mile radius around each PWS as the zone of evaluation for potential impacts from new ASR projects. Figure **39** shows that 37 PWS wells have been identified within a one mile radius of the proposed injection wells. The minimum distance from a PWS well to the proposed ASR wells in Figure 39 is 412 feet. With the maximum water table elevation increase of 11 feet, which is less than the 20-foot water table fluctuations between the wettest and driest years in that area, it would appear the water table mound produced by the ASR well is within the natural limits of the aquifer. As long as water quality is not degraded by the increased injection it would seem the locations of the proposed ASR wells are within the minimum standards set forth by IDWR and IDEQ.

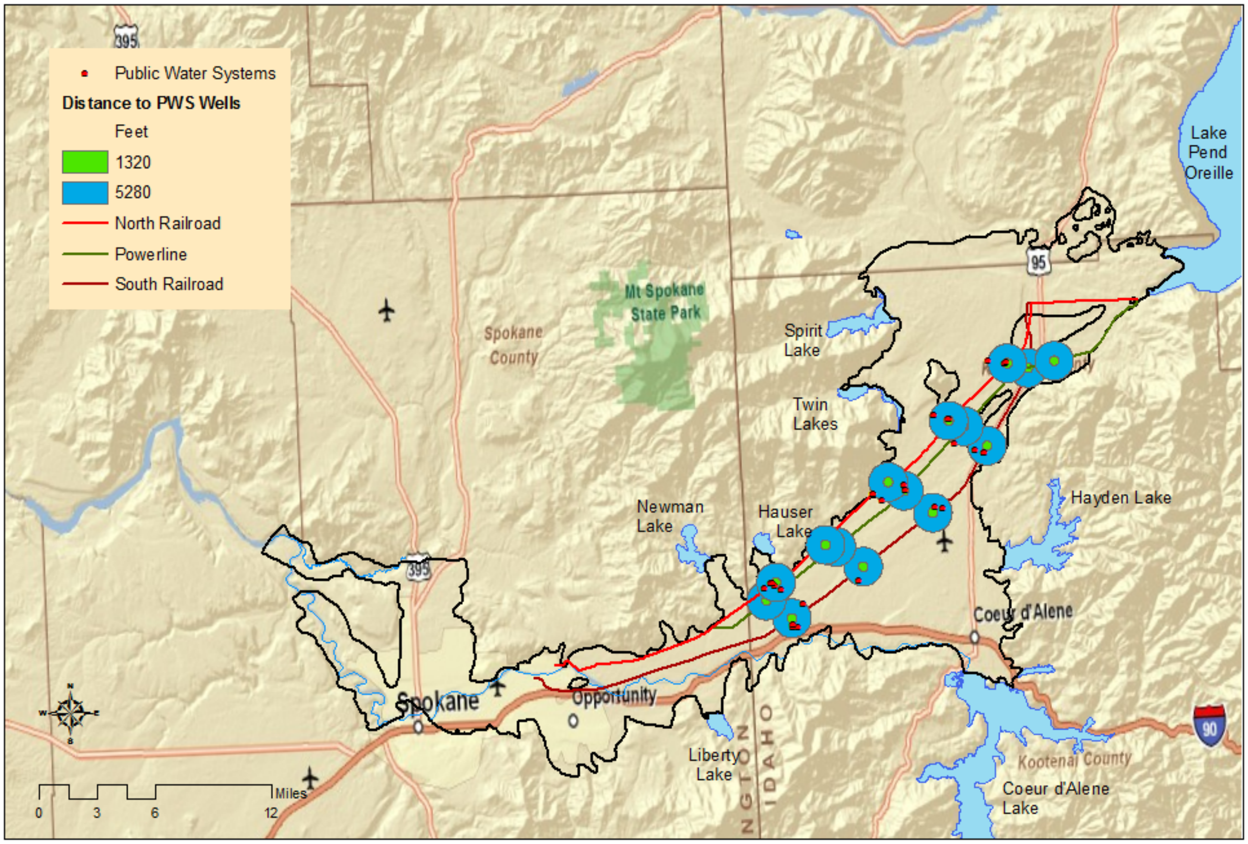

Figure 39. Potential conflicts between Public Water System wells and proposed injection wells.

5.0 Target Design Objective

Preliminary modeling work for 25 ft^3/s infiltration basin and injection well facilities placed at three arbitrarily selected locations in the SVRP demonstrated that there was a potential to increase summer flows at the Spokane River gage (Barber et al. 2009). This is illustrated below in Figure 40 where peak increases of 30% of the injected amount were seen in the river. However, in the preliminary analysis, other alternative locations examined were not examined nor were other flow rates analyzed. Additionally, no attempts were made to match the value of these increases in terms of potential downstream Columbia Basin demand. This study and report builds upon this preliminary analysis by exploring alternative locations and additional flow rates. These will be important components and considerations of this project.

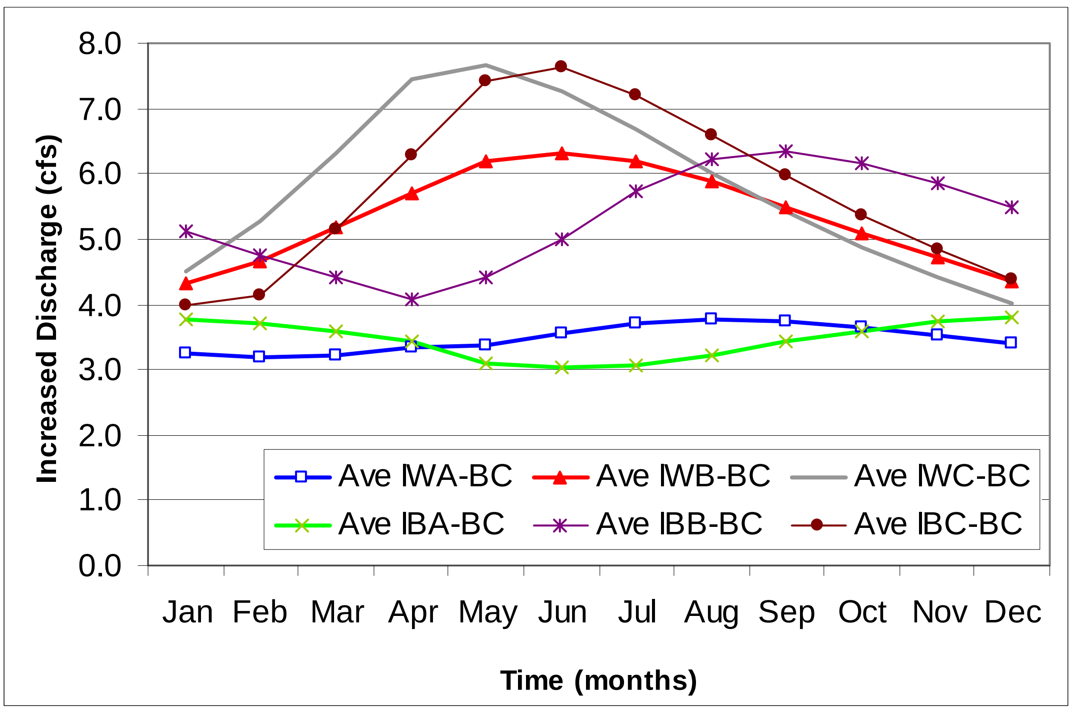

Figure 40. Lag time impacts on average monthly flows at the Spokane River at Spokane gage.

Typically, by comparing water needs versus system limitations, a preferred Target Design Objective can be determined. In this case, however, it is somewhat difficult to adopt this strategy without looking at cost and acknowledging that this one project will not likely satisfy all of the downstream demands. Furthermore, while the initial objective of this project was to analyze the feasibility of a full-scale project aimed at maximizing water supplies using two to four incremental projects (those associated with the difference between the Target Design Objective and those of smaller options), recent studies and reports identifying future regional demand increases quickly led to larger flow alternatives. As we summarize the ASR project's incremental costs in terms of both development investment and appropriate operation and maintenance costs in the following sections, more emphasis is placed on construction of a larger facility. As results became available and in discussion with Ecology staff, the preferred design alternative ended up focusing on a series of three extraction flow alternatives (100, 200, and 300 ft^3/s) based on estimates of flow increases versus increases in cost. While lower flow scenarios were examined, the impact to the Spokane River in terms of monthly flow increases becomes quite small and so the economic justification becomes questionable.

Conversely, it appears that large diversions could be made from the Lake Pend Oreille area. However, at flows of 500-1000 ft^3/s, drawdown becomes excessive such that groundwater pumping becomes infeasible and the impacts on other wells in the area would be significant. Surface water withdrawals, treatment, and a canal system would likely be required at this point. This would involve a comprehensive assessment of source and aquifer water quality, treatment costs, and canal designs that were beyond the scope of this project.

6.0 Alternative Evaluation

This chapter provides a brief overview of the MODFLOW groundwater model and a detailed discussion of the framework used to evaluate each potential ASR solution. This work entailed examining the water availability from each source to determine feasible diversion windows, predicting the ability to satisfy monthly demand projections as a result of additional water quantities, and determining pumping requirements and other engineering considerations associated with pipelines, diversion/injection structures, routes, and water treatment plants (if necessary). The impacts of recharge duration scenarios were included to determine the minimum diversion amounts necessary to reach given flow targets. For instance, diverting and injecting flows from December through April at one location may produce the same increase in summer streamflow as February through March at another location. Trade-offs between transmission pipeline length, diameter, and pumping requirements were also examined. In all, over 275 spatial variations were examined to determine the impacts of diversion quantities and injection well locations. Each of the spatial variation tests also included numerous temporal pumping variations in order to obtain a complete understanding of system response. Reoccurring patterns and linear trends permitted fewer runs than would have been needed to explore all of the possible alternatives.

6.1 Description of the MODFLOW Groundwater Model

MODFLOW stands for MODular three-dimensional groundwater FLOW model developed by the U.S. Geological Survey (McDonald and Harbaugh, 1984). The model employs a finite-difference algorithm to solve the groundwater equation. MODFLOW uses modular programming components referred to as "packages" to simulate specific aspects of groundwater flow systems. The model can simulate steady and nonsteady flows in irregularly shaped flow systems in which the aquifer layers can be confined, unconfined, or a combination of confined and unconfined. Since its inception, the model has undergone four major releases (updates) in 1988, 1996, 2000, and 2005. When the original bi-state model was constructed (Hseih et al. 2007), MODFLOW-2000 was used because it was widely available and it has a superior numerical solver not included in MODFLOW-2005 because of licensing restrictions. For additional details, readers are referred to the USGS MODFLOW website at: http://water.usgs.gov/nrp/gwsoftware/modflow.html.

Because of the popularity of the MODFLOW model, a number of graphical user interfaces (GUIs) have been developed. GUIs do not impact solution of the flow equation rather they enable users to seamlessly switch between model modules to build or modify the model input parameters, run simulations, and display results. The industry standard is Visual MODFLOW.

6.1.1 Update from MODFLOW-2000 to Visual MODFLOW-2009

The original bi-state SVRP groundwater model was developed using MODFLOW-2000 (Harbaugh et al. 2000) in a collaborative effort between the Idaho Department of Water Resources, Washington State Department of Ecology, University of Idaho, Washington State University, and US Geological Survey (Hsieh et al. 2007). As described in previous chapters, this bi-state model has been subsequently used by Wylie (2010) to evaluate Idaho's future impact on the Spokane River and by Barber et al. (2009) to demonstrate the preliminary feasibility of this ASR study. During the preliminary feasibility study, researchers had difficulty extracting seepage information along the Little Spokane River reaches. The FORTRAN code developed to compile the Spokane River fluxes derived from the Stream Flow Routing (SFR) package did not work with the River Package (RP) used to simulate the Little Spokane River. It was determined that consistent handling of the Spokane and the Little Spokane Rivers would enable better interpretation of stream-aquifer interactions. After comparing features and capabilities of MODFLOW-2000 and Visual MODFLOW-2009, we elected to use Visual MODFLOW-2009. The 2009 version of Visual MODFLOW is equipped with a user friendly interface that allows for updating scenario input values quickly and efficiently. With 5,203 wells in the model, manually updating the scenario values would have taken large amounts of time and would have made error checking difficult. Visual MODFLOW-2009 (VM-2009) also has a well database which can be modified from within the program, speeding up the time for changing scenario inputs and it has a graphical visualization program that allows the user to view aquifer outputs and create cross-sections and other meaningful visual aids for examining the data.

In order to update and use the newer VM-2009 model, we had to convert two of the original MODFLOW-2000 data files to packages supported by VM-2009 and then conduct a thorough comparison of both MODFLOW models to ensure they yielded the same results. The following sections of this report provide the details of the conversion and evaluation.

6.1.2 Conversion to Visual MODFLOW

Visual MODFLOW-2009 does not support the Stream Flow Routing (SFR) or the Flow Head Boundary (FHB) packages that were both used in the original MODFLOW-2000 model. The SFR package used to simulate flows in the Spokane River was converted to the River package (RP) in the new VM-2009 model. The SFR package and the RP both calculate flux between surface water bodies and the aquifer using Darcy's Law, but the SFR package has the added flexibility to allow the conductance term to change as flow in the stream changes for some of the package options. The SFR package allows users to input a flow for the river and has the ability to use up to five methods to calculate the depth and width of the river from the flow and the cross-section. The five methods are: (1) specifying a depth at the beginning of each stress period for the first and last reach (cell) of a segment and linearly interpolating between the two points, (2) using Manning's equation assuming a wide rectangular channel, (3) using Manning's equation assuming an eight-point cross section for each segment and using a bisection-secant method used to determine depth of stream, (4) estimating stream depth and width using a power function relating each to streamflow, and (5) calculating stream depth and width through linear interpolation of values bracketing the calculated value using a table relating streamflow to depth and width.

VM-2009 supports the River package stream-aquifer routine. As mentioned above, the RP was used to model the Little Spokane River in the MODFLOW-2000 model and is similar to the SFR package except that only option (1) is available in the River package. This widely-used approach does not allow for the conductance factor to vary with changes in streamflow. This is because the conductance factor is calculated from the length of a reach in a cell, the width of the river in the cell, the thickness of the riverbed, and the vertical hydraulic conductivity of the riverbed materials. Since all four of these values are constant in option (1), varying the conductance factor as a function of streamflow is not possible. While the RP is less versatile than the SFR package, the original model specified option 1 in the SFR package. Therefore, converting to the RP did not constitute a major change in methodology.

The FHB package is designed to simulate transient head and flow conditions in an aquifer without the need for additional stress periods. The FHB package was used to simulate the flow into the SVRP aquifer from tributary basins and all the lakes surrounding the SVRP aquifer except Lake Pend Oreille and Lake Coeur d'Alene. Since VM-2009 does not support the FHB package, the flows simulated by the FHB package were added into the recharge layer. This was done by dividing the flow to the cell from the FHB package by the area of the cell to obtain a flow volume per unit time in the cell.

Changes to the input data caused by using the recharge layer and RP verses the more robust FHB and SRF packages were analyzed for consistency to ensure output data from Visual MODFLOW-2009 was reasonably similar to results from the MODFLOW-2000 model. To test the two models, input data from the calibrated SVRP model developed by Hsieh et al. (2007) was used in both the MODFLOW-2000 and the modified VM-2009 models. The MODFLOW-2000 version was simply rerun in order to have the output data available, while the VM version was run for the first time using the calibrated data. The calibrated MODFLOW-2000 input data will henceforth be referred to as the "existing conditions."

VM-2009 and MODFLOW-2000 use different methods to solve the underlying groundwater finite-difference equation. As such, a total of about eighteen variations of the solver and the solver parameters used were modified to determine the best match to the original model. In the original model, the Preconditioned Conjugate-Gradient (PCG) package was used to solve the finite-difference equation. In VM-2009 there are six different solver packages available, which are: Preconditioned Conjugate Gradient Package (PCG2), Strongly Implicit Procedure Package (SIP), Slice-Successive Overrelaxation Package (SOR), WHS Solver for Visual MODFLOW (WHS), Algebraic Multigrid Method for Systems (SAMG) and Algebraic Multigrid Solver (AMG), and Geometric Multigrid Solver (GMG). Each solver had slightly different parameter available to solve the finite-difference equation. After analysis, it was found that the SAMG solver provided the closest match to the MODFLOW-2000 results and was considerably quicker than other solvers. For a discussion of types of solvers used in VM-2009 and their potential errors, see the study conducted by Osiensky and Williams (1997) which also discusses the PCG2 solver. Additionally VM-2009 allows the user to pick an initial head value for the model from other MODFLOW runs. The head values from stress period one in the MODFLOW-2000 version were chosen as the initial head values in VM-2009 to match as close as possible the conditions in MODFLOW-2000. All other settings in VM-2009 were consistent with the MODFLOW-2000 model.

While conducting runs for determining the solver parameters it was discovered that during the importing process from MODFLOW-2000, wells that were inactive during winter stress periods would be given small pumping or injection values that were not present in the MODFLOW-2000 version. The remedy for the issue was found by specifying a zero pumping rate for all inactive wells in the model, which was not needed in MODFLOW-2000.

To determine the differences between the two models, ArcGIS was used to develop head difference maps for the predicted water table elevations in the SVRP aquifer. Figure 41 shows seven representative stress periods out of the 181 that were available.

The output from the original MODFLOW-2000 model was subtracted from output from the VM-2009 model to determine the difference between the models. The maximum head difference ranged from 2.1 ft to -1.0 ft. The differences on the negative end of the range were -0.2 ft or less for most stress periods except the first stress period which was -1.0 ft. The highest positive difference between the models was 2.1 ft. Differences did not drop much below 1.8 ft, seen in stress period 120, and then came back up to about 2.0 ft for later stress periods. Overall, predicted water levels using VM-2009 seemed to be a little lower in the middle of the aquifer near Lake Coeur d'Alene and near Rathdrum and very similar near the eastern and western boundaries of the aquifer. As Figure 42 shows, the difference between model versions in the majority of the area near Lake Coeur d'Alene is between 1.26 to 1.56 feet and reaches 2.0 feet in only a very small region near the outflow of Lake Coeur d'Alene to the Spokane River. The area where model differences are between 1.26 to 1.56 feet around the Spokane River is also a perched aquifer, which does not directly interact with the Spokane River. When the Spokane River starts to influence and be influenced by the aquifer near the Washington-Idaho state line, the difference between the two models starts to reduce. The water in the Spokane River and the SVRP aquifer are virtually at the same elevation as the Spokane River turns northward about 7 miles from the state line. It is in this reach of the Spokane River that most of the ASR Project water is recovered and the difference between the two models ranges from 0 to 0.5 feet. Since the critical areas of interest for this project are the Spokane River and Little Spokane River where the aquifer and the rivers interact, results were deemed acceptable.

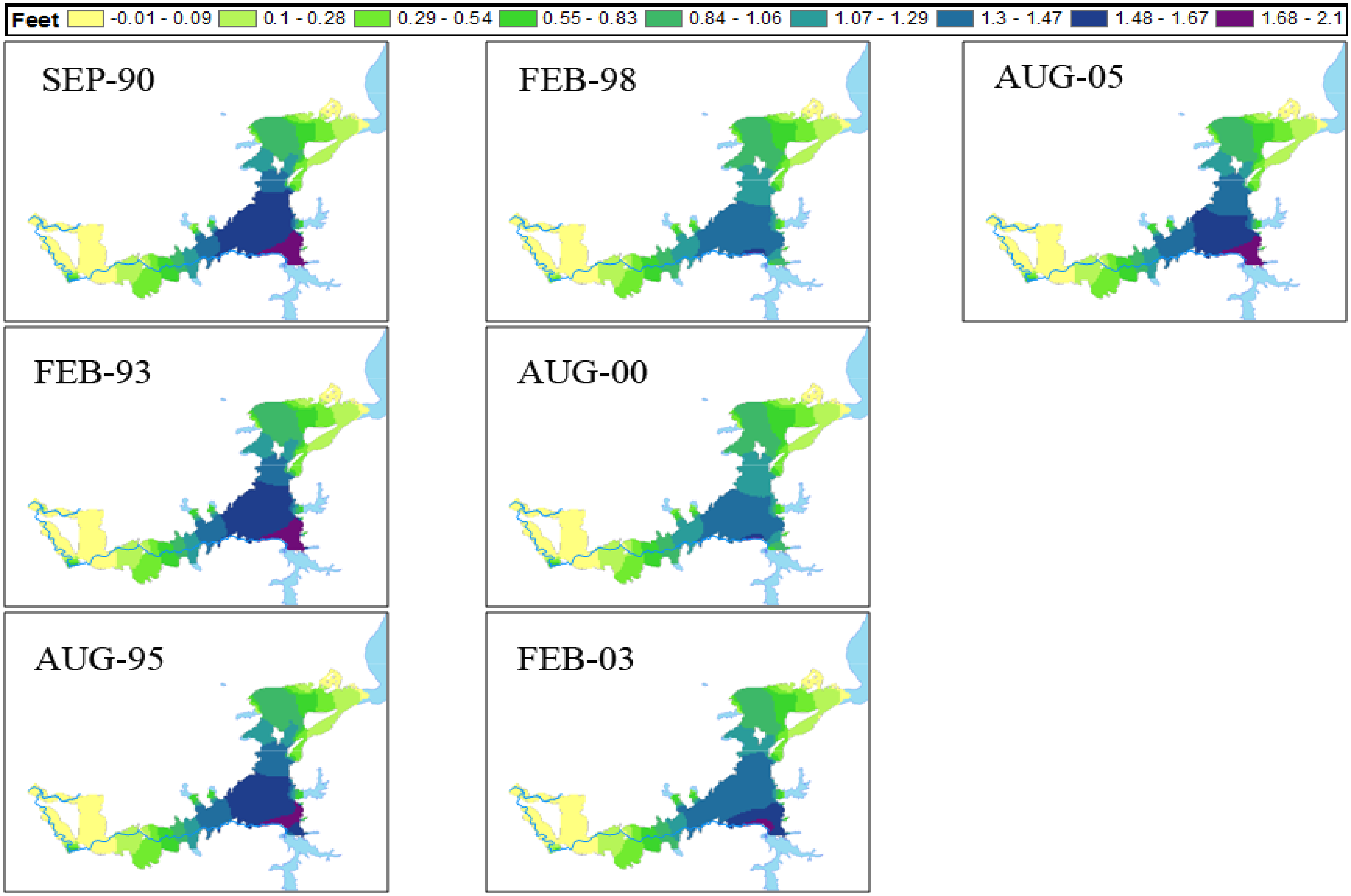

Figure 41. MODFLOW-2000 and Visual MODFLOW-2009 model differences.

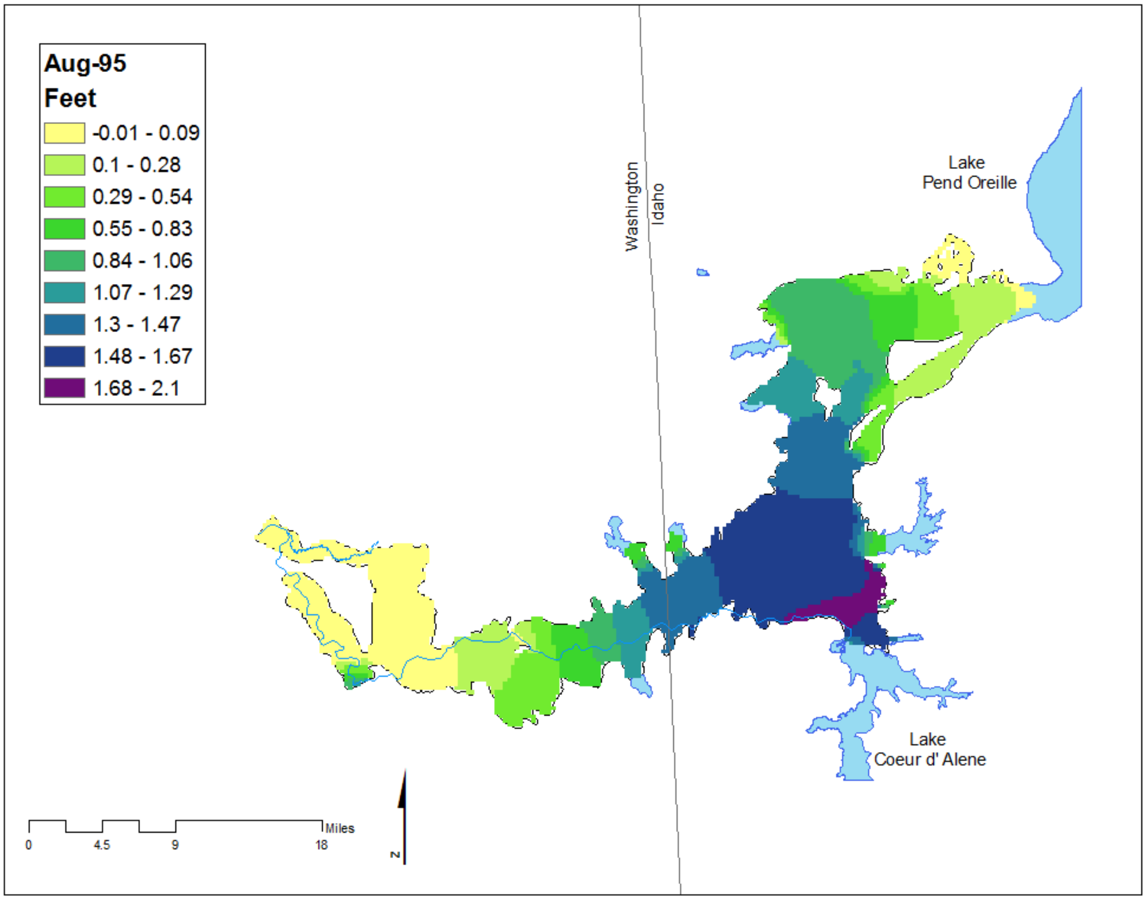

Figure 42. Head differences between the MODFLOW-2000 and Visual MODFLOW-2009 models, August 1995.

While the agreement between head values near the Spokane River is reasonable, the effect of the interaction of the flow crossing the riverbed from the aquifer still needs to be determined. To determine the actual difference in the amount of flow exchanged between the Spokane River and the aquifer, the "zone budget" from MODFLOW-2000 was used to create an identical zone budget for VM-2009. For the MODFLOW-2000 model, the cells used for the Spokane River in the SFR package, which includes a zone budget tool that gives the output values of flow into and out of a river, was used to define the zones. For VM-2009, the RP does not specifically include a zone budget option although a separate package called Zone Budget exists which provides the same function. The zones used to compare the flows into and out of each zone are shown in Figure 43. Since the Zone Budget package is also available in

MODFLOW-2000, a check of the flow terms around the Spokane River was conducted and used later in the analysis.

The Little Spokane River, shown in Figure 43 as Zone 5, is not included in the SFR package calculations because the Little Spokane was originally modeled in the River Package in MODFLOW-2000. To overcome the lack of data for the Little Spokane, the Zone Budget package was run in the MODFLOW-2000 version. Running the Zone Budget package for both models had the added benefit that the SFR package results could be verified as well.

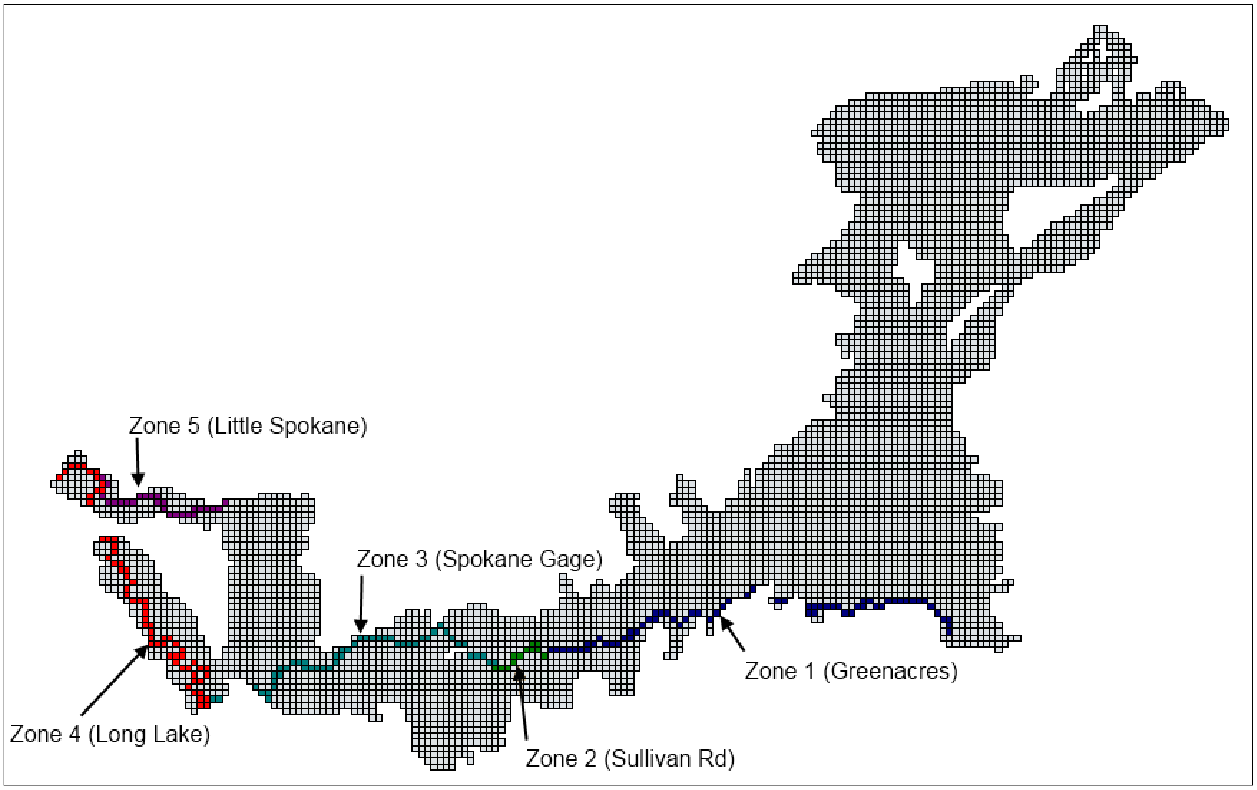

Figure 43. Zone budgets used in MODFLOW-2000 and Visual MODFLOW-2009.

To establish the validity of the Zone Budget (ZB) package, MODFLOW-2000 was run using both the ZB and the SFR packages. Zone budgets track the amount of water flowing into and out of a specified region over time. This helped ensure the differences measured between MODFLOW-2000 and VM-2009 were the actual differences between the models and not simply the variations in post-processing data extraction methods. Figure 44 shows a graph of differences in flows between the ZB and SFR package.

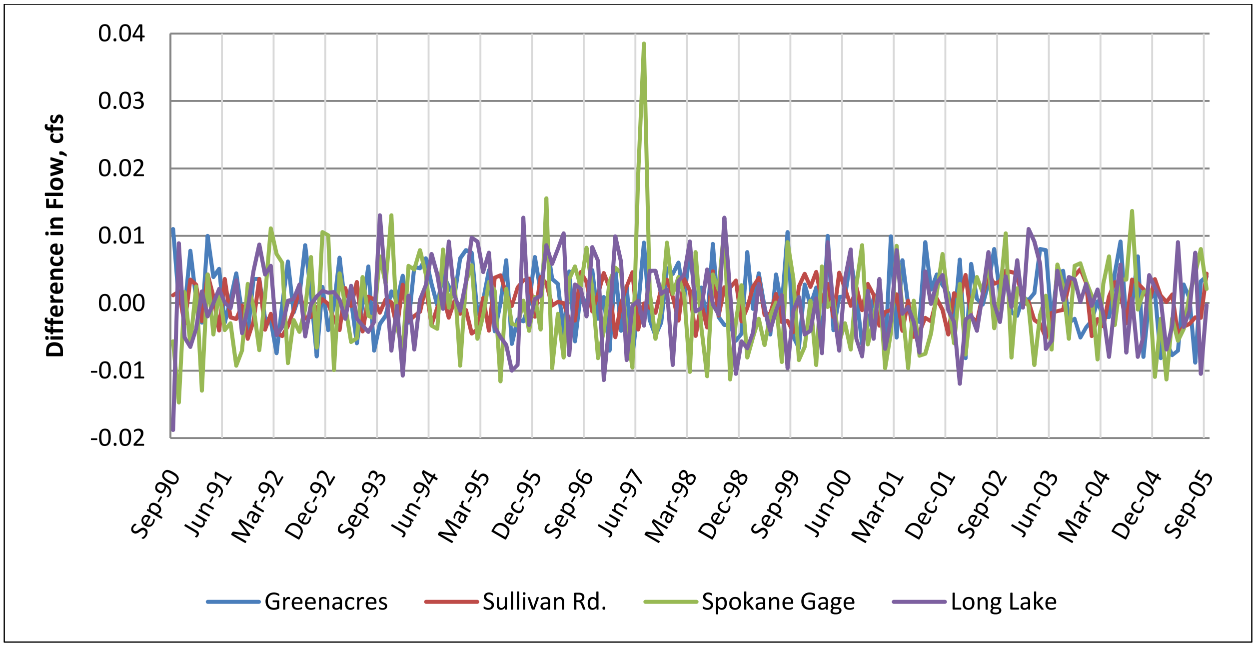

Figure 44. Difference in net flows between the SFR and ZB packages.

On average, the agreement shown in Figure 44 produced a percentage difference of about 0.01%. The only exception was the one time when the change in flow at the Spokane Gage was very small (0.04 ft^3/s) which led to a 5.41% difference. This is very good agreement and indicates the difference between the two models is in fact due to the models and not the collection of the flow terms.

Further comparisons of MODFLOW-2000 and VM-2009 show that monthly percentage differences in flows flowing across the stream/aquifer boundaries are generally small when compared to monthly river discharges at each location. So, while in some cases the differences between models appear high in terms of the flow rate magnitude (see Figure 45), the relative differences remain within the margin of error. Conversely, there are times when the percentage differences are very large but the calculated flows entering or leaving the river segments are very small (see Figure 46). The large percentage difference can therefore be explained in terms of the very small flow and can be viewed as a function of the water volumes exchanged between the aquifer and the river rather than discrepancies between the models.

To analyze the overall comparison of model results, each zone was examined independently. For example, the flow differences between the two models in the Green Acres zone budget are approximately -37 ft^3/s. Flow into the rivers is considered positive and flow leaving the river is considered negative. Combining these flows provided the net flow for each model. While the -37 ft^3/s value appears to be a relatively large value, the total average flow crossing the boundary of the river in the Green Acres zone is 568 ft^3/s so the discrepancy is approximately 6.5% of the total outflow into the aquifer.

Further analysis indicated that the Green Acres zone has the same values for differences between the models with respect to flow leaving the stream and entering the aquifer (see Figure 47) as was previously illustrated in the net flow relationship (see Figure 45). Figure 48 shows no differences between the models regarding the amount of water flowing from the aquifer to the river segments. Since the entire Green Acres reach is a losing zone, it is good that both models agree no water flows into the zone (from the aquifer to the river). Figure 46 shows average percent differences between models of approximately -6.30%.

Figure 49 shows nearly identical percentage differences in flows out of the river since no flow entered the river in either model (see Figure 50). Overall, results for this zone indicated favorable comparisons between models and acceptable differences.

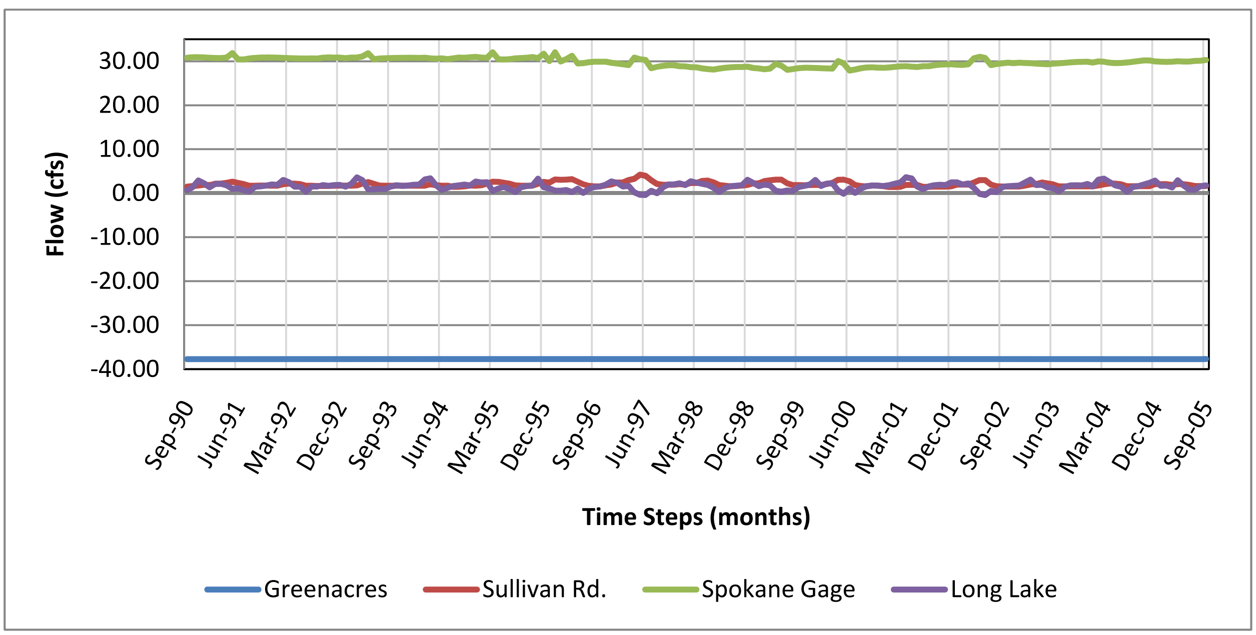

Figure 45. Net difference in flow
(Visual MODFLOW-2009 minus MODFLOW-2000).

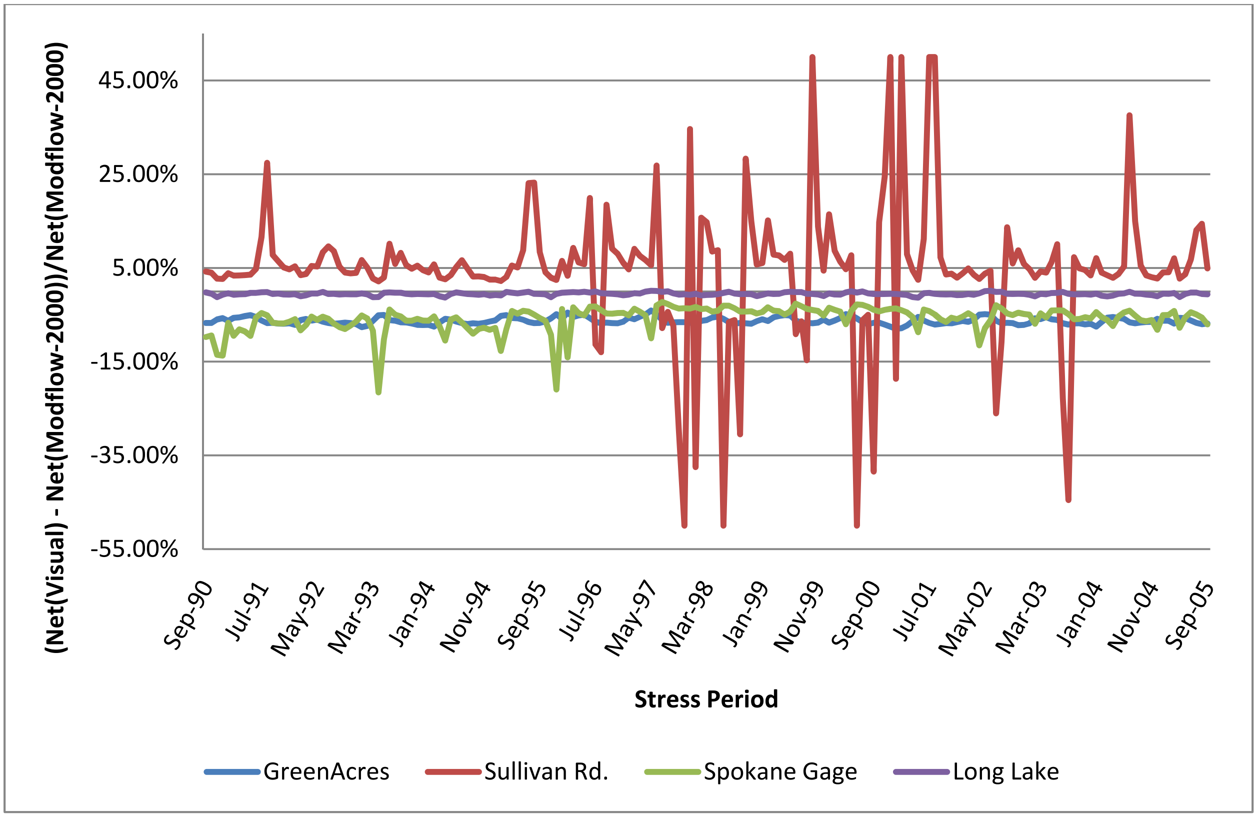

Figure 46. Net percent difference in flow.

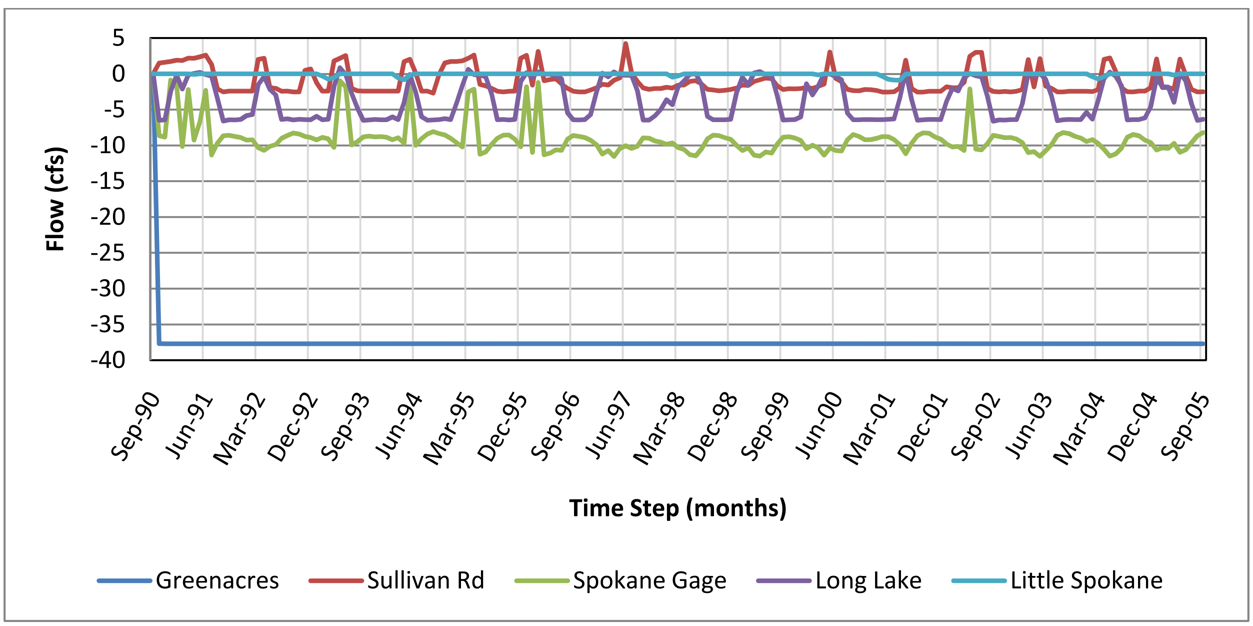

Figure 47. Flow differences OUT of the Spokane and Little Spokane Rivers
(Visual MODFLOW-2009 minus MODFLOW-2000).

The Sullivan Road zone is a transitional zone from losing to gaining depending on the time of year. The average net exchange for this reach is approximately 2.0 ft^3/s (see Figure 43). Figure 45 and Figure 46 show model differences ranging between -1.0 ft^3/s and -3.0 ft^3/s into and out of this zone, respectively, which is well within acceptable ranges. The average flow out of the reach is 40.0 ft^3/s and occurs when river stages are high. Average flow into the reach is 13.0 ft^3/s although the flows coming into the river are zero or close to zero at times (particularly during transition periods). These near zero flow rates are the source of the large percent difference (see Figure 44 and Figure 48). Overlooking the transitional nature of this zone, the flows into and out of the river as well as the net flows show the most accurate picture of model differences which are within acceptable limits.

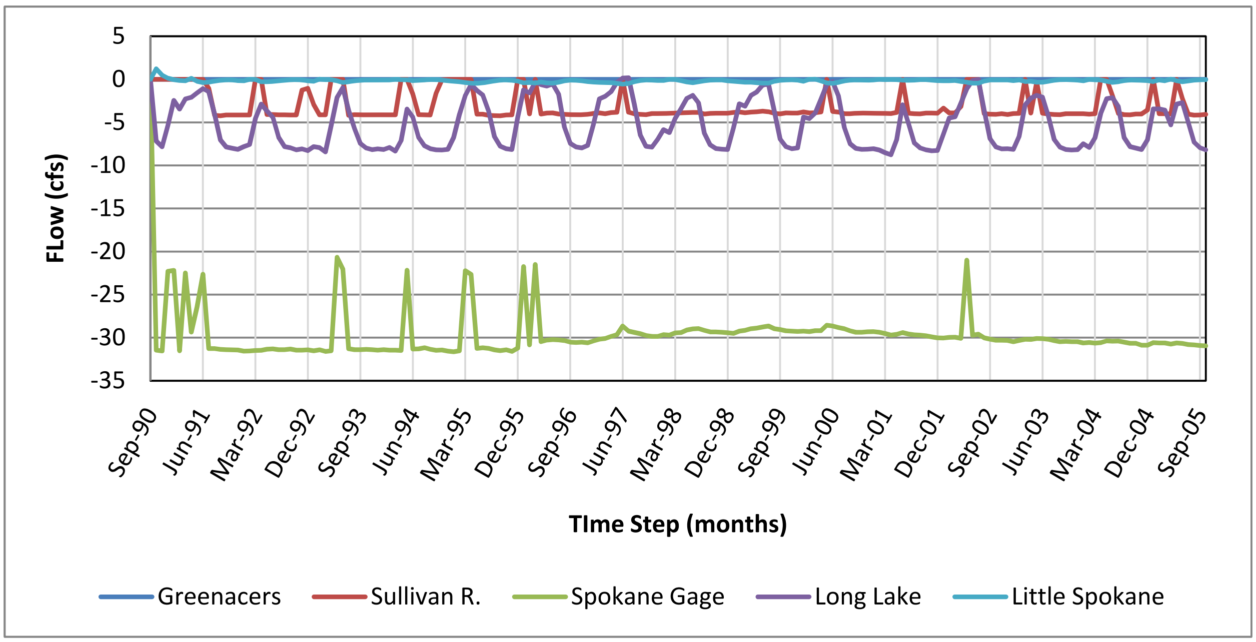

Figure 48. Flow difference INTO the Spokane and Little Spokane Rivers
(Visual MODFLOW minus MODFLOW-2000).

The Spokane Gage zone also has flows entering and leaving the river although the rates are generally higher than the Sullivan Road zone with average rates of about 736 ft^3/s and 241 ft^3/s, respectively. Figure 43 shows the difference between the two models based on net flow. The difference in net flow at the Spokane Gage for the two models is about 30 ft^3/s. Figure 45 shows the average net percent flow differences are approximately 6.4%. Figure 47 and Figure 48 both show the VM-2009 model predicts less flows both into and out of the river compared to the MODFLOW-2000 model, even though Figure 45 shows that VM-2009 has a net difference greater than MODFLOW-2000. The discrepancy is because more water is leaving the river than is entering it thus causing the net flow to be negative. This indicates that the VM-2009 is simulating less water entering and leaving the Spokane Gage zone compared to the MODFLOW-2000 model. Figure 45 shows a positive net difference because the VM-2009 results are less negative than the MODFLOW-2000 values. This does not affect model accuracy but it is an artifact of the way the flow terms were combined. Figure 47 and Figure 48 both indicate that VM-2009 simulates an average of 9 ft^3/s less flow leaving the river and approximately 30 ft^3/s less flow entering the river. Compared to the total flow entering and leaving the Spokane Gage (

Figure 49 and Figure 50) the percent difference is approximately -4% overall with a range from -2.4% to 6%. These percentage differences demonstrate that the VM-2009 model produces acceptable results for this river segment.

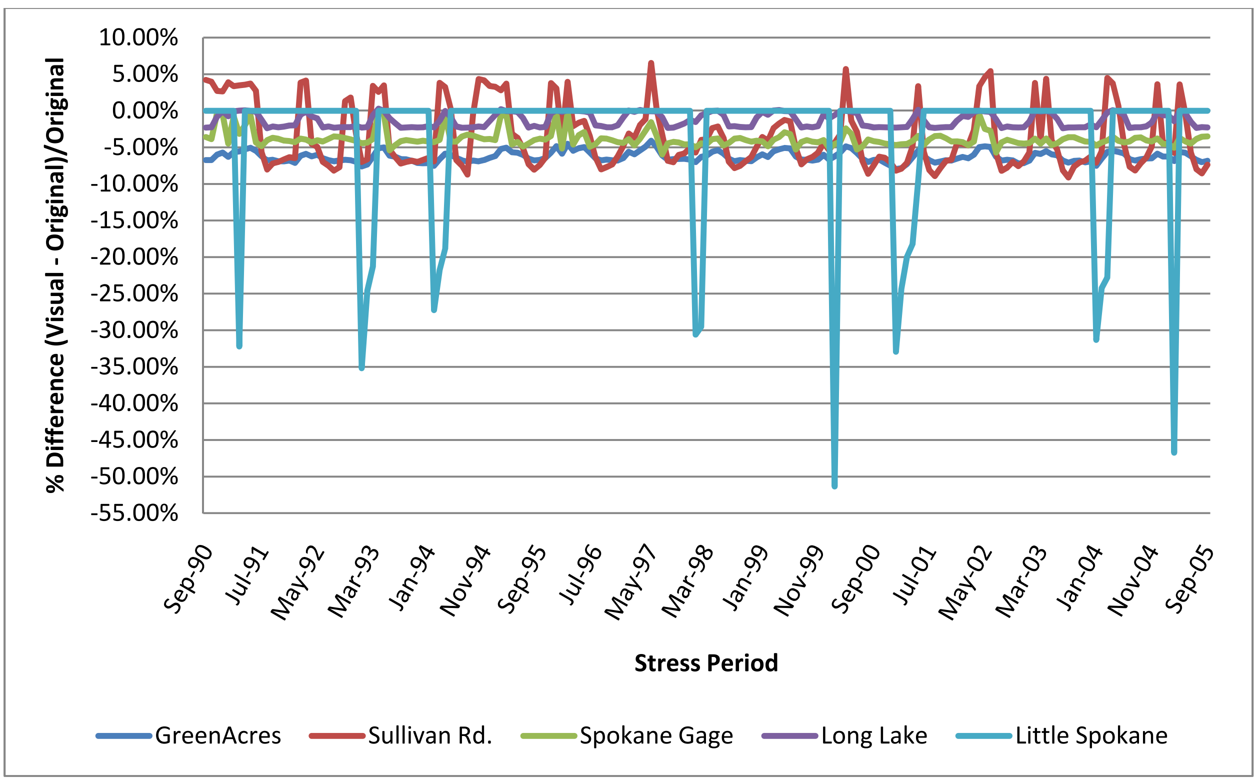

Figure 49. Percent flow differences OUT of the Spokane and Little Spokane Rivers.

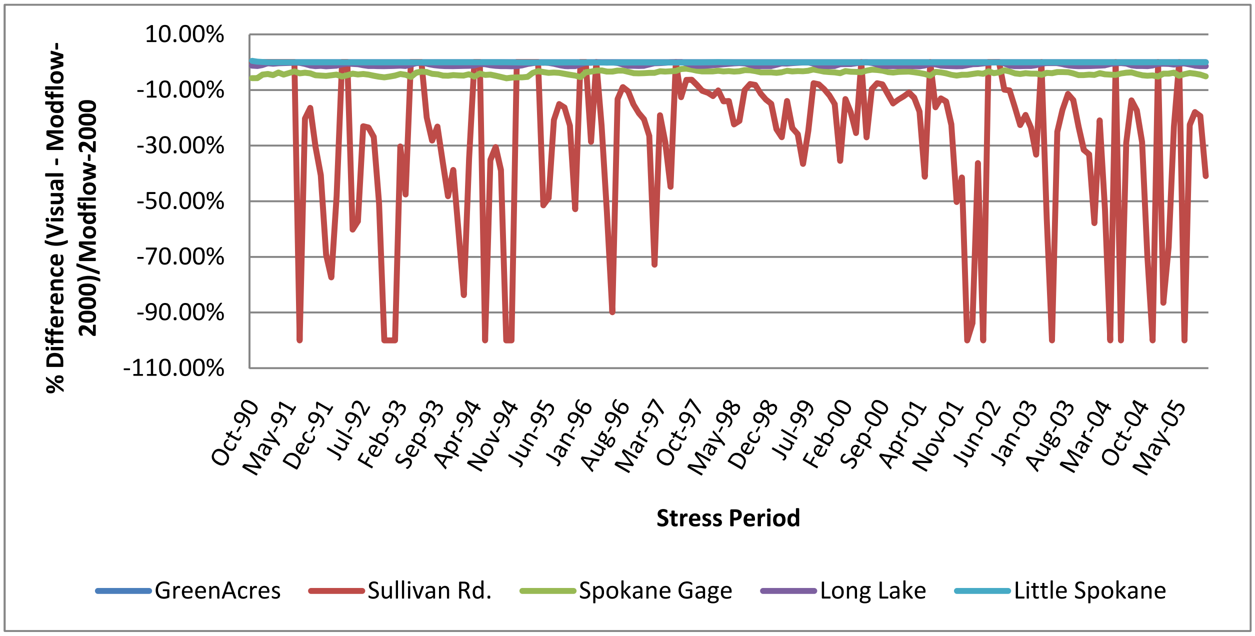

Figure 50. Percent flow differences INTO the Spokane and Little Spokane Rivers.

Water levels are very constant in the Long Lake zone as shown in all six related figures and are nearly the same for both models. Average flow differences are approximately 1.0 ft^3/s for the net flow and approximately -4.0 and -5.0 ft^3/s for flow out of and into the river, respectively. The average percent difference is -0.5% for net flow and -1.0% for flow out of and into the river. The percentage differences for this zone are accurate reflections of the differences between models and are well within acceptable limits.

Because the Little Spokane zone was modeled using the RP and not the SFR package in the original MODFLOW-2000 model, there are no Little Spokane zone comparisons. Figure 47 and Figure 48, show near zero differences in flows between the two models and

Figure 49 shows differences between the models as large as 50%. The high percentage difference in flow is an artifact of the flows leaving the river which are frequently as small as zero. With the small flows, large percentage differences occur when differences in flow range from -1.0 ft^3/s to 1.0 ft^3/s. Figure 50 shows a near zero percent difference as well. For this zone the difference in flow is an accurate portrayal of differences between the models and it is within acceptable limits.

Figure 51 shows the final zone budget map used in VM-2009 for all the runs. It is slightly different than Figure 43 to allow more information about the actual flow in the Spokane and Little Spokane Rivers to be determined instead of only the exchange of flows between the river and the aquifer. The USGS gages used to determine flow in the rivers are also shown in Figure 51. A new zone was also added near Lake Pend Oreille in order to determine the amount of additional flow into the aquifer from Lake Pend Oreille due to the extraction wells. This zone allowed calculation of how much flow was being added to the aquifer from the lake versus flow redistributed in time and space within the aquifer. Zone 1 (the large white area) encompassed all other cells in the model thus providing a check for the data in the other zones since all zone shared a border with Zone 1 and flow terms are calculated based on the zone the flow crosses.

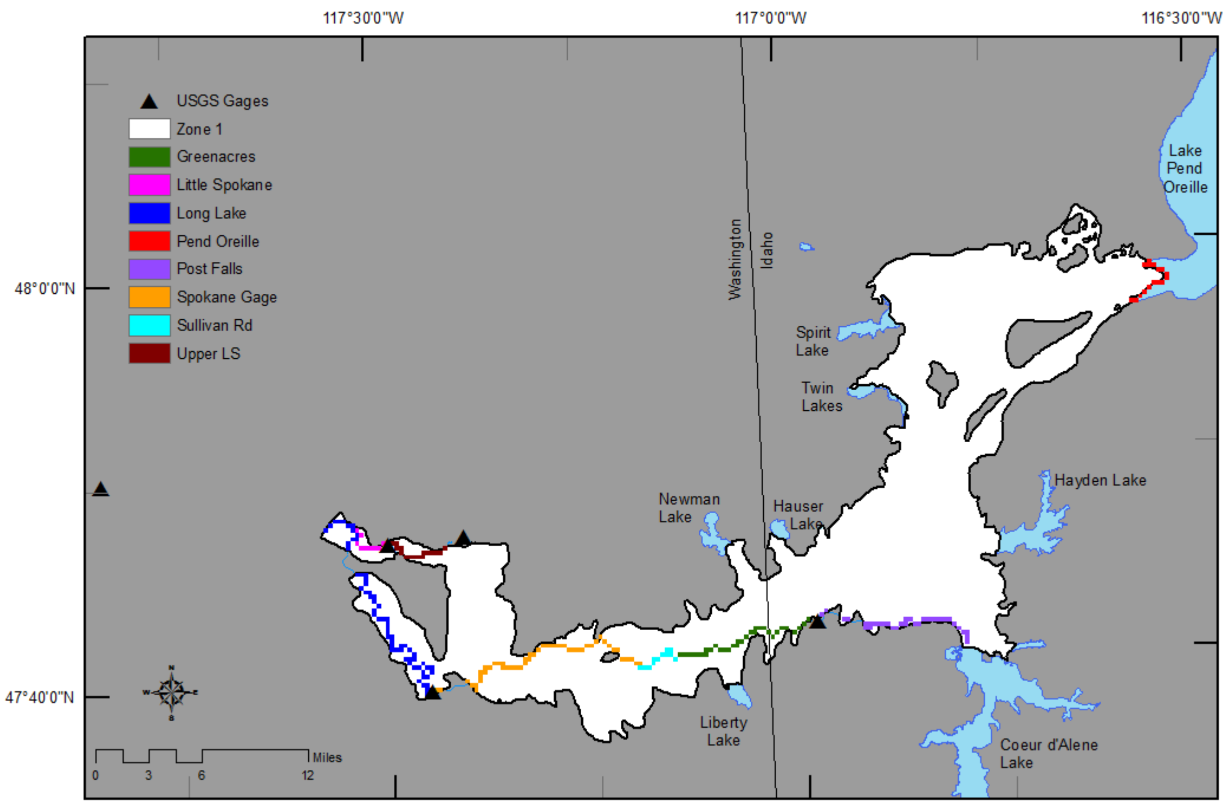

Figure 51. Final reaches for zone budget calculations.

6.2 Visual MODFLOW Modeling Results

The hydrologic analysis conducted with the VM-2009 model was used to simulate SVRP aquifer levels and streamflow interactions from 1995 through 2005, which included both wet and dry water years. The model was modified to examine the effects of injection well fields and infiltration basins placed at various locations throughout the aquifer on groundwater discharges to the Spokane and Little Spokane Rivers. The difference between the injection wells and the infiltration basins scenarios is the lag time. Direct injection implies the water will immediately become part of the saturated water table and begin moving down gradient towards the river. With infiltration basins, the water is discharged near the ground surface and must travel vertically through the unsaturated soil column above the aquifer. The feasibility of using this additional lag time to extend the pumping season and thereby maximize the utility of the distribution pipeline was examined. The lag times between injection and infiltration of water and river response for various recharge scenarios were determined to quantify the water delivered to the Spokane River. These flows will ultimately be routed downstream to critical locations along the Columbia River. The following sections provide details of this effort.

6.2.1 Direct Injection

To understand the response of the SVRP aquifer to an ASR project, a total of 275 model scenarios were completed with 15 different injection sites and 2 extraction sites. Each potential injection and extraction site was tested using a discharge of 25 ft^3/s in order to determine which month pumping needed to commence to achieve maximum return flows to the river in August. The month the pumping period could start was restricted to December through May based on the assumed availability of water. The actual months where water would be available for extraction were determined based on the extraction site chosen and the amount of pumping required compared to the existing streamflow data. Pumping periods were assumed to last from 1 to 4 months. For each injection and extraction site, flow rates of 25, 50, 75, 100, 150, 200, and 300 ft^3/s were tested. Because the model reports output in monthly increments, subtle variations between adjacent grid cells were often masked so regions could be lumped together based on distance to the river. Site locations could then focus along right-of-way corridors. Figure 52 shows the locations of the extraction and injection sites in the aquifer and the alternative distribution pipeline routes used to connect the extraction flows to the injection wells. Both extraction sites consist of wells drilled in the SVRP aquifer. It was assumed that multiple wells would be needed to extract large quantities of flow but since MODFLOW treats all wells within a grid as a single demand, the difference on simulation results is negligible. Once the water is extracted, booster pumps will distribute the water to the injection sites.

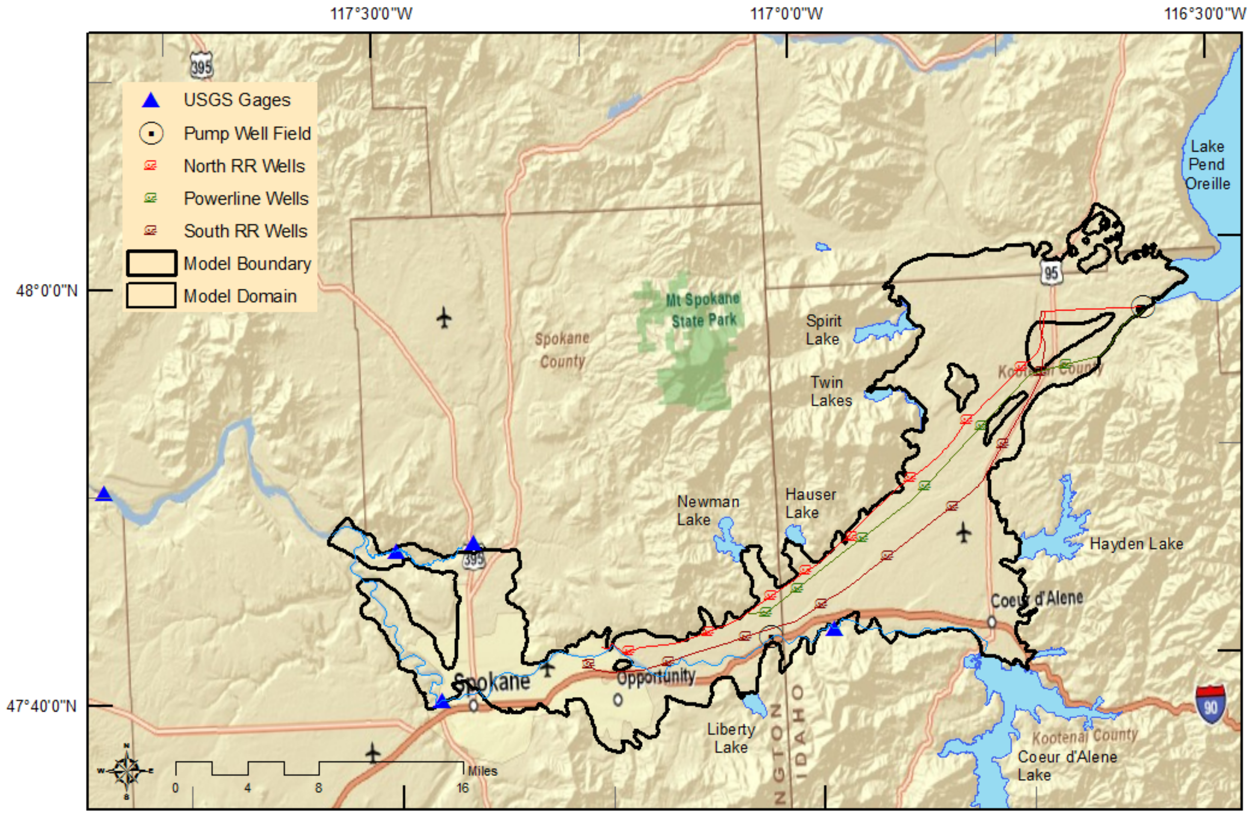

Figure 52. Well location map.

Injection well locations are named according to which right-of-way pipeline they are on and numbered in ascending order from east to west for ease of explanation. For example, the light red colored well shown in the Hauser Lake valley is labeled NR5 because it is on the Northern Railroad (NR) injection line and is the fifth well on that line heading west. The wells shown in Washington were ultimately not used due to the short lag time for the water to reach the river after injection. Table 7 shows the model coordinates for each well with the appropriate name.

Table 7. Well grid locations used in the Visual MODFLOW-2009 model.

Pumping Well		
Description	Column	Row
Pend Oreille Well Field (primary)	234	61
Pend Oreille Well Field (secondary)	236	59
Spokane River Well Field (primary)	152	134
Spokane River Well Field (secondary)	153	133
Injection Well Sites		
Description	Column	Row
Northern Railroad Track (NR)		
NR1	207	75
NR2	194	88
NR3	183	99
NR4	170	112
NR5	160	120
NR6	152	126
NR7	138	133
NR8	121	138
Power Lines (PL)		
PL1	217	74
PL2	198	88
PL3	186	101
PL4	172	113
PL5	158	124
PL6	151	129
Southern Railroad Track (SR)		
SR1	211	76
SR2	203	92
SR3	192	106
SR4	177	117
SR5	163	128
SR6	146	135
SR7	128	140
SR8	111	144

Rather than provide the details of each run, Table 8 shows a summary of the maximum flow (ft^3/s), maximum percentage of the injection rate returned in a one month time span, yearly percent of the injected flow captured, and the top three months of return flows to the Spokane River. Table 8 is organized by location, the length of the injection period, rate of injection, and month injection/pumping was started.

Table 8. Summary table of alternative designs.

Location	Starting Month	Rate (ft^3/s)	Length of Injection (Months)	Peak Monthly Return (ft^3/s)	Peak Monthly % Return	Average Yearly Return	Max Month	2nd Highest	3rd Highest
NR1	Jan	25	1	1.13	4.51%	46.81%	July	N/A	N/A
NR1	Feb	25	1	1.05	4.63%	47.29%	August	N/A	N/A
NR1	Mar	25	1	1.12	4.46%	46.71%	August	N/A	N/A
NR1	Apr	25	1	1.07	4.44%	46.55%	October	N/A	N/A
NR1	May	25	1	1.11	4.45%	46.60%	October	N/A	N/A
NR1	Dec	25	1	1.12	4.46%	46.91%	July	N/A	N/A
NR1	Mar	50	1	2.24	4.46%	46.78%	August	N/A	N/A
NR1	Mar	75	1	3.37	4.46%	46.84%	August	N/A	N/A
NR1	Mar	100	1	4.48	4.47%	46.82%	August	N/A	N/A
NR1	Mar	150	1	6.72	4.48%	46.81%	August	N/A	N/A
NR1	Mar	200	1	8.98	4.49%	46.80%	August	N/A	N/A
NR1	Mar	300	1	13.59	4.53%	47.03%	August	N/A	N/A
NR1	Jan	25	2	2.15	4.52%	47.45%	July	N/A	N/A
NR1	Feb	25	2	2.14	4.50%	47.40%	August	N/A	N/A
NR1	Mar	25	2	2.20	4.43%	47.08%	August	N/A	N/A
NR1	Apr	25	2	2.35	4.62%	47.13%	October	N/A	N/A
NR1	May	25	2	2.17	4.42%	46.92%	December	N/A	N/A
NR1	Dec	25	2	2.24	4.49%	47.32%	July	N/A	N/A
NR1	Feb	50	2	4.28	4.50%	47.43%	August	N/A	N/A
NR1	Feb	75	2	6.43	4.51%	47.45%	August	N/A	N/A
NR1	Feb	100	2	8.59	4.51%	47.49%	August	N/A	N/A
NR1	Feb	150	2	12.92	4.53%	47.48%	August	N/A	N/A
NR1	Feb	200	2	17.25	4.53%	47.44%	August	N/A	N/A
NR1	Feb	300	2	26.41	4.63%	48.27%	August	N/A	N/A
NR1	Jan	25	3	3.26	4.49%	47.35%	August	July	October
NR1	Feb	25	3	3.52	4.86%	51.56%	August	October	July
NR1	Mar	25	3	3.37	4.45%	47.15%	October	August	July
NR1	Apr	25	3	3.26	4.44%	46.98%	October	December	August
NR1	May	25	3	3.26	4.39%	46.88%	December	N/A	N/A
NR1	Dec	25	3	3.27	4.50%	47.40%	July	August	May
NR1	Jan	50	3	6.53	4.50%	47.33%	August	July	October
NR1	Jan	75	3	9.82	4.51%	47.37%	August	August	October
NR1	Jan	100	3	13.11	4.51%	47.37%	July	August	October
NR1	Jan	150	3	19.69	4.52%	47.34%	July	August	October
NR1	Jan	200	3	26.27	4.52%	47.32%	July	August	October
NR1	Jan	300	3	40.67	4.67%	48.71%	July	August	October
NR1	Jan	25	4	4.32	4.47%	47.26%	August	July	October

Location	Starting Month	Rate (ft^3/s)	Length of Injection (Months)	Peak Monthly Return (ft^3/s)	Peak Monthly % Return	Average Yearly Return	Max Month	2nd Highest	3rd Highest
NR1	Feb	25	4	4.34	4.45%	47.25%	August	October	July
NR1	Dec	25	4	4.37	4.48%	47.33%	July	August	October
NR1	Feb	50	4	8.66	4.45%	47.26%	August	August	July
NR1	Feb	75	4	12.99	4.47%	47.25%	August	October	July
NR1	Feb	100	4	17.35	4.47%	47.24%	August	October	July
NR1	Feb	150	4	26.04	4.48%	47.21%	August	October	July
NR1	Feb	200	4	34.76	4.49%	47.18%	August	October	July
NR1	Feb	300	4	53.79	4.63%	48.60%	August	October	July
NR2	Jan	25	1	1.65	6.48%	57.85%	March	March	June
NR2	Feb	25	1	1.53	6.77%	57.72%	May	N/A	N/A
NR2	Mar	25	1	1.64	6.44%	57.07%	July	N/A	N/A
NR2	Apr	25	1	1.59	6.58%	57.06%	July	N/A	N/A
NR2	May	25	1	1.64	6.57%	57.04%	August	N/A	N/A
NR2	May	50	1	3.29	6.57%	57.05%	August	N/A	N/A
NR2	May	75	1	4.93	6.57%	57.13%	August	N/A	N/A
NR2	May	100	1	6.56	6.56%	57.09%	August	N/A	N/A
NR2	May	150	1	9.86	6.57%	57.08%	August	N/A	N/A
NR2	May	200	1	13.16	6.58%	57.07%	August	N/A	N/A
NR2	May	300	1	19.78	6.59%	57.29%	August	N/A	N/A
NR2	Feb	25	2	3.11	6.52%	58.27%	May	N/A	N/A
NR2	Mar	25	2	3.21	6.49%	58.07%	July	N/A	N/A
NR2	Apr	25	2	3.20	6.50%	58.18%	July	N/A	N/A
NR2	May	25	2	3.18	6.47%	58.16%	August	N/A	N/A
NR2	May	50	2	6.38	6.49%	58.20%	August	N/A	N/A
NR2	May	75	2	9.58	6.49%	58.23%	August	N/A	N/A
NR2	May	100	2	12.77	6.49%	58.21%	August	N/A	N/A
NR2	May	150	2	19.15	6.49%	58.19%	August	N/A	N/A
NR2	May	200	2	25.54	6.49%	58.14%	August	N/A	N/A
NR2	May	300	2	38.99	6.60%	59.50%	August	September	October
NR2	Mar	25	3	4.81	6.48%	58.12%	July	August	June
NR2	Apr	25	3	4.73	6.45%	58.15%	August	July	September
NR2	May	25	3	4.77	6.28%	58.11%	October	August	September
NR2	Apr	50	3	9.48	6.46%	58.21%	August	July	September
NR2	Apr	75	3	14.22	6.46%	58.20%	August	July	September
NR2	Apr	100	3	18.96	6.46%	58.18%	August	July	September
NR2	Apr	150	3	28.43	6.46%	58.15%	August	July	September
NR2	Apr	200	3	37.89	6.45%	58.09%	August	July	September
NR2	Apr	300	3	57.85	6.57%	59.43%	August	July	September
NR2	Mar	25	4	6.24	6.34%	58.13%	August	July	September

Location	Starting Month	Rate (ft^3/s)	Length of Injection (Months)	Peak Monthly Return (ft^3/s)	Peak Monthly % Return	Average Yearly Return	Max Month	2nd Highest	3rd Highest
NR2	Apr	25	4	6.21	6.26%	58.15%	August	September	October
NR2	May	25	4	6.26	6.31%	58.08%	October	September	November
NR2	Apr	50	4	12.42	6.28%	58.19%	August	September	October
NR2	Apr	75	4	18.62	6.27%	58.15%	August	September	October
NR2	Apr	100	4	24.83	6.27%	58.13%	August	September	October
NR2	Apr	150	4	37.23	6.27%	58.08%	August	September	October
NR2	Apr	200	4	49.61	6.26%	58.01%	August	September	October
NR2	Apr	300	4	75.83	6.38%	59.36%	August	September	October
NR3	Feb	25	1	1.69	7.46%	61.25%	May	N/A	N/A
NR3	Mar	25	1	1.83	7.34%	60.56%	May	N/A	N/A
NR3	Apr	25	1	1.77	7.28%	60.59%	July	N/A	N/A
NR3	May	25	1	1.84	7.37%	60.64%	July	N/A	N/A
NR3	May	50	1	3.66	7.32%	60.63%	July	N/A	N/A
NR3	May	75	1	5.50	7.33%	60.71%	July	N/A	N/A
NR3	May	100	1	7.34	7.34%	60.72%	July	N/A	N/A
NR3	May	150	1	11.01	7.34%	60.70%	July	N/A	N/A
NR3	May	200	1	14.69	7.34%	60.70%	July	N/A	N/A
NR3	May	300	1	22.09	7.36%	60.93%	July	N/A	N/A
NR3	Mar	25	2	3.60	7.10%	61.67%	July	N/A	N/A
NR3	Apr	25	2	3.60	7.32%	61.81%	July	N/A	N/A
NR3	May	25	2	3.58	7.27%	61.88%	August	N/A	N/A
NR3	May	50	2	7.18	7.30%	61.91%	August	N/A	N/A
NR3	May	75	2	10.78	7.31%	61.94%	August	N/A	N/A
NR3	May	100	2	14.37	7.30%	61.93%	August	N/A	N/A
NR3	May	150	2	21.57	7.31%	61.90%	August	N/A	N/A
NR3	May	200	2	28.76	7.31%	61.88%	August	N/A	N/A
NR3	May	300	2	43.83	7.42%	63.25%	August	July	September
NR3	Mar	25	3	5.32	7.25%	62.40%	July	June	August
NR3	Apr	25	3	5.25	7.15%	61.83%	August	July	September
NR3	May	25	3	5.28	7.08%	61.82%	August	September	October
NR3	May	50	3	10.56	7.08%	61.86%	August	September	October
NR3	May	75	3	15.85	7.10%	61.90%	August	September	October
NR3	May	100	3	21.13	7.09%	61.87%	August	September	October
NR3	May	150	3	31.69	7.10%	61.83%	August	September	October
NR3	May	200	3	42.26	7.09%	61.80%	August	September	October
NR3	May	300	3	64.47	7.21%	63.15%	August	September	October
NR3	Mar	25	4	6.94	7.06%	61.76%	July	August	June
NR3	Apr	25	4	6.92	7.03%	61.82%	August	September	July
NR3	May	25	4	6.94	6.90%	61.78%	October	September	August

Location	Starting Month	Rate (ft^3/s)	Length of Injection (Months)	Peak Monthly Return (ft^3/s)	Peak Monthly % Return	Average Yearly Return	Max Month	2nd Highest	3rd Highest
NR3	Apr	50	4	13.85	7.04%	61.88%	August	September	July
NR3	Apr	75	4	20.78	7.04%	61.84%	August	September	July
NR3	Apr	100	4	27.71	7.04%	61.82%	August	September	July
NR3	Apr	150	4	41.55	7.04%	61.78%	August	September	July
NR3	Apr	200	4	55.38	7.04%	61.73%	August	September	July
NR3	Apr	300	4	84.47	7.15%	63.10%	August	September	July
NR3[A]	Jan	25	1	0.54	2.16%	-8.80%	August	July	September
NR3[A]	Feb	25	1	0.49	2.15%	-8.13%	October	August	September
NR3[A]	Mar	25	1	0.54	2.15%	-8.42%	October	November	December
NR3[A]	May	25	1	0.52	2.10%	-9.59%	December	January	November
NR3[A]	Jan	50	1	1.08	2.16%	-8.75%	August	July	September
NR3[A]	Jan	75	1	1.63	2.17%	-8.63%	August	July	September
NR3[A]	Jan	100	1	2.17	2.17%	-8.53%	August	July	September
NR3[A]	Jan	150	1	3.26	2.18%	-8.41%	August	July	September
NR3[A]	Jan	200	1	4.34	2.17%	-8.38%	August	July	September
NR3[A]	Jan	300	1	6.28	2.09%	-8.19%	August	July	September
NR3[A]	Jan	25	4	2.03	2.10%	-8.51%	October	November	November
NR3[A]	Dec	25	4	2.06	2.08%	-8.48%	October	August	September
NR3[A]	Dec	50	4	4.11	2.07%	-8.47%	October	August	September
NR3[A]	Dec	75	4	6.17	2.08%	-8.42%	October	August	September
NR3[A]	Dec	100	4	8.21	2.07%	-8.41%	October	August	September
NR3[A]	Dec	150	4	12.31	2.07%	-8.37%	October	August	September
NR3[A]	Dec	200	4	16.40	2.07%	-8.33%	October	August	September
NR3[A]	Dec	300	4	23.67	2.00%	-8.11%	October	August	September
NR4	Apr	25	1	2.11	8.74%	64.19%	May	N/A	N/A
NR4	Mar	25	1	2.19	8.74%	64.15%	May	N/A	N/A
NR4	May	25	1	2.18	8.72%	64.46%	July	N/A	N/A
NR4	May	50	1	4.40	8.73%	64.48%	July	N/A	N/A
NR4	May	75	1	6.58	8.73%	64.54%	July	N/A	N/A
NR4	May	100	1	8.77	8.74%	64.54%	July	N/A	N/A
NR4	May	150	1	13.10	8.74%	64.54%	July	N/A	N/A
NR4	May	200	1	17.48	8.74%	64.53%	July	N/A	N/A
NR4	May	300	1	26.29	8.76%	64.82%	July	N/A	N/A
NR4	Apr	25	2	4.30	8.46%	65.55%	June	N/A	N/A
NR4	Mar	25	2	4.29	8.72%	65.24%	May	N/A	N/A
NR4	May	25	2	4.29	8.73%	65.77%	July	N/A	N/A
NR4	May	50	2	8.60	8.74%	65.78%	July	N/A	N/A
NR4	May	75	2	12.88	8.73%	65.82%	July	N/A	N/A
NR4	May	100	2	17.18	8.73%	65.82%	July	N/A	N/A

Location	Starting Month	Rate (ft^3/s)	Length of Injection (Months)	Peak Monthly Return (ft^3/s)	Peak Monthly % Return	Average Yearly Return	Max Month	2nd Highest	3rd Highest
NR4	May	150	2	25.77	8.73%	65.82%	July	N/A	N/A
NR4	May	200	2	34.35	8.73%	65.82%	July	N/A	N/A
NR4	May	300	2	52.27	8.85%	67.21%	July	August	June
NR4	Apr	25	3	6.23	8.49%	65.65%	July	August	June
NR4	May	25	3	6.28	8.46%	65.76%	August	July	September
NR4	May	50	3	12.56	8.47%	65.80%	August	July	September
NR4	May	75	3	18.86	8.47%	65.84%	August	July	September
NR4	May	100	3	25.15	8.47%	65.83%	August	July	September
NR4	May	150	3	37.72	8.47%	65.81%	August	July	September
NR4	May	200	3	50.31	8.48%	65.80%	August	July	September
NR4	May	300	3	76.58	8.60%	67.16%	August	July	September
NR4	Apr	25	4	7.98	8.11%	65.68%	August	July	September
NR4	May	25	4	8.02	8.07%	65.72%	August	September	October
NR4	May	50	4	16.06	8.08%	65.79%	August	September	October
NR4	May	75	4	24.07	8.08%	65.79%	August	September	October
NR4	May	100	4	32.10	8.09%	65.77%	August	September	October
NR4	May	150	4	48.17	8.09%	65.75%	August	September	October
NR4	May	200	4	64.28	8.09%	65.72%	August	September	October
NR4	May	300	4	98.13	8.23%	67.22%	August	September	October
NR5	Apr	25	1	2.75	11.38%	66.77%	May	N/A	N/A
NR5	Mar	25	1	2.83	10.97%	67.02%	April	N/A	N/A
NR5	May	25	1	2.84	11.00%	67.32%	June	N/A	N/A
NR5	May	50	1	5.74	11.12%	67.33%	June	N/A	N/A
NR5	May	75	1	8.56	11.05%	67.42%	June	N/A	N/A
NR5	May	100	1	11.38	11.02%	68.51%	June	May	July
NR5	May	150	1	17.05	11.00%	67.38%	June	N/A	N/A
NR5	May	200	1	22.71	10.99%	67.43%	June	N/A	N/A
NR5	May	300	1	34.43	11.11%	70.02%	June	May	July
NR5	Apr	25	2	5.28	10.73%	68.20%	May	N/A	N/A
NR5	Mar	25	2	5.28	10.62%	67.69%	May	N/A	N/A
NR5	May	25	2	5.30	10.70%	68.70%	July	N/A	N/A
NR5	May	50	2	10.64	10.64%	68.66%	July	N/A	N/A
NR5	May	75	2	15.92	10.64%	68.72%	July	N/A	N/A
NR5	May	100	2	21.21	10.65%	68.71%	July	N/A	N/A
NR5	May	150	2	31.81	10.64%	68.73%	July	N/A	N/A
NR5	May	200	2	42.44	10.64%	68.74%	July	N/A	N/A
NR5	May	300	2	64.34	10.77%	70.18%	July	June	August
NR5	Apr	25	3	7.72	10.18%	68.40%	June	July	August
NR5	May	25	3	7.80	10.51%	68.72%	July	August	September

Location	Starting Month	Rate (ft^3/s)	Length of Injection (Months)	Peak Monthly Return (ft^3/s)	Peak Monthly % Return	Average Yearly Return	Max Month	2nd Highest	3rd Highest
NR5	May	50	3	15.61	10.52%	68.77%	July	August	September
NR5	May	75	3	23.41	10.52%	68.81%	July	August	September
NR5	May	100	3	31.21	10.52%	68.81%	July	August	September
NR5	May	150	3	46.80	10.51%	68.81%	July	August	September
NR5	May	200	3	62.40	10.51%	68.80%	July	August	September
NR5	May	300	3	94.72	10.64%	70.22%	July	August	September
NR5	Apr	25	4	9.77	9.93%	68.52%	July	August	June
NR5	May	25	4	9.82	9.90%	68.69%	August	September	July
NR5	May	50	4	19.64	9.90%	68.76%	August	September	July
NR5	May	75	4	29.45	9.90%	68.80%	August	September	July
NR5	May	100	4	39.28	9.90%	68.79%	August	September	July
NR5	May	150	4	58.94	9.90%	68.79%	August	September	July
NR5	May	200	4	78.64	9.91%	68.77%	August	September	July
NR5	Apr	300	4	118.84	10.07%	70.07%	July	August	June
NR5	May	300	4	119.65	10.05%	70.31%	August	September	July
PL1	Mar	100	1	2.72	2.72%	17.36%	December	January	July
PL1	Feb	100	2	5.27	2.73%	18.42%	December	November	October
PL1	Jan	100	3	8.09	2.79%	19.36%	December	November	October
PL1	Feb	100	4	10.76	2.78%	19.69%	December	November	October
PL2	May	100	1	6.43	6.43%	57.61%	August	July	September
PL2	May	100	2	12.50	6.35%	57.55%	August	September	October
PL2	Apr	100	3	18.58	6.33%	57.54%	August	July	September
PL2	Apr	100	4	24.39	6.14%	57.49%	August	September	October
PL3	May	25	1	1.84	7.37%	62.06%	July	August	September
PL3	May	50	1	3.69	7.37%	62.03%	July	August	September
PL3	May	75	1	5.54	7.39%	62.12%	July	August	September
PL3	May	100	1	7.39	7.39%	62.07%	July	August	September
PL3	May	150	1	11.08	7.39%	62.08%	July	August	September
PL3	May	200	1	14.79	7.39%	62.10%	July	August	September
PL3	May	300	1	22.54	7.51%	63.48%	July	August	September
PL3	May	25	2	3.60	7.33%	62.02%	August	July	September
PL3	May	50	2	7.21	7.33%	62.06%	August	July	September
PL3	May	75	2	10.84	7.34%	62.10%	August	July	September
PL3	May	100	2	14.45	7.34%	62.08%	August	July	September
PL3	May	150	2	21.68	7.35%	62.07%	August	July	September
PL3	May	200	2	28.92	7.35%	62.04%	August	July	September
PL3	May	300	2	45.29	7.67%	65.83%	August	July	September
PL3	May	100	3	21.22	7.14%	62.04%	August	September	October
PL3	Apr	100	4	27.87	7.08%	61.98%	August	September	July

Location	Starting Month	Rate (ft^3/s)	Length of Injection (Months)	Peak Monthly Return (ft^3/s)	Peak Monthly % Return	Average Yearly Return	Max Month	2nd Highest	3rd Highest
PL4	May	100	1	8.70	8.70%	65.67%	July	June	August
PL4	May	100	2	17.07	8.67%	65.64%	July	August	June
PL4	May	100	3	25.05	8.44%	65.75%	August	July	September
PL4	May	100	4	31.98	8.05%	65.70%	August	September	October
PL5	May	100	1	12.31	11.92%	69.32%	June	July	July
PL5	May	100	2	23.51	11.56%	69.44%	June	July	August
PL5	May	100	3	33.85	11.41%	69.68%	July	August	June
PL5	May	100	4	41.87	10.55%	69.69%	August	July	September
SR1	Mar	100	1	3.83	3.83%	35.30%	December	March	July
SR1	Feb	100	2	7.37	3.87%	35.86%	December	March	October
SR1	Jan	100	3	11.17	3.82%	36.20%	December	October	March
SR1	Feb	100	4	14.83	3.83%	36.16%	December	May	July
SR2	May	100	1	5.37	5.37%	52.84%	October	December	August
SR2	May	100	2	10.66	5.42%	53.44%	October	December	November
SR2	Apr	100	3	16.01	5.46%	54.10%	October	December	August
SR2	Apr	100	4	21.45	5.45%	54.46%	October	December	November
SR3	May	100	1	7.49	7.49%	62.53%	July	August	September
SR3	May	25	2	3.64	7.41%	62.47%	August	July	September
SR3	May	50	2	7.30	7.42%	62.51%	August	July	September
SR3	May	75	2	10.97	7.43%	62.53%	August	July	September
SR3	May	100	2	14.64	7.44%	62.54%	August	July	September
SR3	May	150	2	21.95	7.44%	62.51%	August	July	September
SR3	May	200	2	29.27	7.44%	62.48%	August	July	September
SR3	May	300	2	44.60	7.56%	63.85%	August	July	September
SR3	May	25	3	5.37	7.20%	62.41%	August	September	October
SR3	May	50	3	10.73	7.21%	62.49%	August	September	October
SR3	May	75	3	16.10	7.23%	62.50%	August	September	October
SR3	May	100	3	21.46	7.22%	62.48%	August	September	October
SR3	May	150	3	32.20	7.23%	62.43%	August	September	October
SR3	May	200	3	42.94	7.22%	62.40%	August	September	October
SR3	May	300	3	65.51	7.35%	63.76%	August	September	October
SR3	Apr	100	4	28.18	7.16%	62.41%	August	September	July
SR4	May	100	1	8.58	8.58%	65.45%	July	June	August
SR4	May	100	2	16.74	8.51%	65.50%	July	August	June
SR4	May	100	3	24.65	8.31%	65.49%	August	July	September
SR4	May	100	4	31.61	7.88%	65.43%	August	September	October
SR5	May	100	1	11.74	11.36%	68.82%	June	July	July
SR5	May	100	2	16.74	8.51%	65.50%	July	August	June
SR5	May	100	3	32.14	10.83%	69.13%	July	August	September

Location	Starting Month	Rate (ft^3/s)	Length of Injection (Months)	Peak Monthly Return (ft^3/s)	Peak Monthly % Return	Average Yearly Return	Max Month	2nd Highest	3rd Highest
SR5	May	100	4	40.26	10.15%	69.12%	August	September	July
NR1-NR5[B]	Mar	125	1	9.08	7.24%	59.94%	May	June	June
NR1-NR5[B]	May	125	3	26.48	7.14%	60.32%	August	July	September

[A] Denotes the test done with the well near the Spokane River vs. near Lake Pond Oreille as all the other tests.

[B] Denotes the multiple wells test, with each well injecting 25 ft^3/s for a total of 125 ft^3/s pumped and injected.

To aid in understanding trends in the data, a series of figures are shown below which highlight certain aspects of the data. Figure 53 represents the way the return flow was captured per zone in the Spokane and Little Spokane Rivers (see Figure 51 for zone identification). Figure 53 assumes a 2 month injection period at the NR3 site starting in May at a flow rate of 100 ft^3/s. The NR3-100 scenario illustrated in Figure 53 is not meant to imply that it is the preferred alternative but simply that the results are typical since NR3 is at the half-way point in the distribution pipeline.

Figure 53 shows the Sullivan Road and Spokane Gage zones as the only contributors of additional flow for the Spokane and Little Spokane Rivers which is true for all the scenarios. This fact makes it convenient to sum the total flow added to the Spokane River at the Spokane Gage (see Figure 51). Since virtually no additional flow is added to the Little Spokane River and no flow is added to the Long Lake Zone via the confined layer of the model, the total gain to the Spokane River is captured at the Spokane Gage (USGS gage 12422500). Thus the gain to USGS gage 12422500 (shown as a line on the graph) represents the total additional flow into the Spokane and Little Spokane Rivers (Little Spokane River having near zero gain) and is the flow that will be shown on subsequent graphs to identity total additional flow to the river system. The USGS gaging station allows for the increase in flow from the scenario runs to be added to the actual historical Spokane River flows to determine how the additional water will impact streamflows.

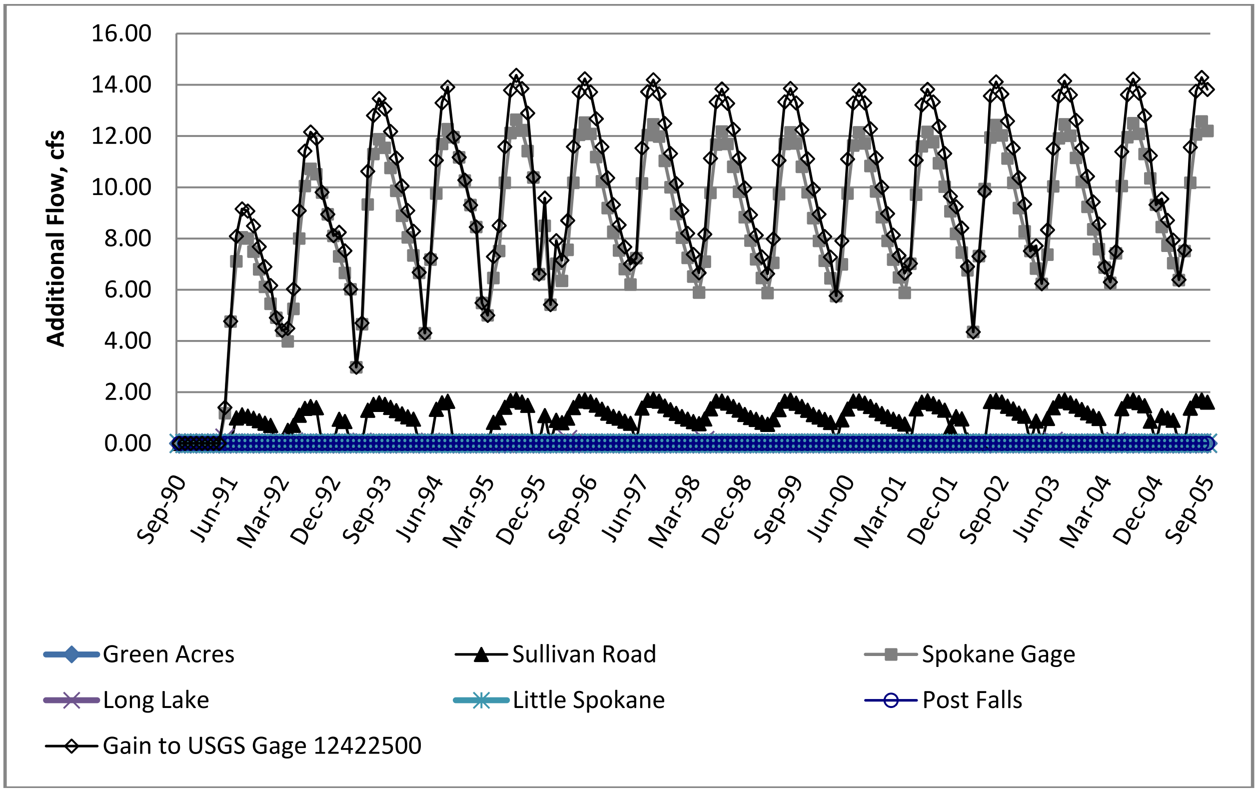

Figure 53. Additional flow by zone into Spokane and Little Spokane Rivers caused by injections at NR3 (May, 100 ft^3/s, 2 months).

The following figures show the trends in the data caused by increased pumping rates, increased length of injection periods, and position within the aquifer. Figure 54 shows that increasing the pumping rate results in a proportional increase in the flow returning to the Spokane River. It also shows that increasing the pumping rate not only increases the peak additional flow but also the total additional flow. In other words, the hydrographs get higher and wider so the total flow returning to the river represents more than just an increase in peak discharges. It is evident from Figure 55, however, that increasing the injection rate does not change the total percentage of return flow but only the actual flow volumes returned. For example, if September returns 7% of a 50 ft^3/s injection rate, it would also return 7% of a 100 ft^3/s injection rate. The increase in river return flow would simply be linearly proportional. This phenomenon is observed at all well locations.

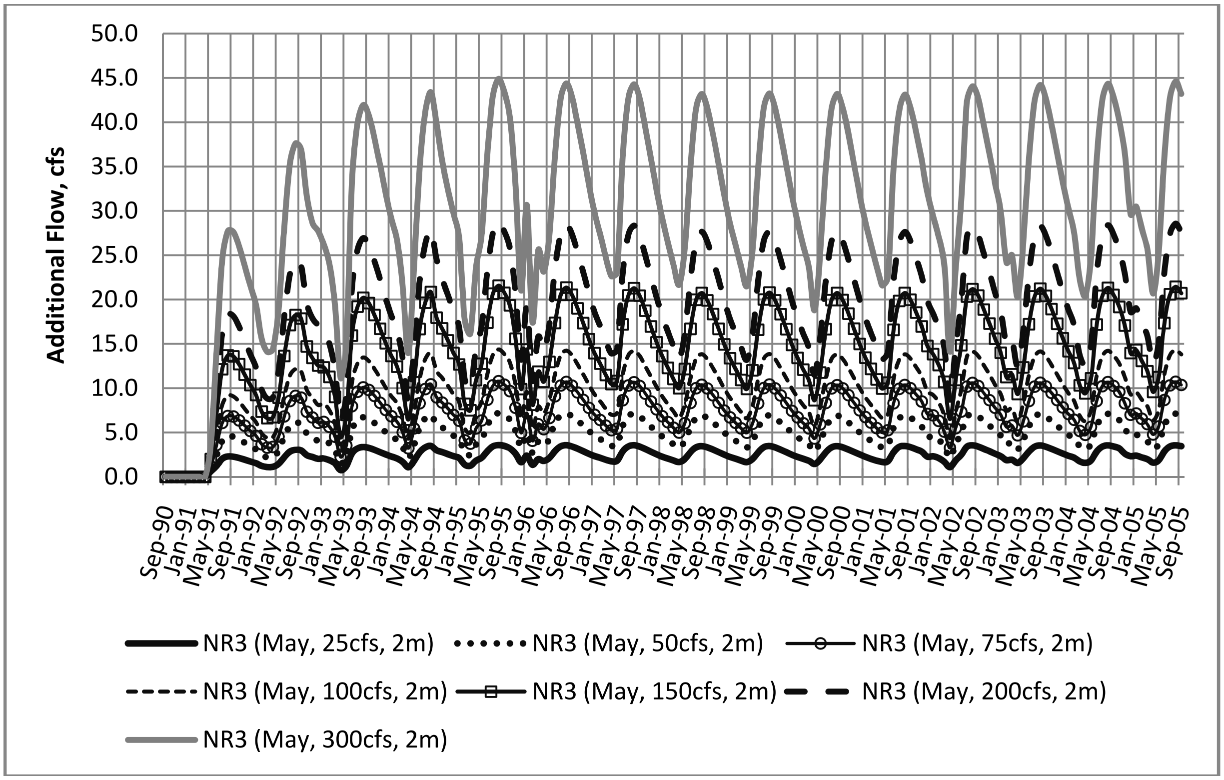

Figure 54. Comparison of rate increases caused by injections at NR3.

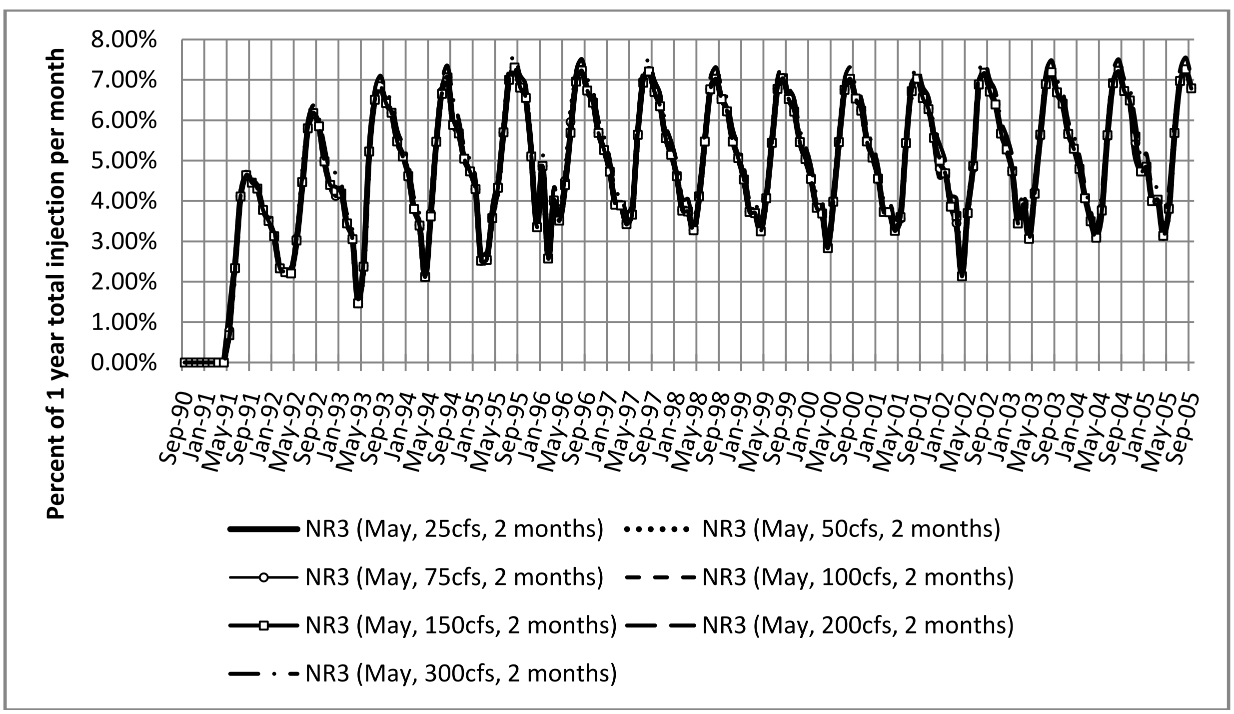

Figure 55. Comparison of percent return flows as a function of increased injection rates.

Figure 56 shows the comparison of how the length of the injection period (duration) affects the additional flow gained in the Spokane River based on a single injection well located at the NR3 site. Much like the increase in injection rate, Figure 56 shows additional flow from a longer duration period increases in equal incremental steps from one month to four months. Again, the linear nature of this response allows interpolation or extrapolation to other flow rates without requiring additional model simulations. Figure 56 also illustrates that the peak return flow from a 1 month injection period occurs a month ahead of the peak return flows for injection periods lasting two to four months. This appears to indicate that longer injection times would help to increase the times to peak, allowing for well locations closer to the Spokane River. At some point, however, steady-state is reached and this would become less effective. Furthermore, the monthly time step used in the model could mean the one month difference only represents a subtle change in the timing.

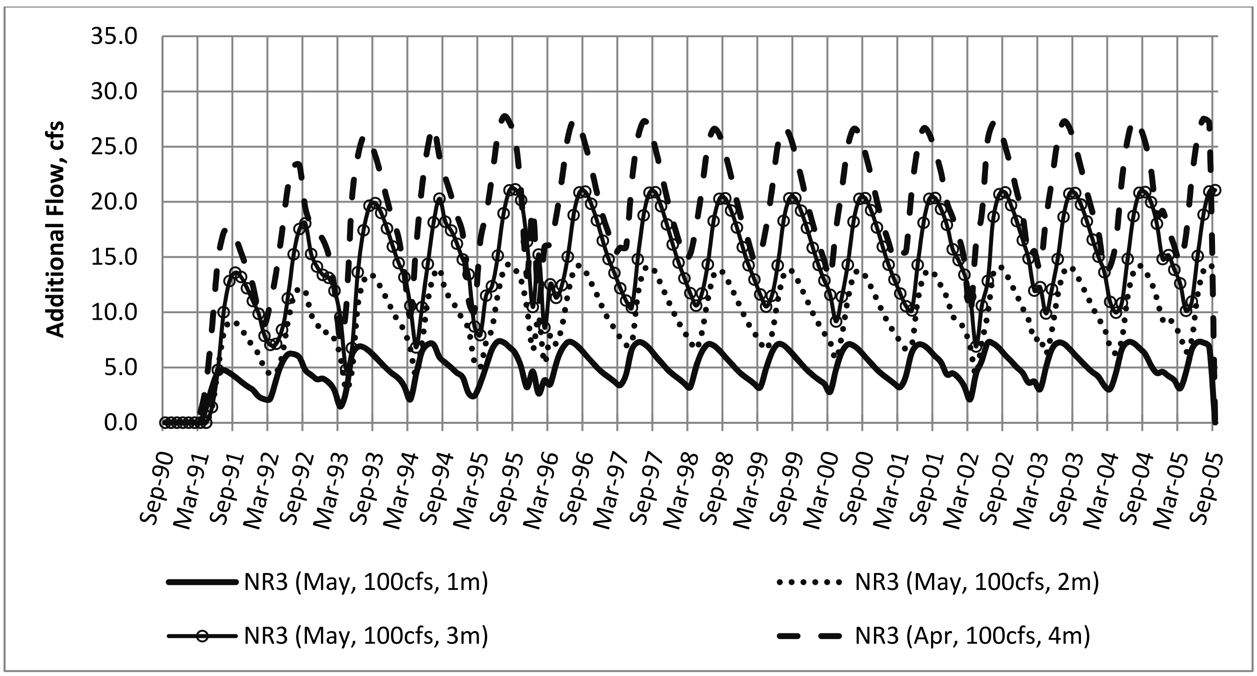

Figure 56. Comparison of injection period duration (length).

Figure 57 shows the comparison of how the length of the injection period (duration) affects the additional flow gained in the Spokane River on an annual basis. It indicates the percentage return flow for each injection duration is the same per month and does not increase for longer duration periods. The timing of the peak return flow for the 1 month injection period compared to the peak return flow lags for the two to four month injection periods is also clearly evident in the figure.

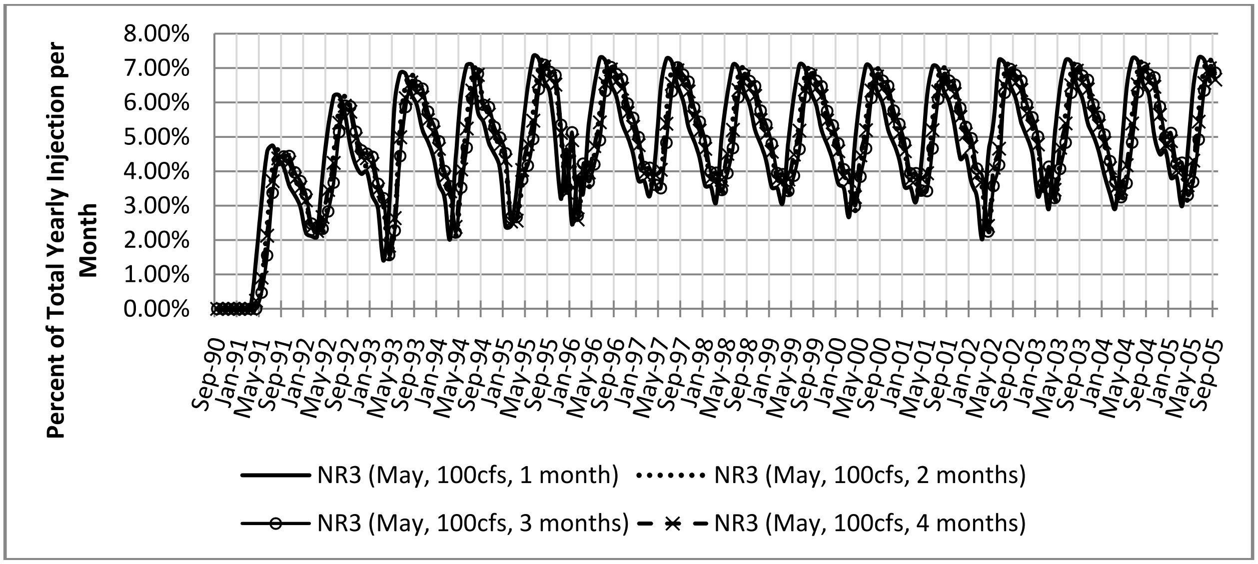

Figure 57. Comparison of monthly percentages of total annual injection.

Figure 58 and Figure 59 show that the peak return flow is directly proportional to the distance from the injection well to the Spokane River. In other words, higher additional peak flows are produced when the injection well is closer to the river. For example, for a well located at NR1 (grid cell 207, 75), the monthly peak return percentage is 4.3% of the total injected amount for that year but for a well located closer to the Spokane River at NR5 (grid cell 160, 120), the peak percentage return is 10.4%.

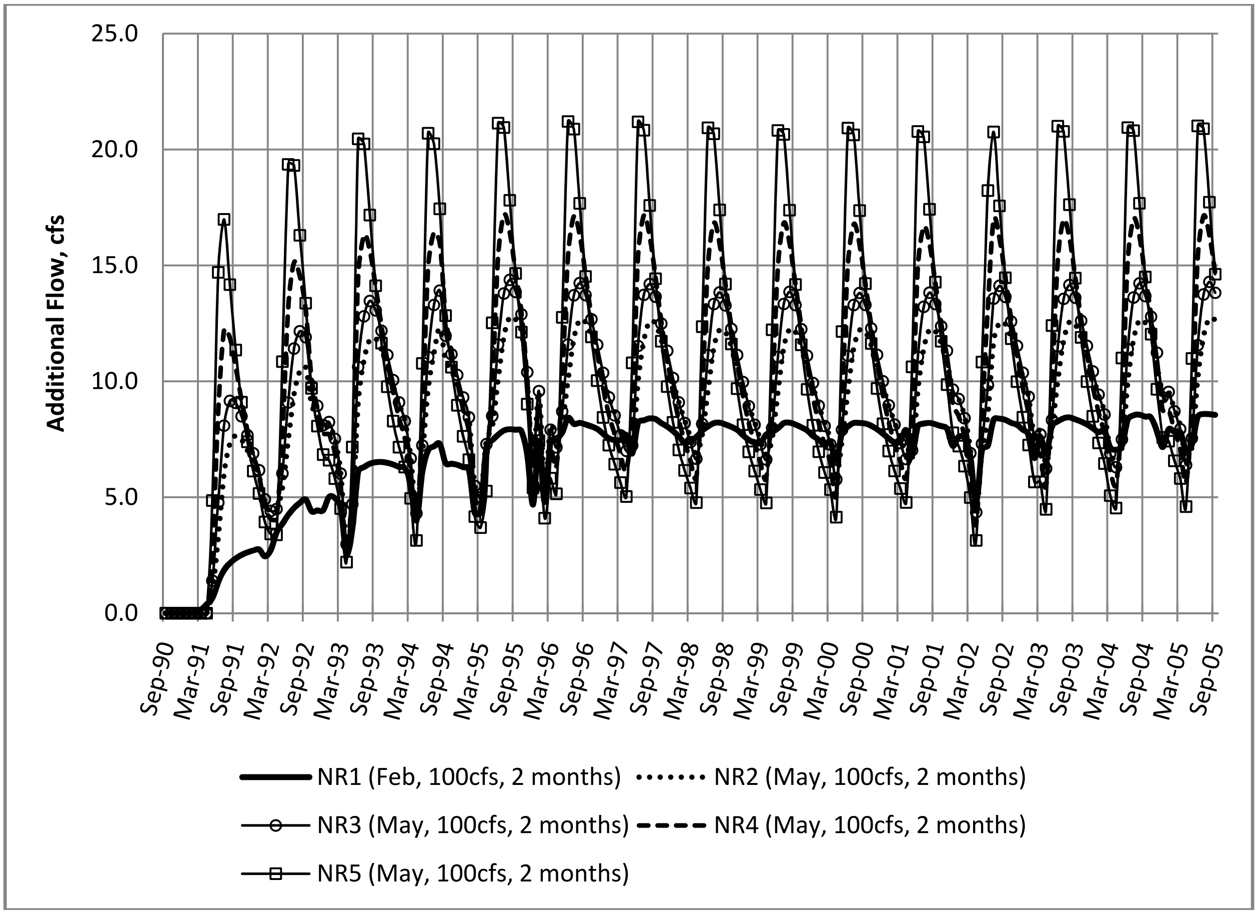

Figure 58. Comparison of well locations.

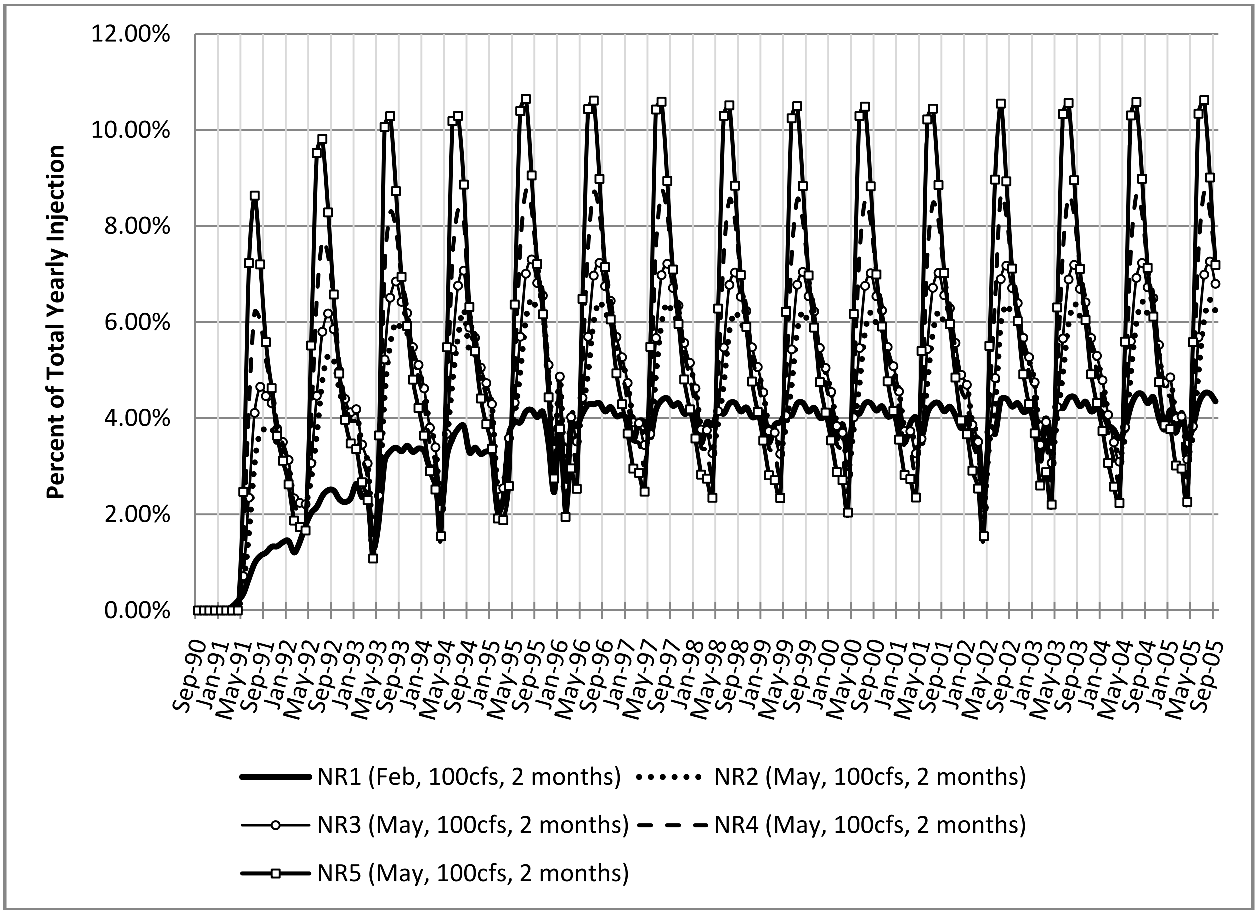

Figure 59. Comparison of percentage of total yearly injection per month.

The previous six figures provide three useful guidelines with respect to selecting well locations in the SVRP aquifer:

1) increasing the rate of injection does not yield higher percentage returns but it will result in a proportional increase in additional flow,
2) increasing the length of the injection period does not yield higher percentage returns but does yield a proportional increase in additional flow and increases lag time, and
3) moving the location of the well towards the Spokane River increases the percentage return.

These same rules apply for all well locations, rates, and injection periods when pumping withdrawals water near Lake Pend Oreille.

Washington Extraction Well Field

Figure 52 previously identified two potential extraction well field locations for water withdrawal: one near the Spokane River and one near Lake Pend Oreille. Through multiple modeling scenarios using groundwater withdrawals near the Spokane River, it was found that pumping from the aquifer at that location was not a viable option. When the extraction well near the Spokane River was pumping even for a one month period, a large cone of depression formed around the well. Because the water was being injected upstream, a considerable portion of the injected water went to refilling the depression (see Figure 60). This led to a change in the direction of interaction between the aquifer and the river so water left the river and filled the depression in the aquifer created by the scenario run in the model. The combined effect was that a very small percentage of the injected water came back as additional flow to the Spokane River. The negative impact on the river grew as the pumping period was lengthened (see Figure 61). Thus, in the Spokane River vicinity, direct surface water diversion seems to be the only technical option even though this would increase treatment costs considerably.

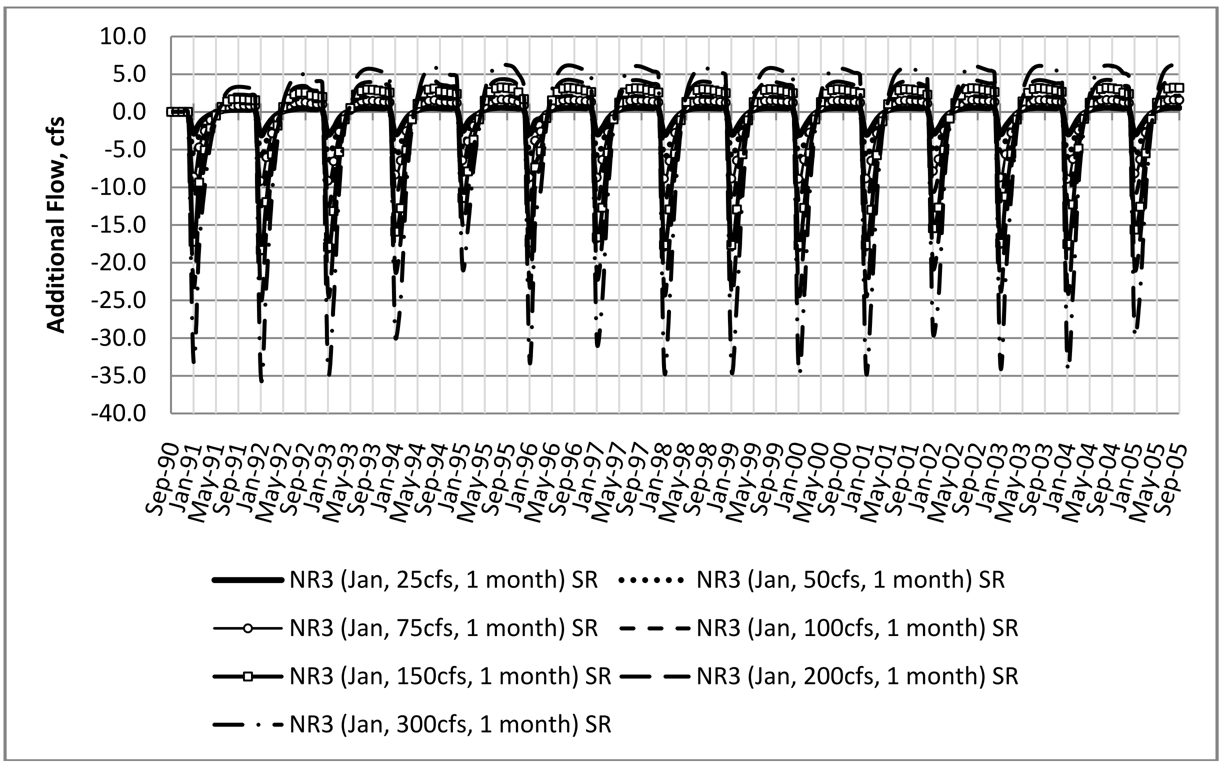

Figure 60. Comparison of river return flows using an extraction well near the Spokane River and a one month injection period.

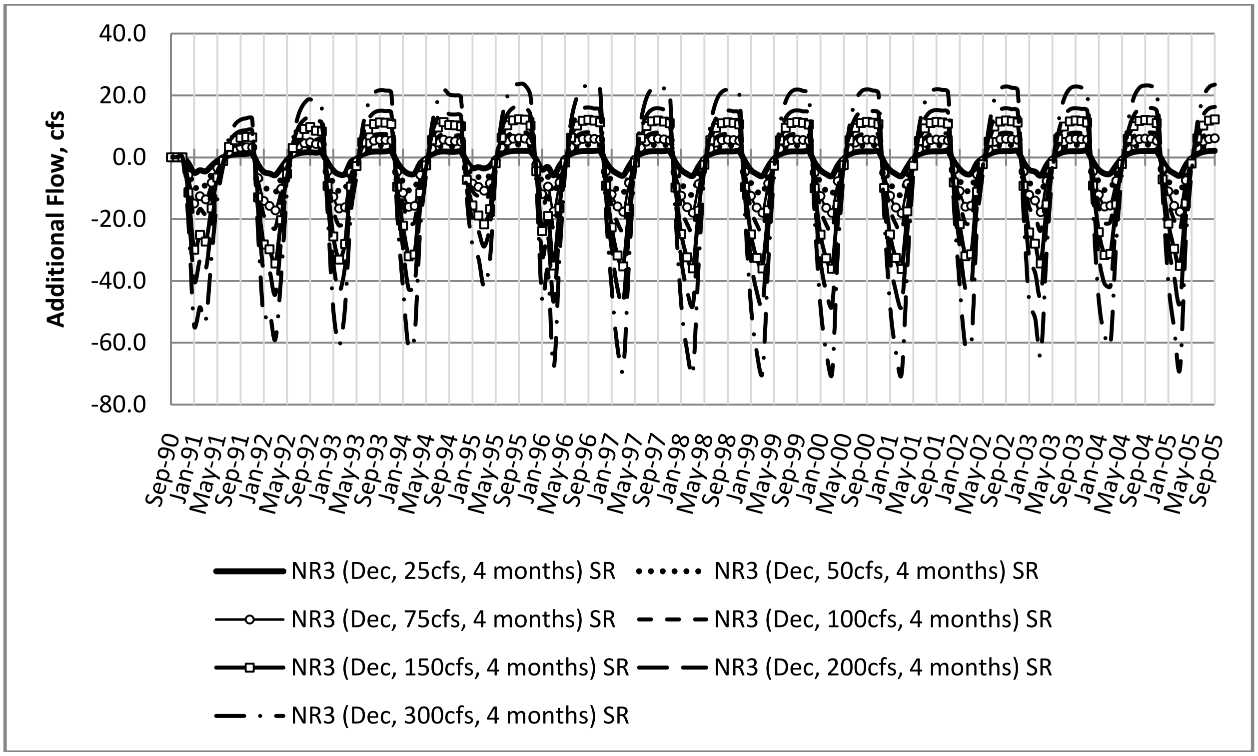

Figure 61. Comparison of river return flows using an extraction well near the Spokane River and a four month injection period.

<u>Impact of Distribution Pipeline Route</u>

The next series of figures show how the three alternative pipeline routes (northern railroad (NR), power line (PL), and southern railroad (SR)) compare in terms of timing and amount of water returned to the Spokane River. The goal was to determine whether or not changes in injection location along a nearly north-south/southeast line across the three routes would impact the timing and/or quantities of flow returning to the river. As seen in Figure 62, the third well in all three pipelines (NR3, PL3, and SR3) produces nearly the same return flow for the same injection scenarios. The same is true for the NR4, PL4, and SR4 and the NR 5, PL5, and SR5 cohorts. It should be noted that the results for the NR1, PL1, and SR1 and the NR2, PL2, and SR2 are not the same because of the presence of discontinuities in the aquifer such as Round Mountain and Eight Mile Prairie. But for locations 3 through 5, these figures show that moving the orientation of the injection wells north and south does not have the same affect on return flows to the Spokane River as does moving the wells east and west in the aquifer. This reduces the number of scenarios that need to be run because the results of all pipeline routes are nearly the same for most of the injection well locations.

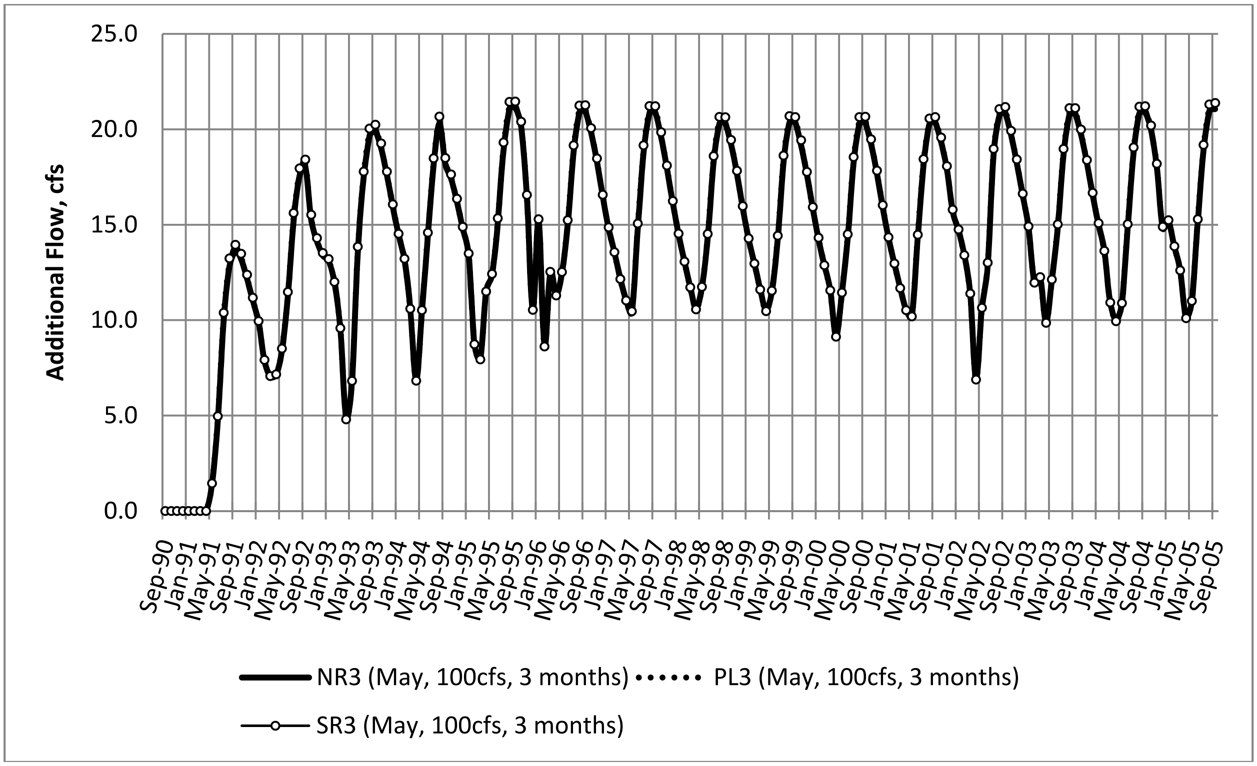

Figure 62. Evaluation of alternative pipeline routes using NR3, PL3, and SR3 locations.

The model was also run to determine the difference between injection in one well versus multiple wells. According to the principle of superposition, individual results obtained from running the model five times with subsequent injections at each of the five locations should sum to running the model once with identical injections at all five locations. To test this potential impact, 25 ft^3/s was injected into each of the five wells locations (NR1, NR2, NR3, NR4, and NR5) along the NR pipeline route for 1 month. Each of the wells began injection in March. The total extraction from the well field near Lake Pend Oreille was 125 ft^3/s for 1 month also starting in March. Figure 63 shows the data for the five individual well runs (NR1-NR5) that were added together to determine a total return flow to the Spokane River. The same thing was done in Visual MODFLOW-2009 by providing each NR1-NR5 site with 25 ft^3/s and pumping 125 ft^3/s

from the well field near Lake Pend Oreille. The results shown in Figure 64 indicate almost no difference between the actual Visual MODFLOW-2009 output data (dotted lines with circle) and for the individual wells added together (solid line). This indicates that individual wells can be added together to determine how multiple wells impact return flows.

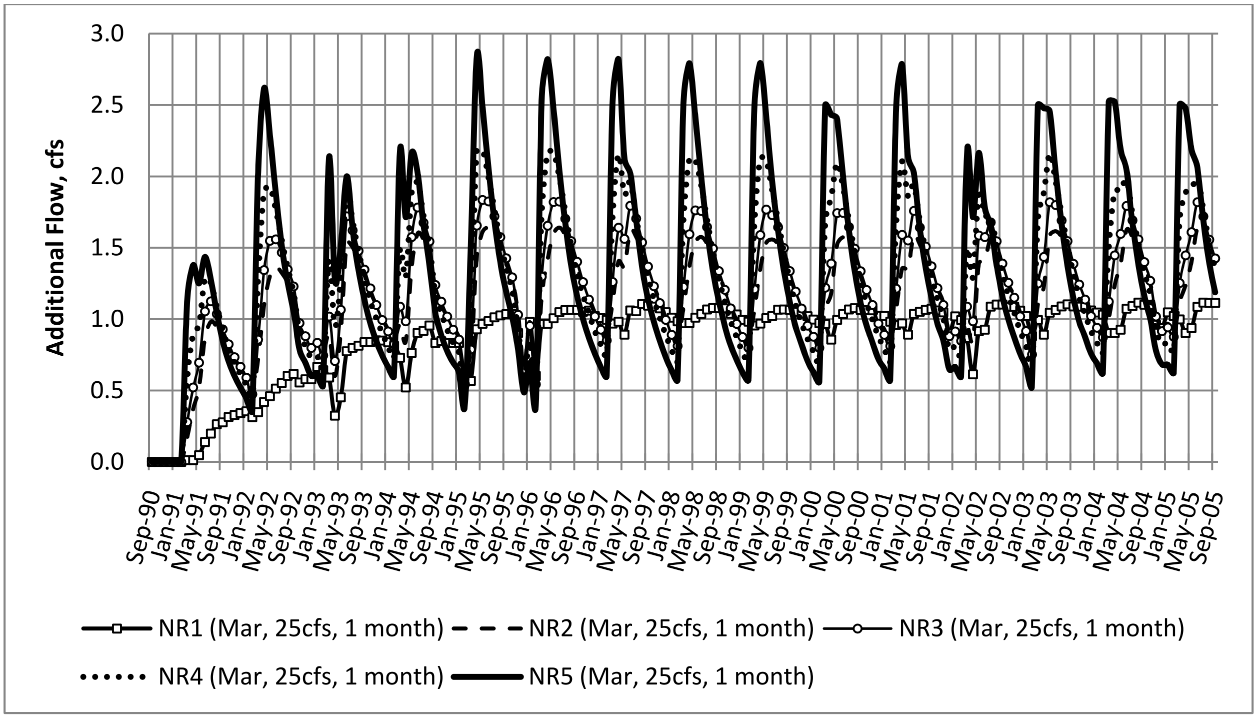

Figure 63. Flow data from individual wells used for superposition testing.

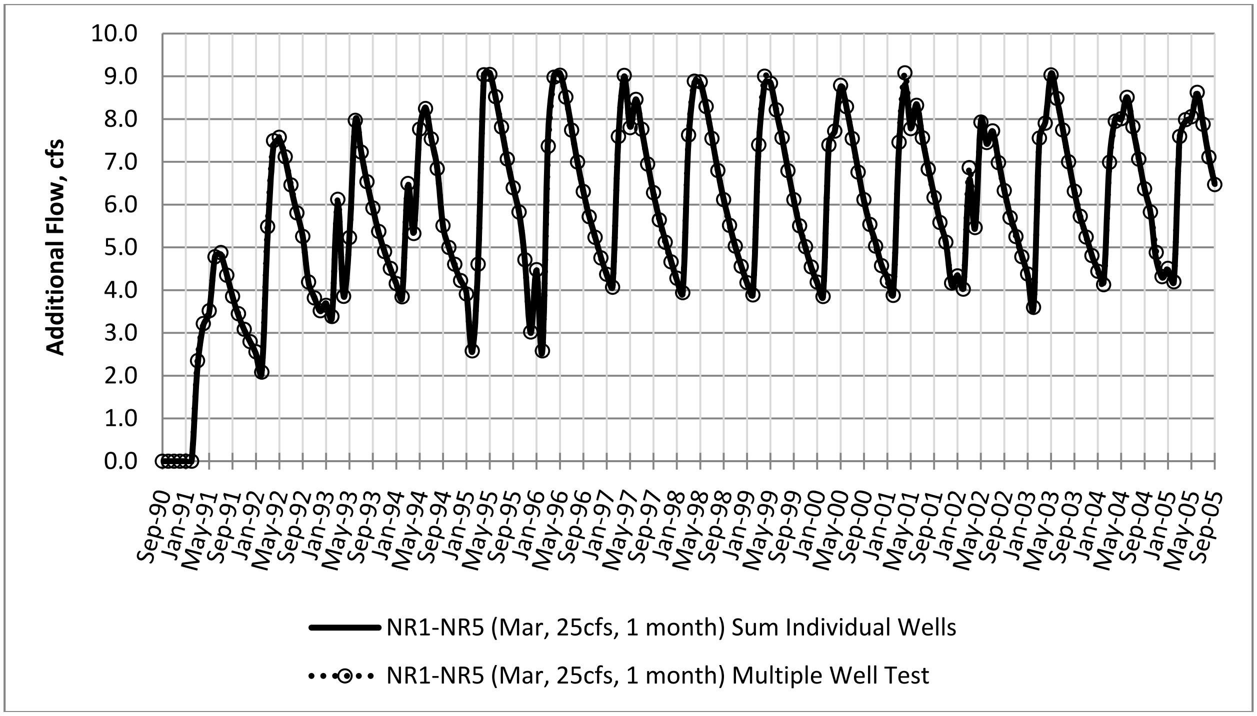

Figure 64. Superposition results for one month pumping and injection scenario.

The same 25 ft^3/s flow rate was used to test a 3 month injection period that started in May and ended in July. The three month injection period shown in Figure 63 is identical to the one month scenario in terms of ability to superimpose individual wells for a multiple well run. This indicates that duration of injection does not cause the process to become invalid. This greatly reduced the number of combinations that needed to be examined.

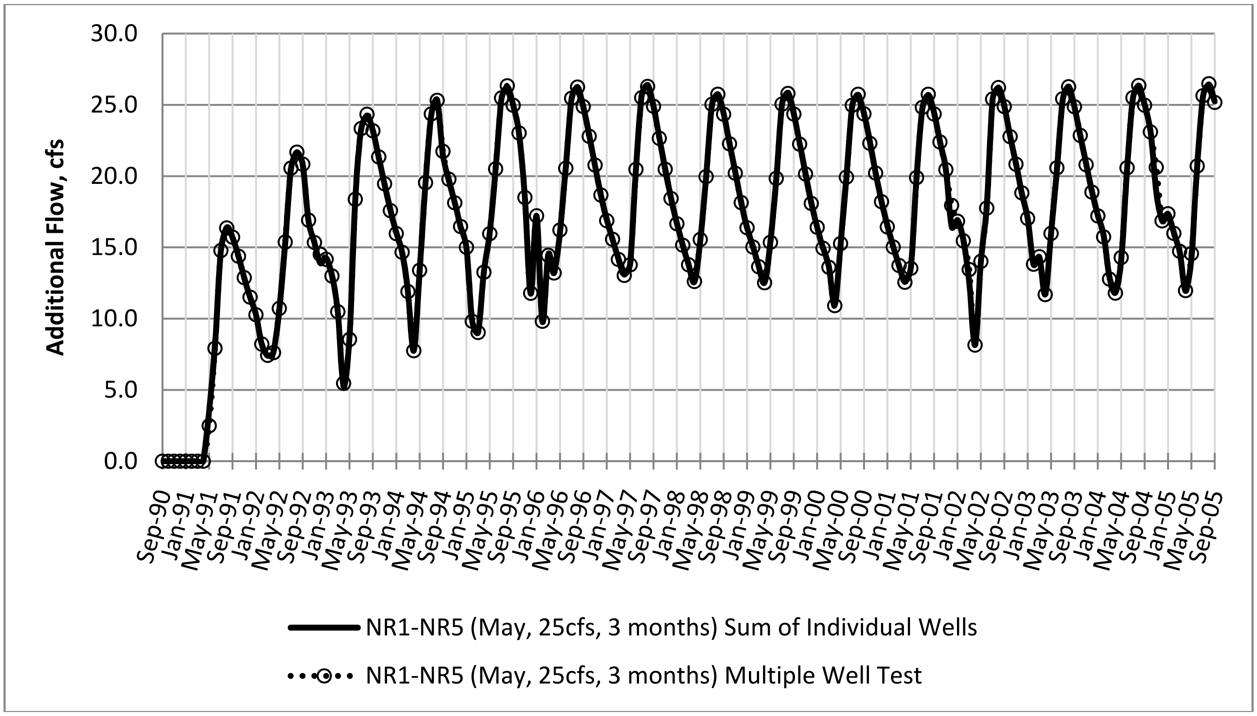

Figure 65. Superposition results for three month pumping and injection scenario.

A total flow rate of 125 ft^3/s was injected into the aquifer in both previous tests. Consequently, a comparison study of how the quantity of flow might impact the return flows from multiple injection sites and a single well for different flows was conducted. Flow rates of 100 ft^3/s and 150 ft^3/s and pumping durations of 1 and 3 months were used. Table 9 shows the additional return flows generated for each scenario. The total injected rate from the previous multiple injection wells scenario was 125 ft^3/s (25 ft^3/s x 5 wells). Return flows to the Spokane River from this run matched the average return flow value obtained from sequential single well injections of 100 ft^3/s and 150 ft^3/s at all five locations along the NR route. As Table 9 shows, the effect of using multiple injection wells is basically the same as taking the average return flow value from individual well results for each of the NR1 – NR5 sites flowing at 100 ft^3/s (which is 7.71 ft^3/s) plus the average for each of the NR1 – NR5 sites flowing at 150 ft^3/s (which is 11.55 ft^3/s) and taking the average of both values which is 9.63 ft^3/s. The return flow for the multiple injection wells scenario is 9.08 ft^3/s; very close to the 9.63 ft^3/s average value determined above. Table 9 also shows the amount of return flow from water injected at the NR3 site is very close to the average return flow to the Spokane River from all five injection wells. The significance of this is that injecting 25 ft^3/s at five multiple locations is very nearly the same as a 125 ft^3/s injection well scenario at NR3. Since NR3 is also the midpoint on the pipeline it is safe to say that using multiple injection sites is the same as averaging the return flow from the five injection well locations at the value of the sum of the multiple injection wells input (i.e., if 25 ft^3/s is used in each of the five wells contained in the multiple injection well scenario, averaging a 125 ft^3/s flow for each well individual would produce the same or nearly the same result). While not universally true for all aquifers, the high hydraulic conductivities present in the SVRP tends to dampen out the spatial differences in response especially when using a 1 month time step in the model. The same response occurs for the 3 month scenario. The practical application of the above discussion is that the return flow to the Spokane River from any combination of multiple injection wells can be determined from the average of NR1 – NR5 and the sum of the input of the multiple injection wells used.

Table 9. Comparison of results for superposition evaluation.

Location	NR1	NR2	NR3	NR4	NR5	NR1-NR5
Scenario	1 month, 100 ft^3/s, starting March					Note
Peak Additional Flow, ft^3/s	4.48	6.56	7.34	8.77	11.38	9.08
Scenario	1 month, 150 ft^3/s, starting March					Note
Peak Additional Flow, ft^3/s	6.72	9.86	11.01	13.10	17.05	9.08
Scenario	3 month, 100 ft^3/s, starting March					Note
Peak Additional Flow, ft^3/s	13.11	18.96	21.13	25.15	31.21	26.48
Scenario	3 month, 150 ft^3/s, starting March					Note
Peak Additional Flow, ft^3/s	19.69	28.43	31.69	37.72	46.80	26.48

Note: Scenario for NR1-NR5 is similar to given scenario but each well has 25 ft^3/s injected vs rate of the scenario

Figure 63 shows how the lag time to the Spokane River changed as the injection well locations moved towards the Spokane River. Starting injections in March at the NR1 and NR5 sites produces peak return flows in August and April, respectively (see Table 8). While the total amount of flow in August, September and October may be just as important as the peak flow, for this analysis only the peak monthly addition is being considered. Figure 66 shows the distribution of pumping length and starting month to get the peak return as close to August as possible. The blue shapes are bar charts of the periods when injections are taking place (1 to 4 months) and the light green rectangles are the peak return flow months. NR1 has the longest travel (lag) time of any of the probable injection well sites due to the relatively long distance down-gradient to the river. On occasion there is overlap between the period of injection and peak return months which is definitely not ideal for the ASR Project. For the NR5 location, the 4 month injection scenario is the only period the peak return month is in August. It is highlighted in red to distinguish it from the other well locations. The short lag-time likely makes this site unacceptable as diversions would have to take place late in the summer to have the intended impact on late summer flows.

Table 8 shows that the yearly percent of capture increases from 50% to 70% but never reaches 100% as the injection site moves from NR1 to NR5. The remaining 30 to 50% of the water is captured flow that is already flowing to the aquifer and, therefore, already accounted for in the model. As the system limitations section described, the additional flow from Lake Pend Oreille is less than the total amount of water withdrawn by the extraction well(s). This occurs because some of the extracted water comes from segments of the aquifer recharged by other sources of water entering from other nearby drainage areas. This remaining flow represents water that would have naturally moved away from Lake Pend Oreille. As the injection wells move closer to the extraction well, the rise in the groundwater table from the injection starts to feed the extraction well. This causes the percent of new water (water pulled from Lake Pend Oreille) to decrease and the amount of aquifer water used to increase. Thus the 30% to 50% gap between what is captured and what is expected is due to the method of accounting rather than a hole in the water budget. Table 10 shows the maximum withdrawal rate from Lake Pond Oreille and the average percent of the total yearly pumping extracted from Lake.

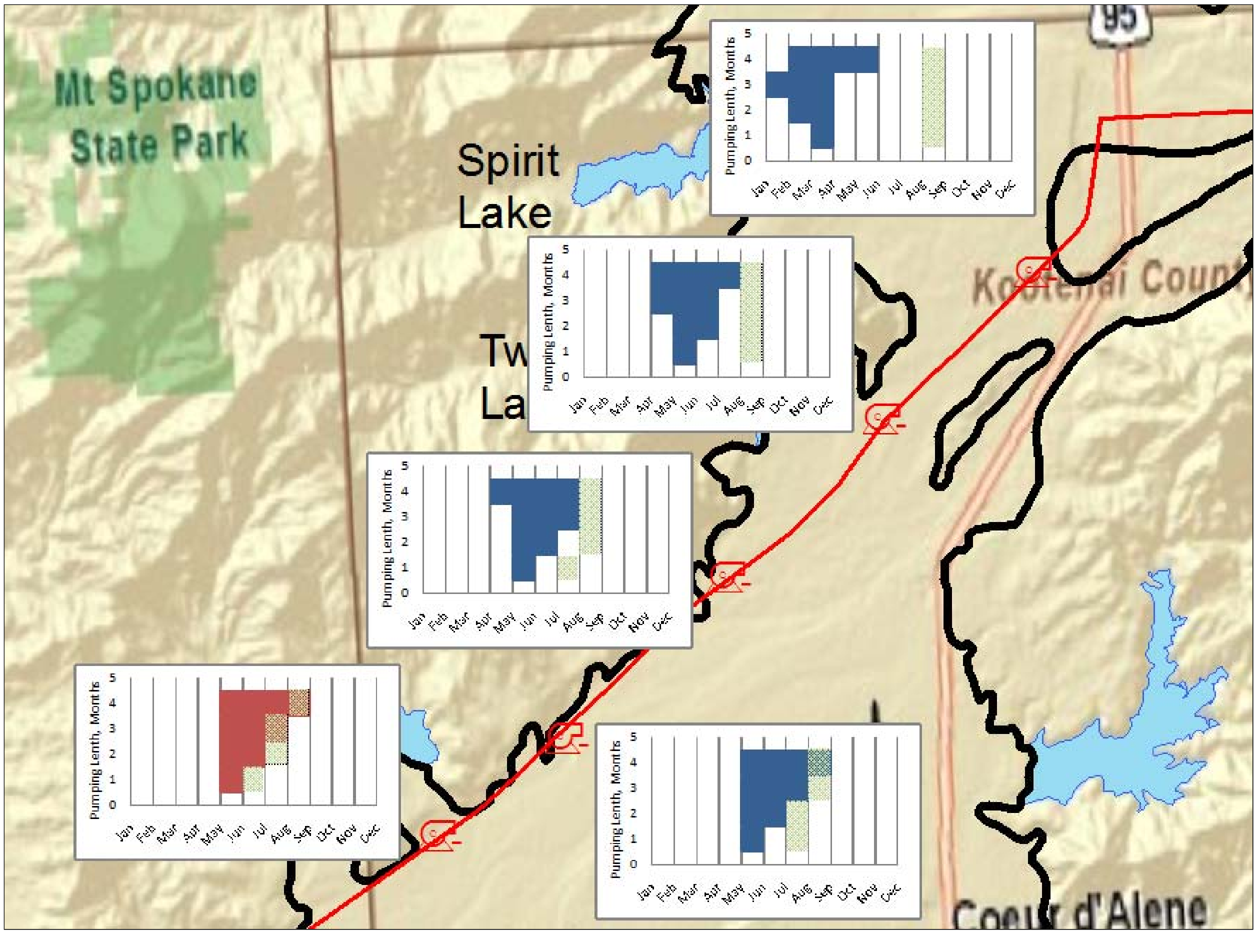

Figure 66. Representative injection period charts.

Table 10 is missing most of the one month scenarios because the zone budget for Lake Pend Oreille was not yet established during the analysis. Since subsequent analysis indicated that one month extraction scenarios were not going to be economically feasible, we did not go back and repeat these analyses. The Spokane River scenarios caused less flow to come out of Lake Pend Oreille by a small amount and shows how even at the distance the Spokane extraction well was from Lake Pend Oreille the changes in the aquifer were able to propagate up the aquifer to the Lake. In addition, the maximum percentage return is the monthly return divided by the cumulative amount of flow injected for a given year (ft^3). For example, if the extraction wells were run for 2 months (61 days) at 100 ft^3/s starting in May, a total of approximately 5.27 * 10^8 ft^3 would be extracted and subsequently be injected at one of the injections sites. Furthermore, if 65.84 ft^3/s was the induced flow from Lake Pend Oreille in the month of June, then the total flow from the lake would be 1.707 * 10^8 ft^3 (65.84 ft^3/s * 30 days in June * 86400 seconds per day). Dividing the induced flow by the cumulative extracted flow (5.27 * 10^8 ft^3) gives the 32.38% shown in the table. The same method is also used in Table 10 for determining the maximum monthly percent return flows.

Table 10. Lake Pend Oreille withdrawals.

Location	Starting Month	Rate (ft^3/s)	Length of Injection (months)	Maximum Monthly Withdrawal (ft^3/s)	Max Monthly % Return	Average Yearly Return	Max Month
NR1	Jan	25	1	N/A	N/A	N/A	N/A
NR1	Feb	25	1	N/A	N/A	N/A	N/A
NR1	Mar	25	1	N/A	N/A	N/A	N/A
NR1	Apr	25	1	N/A	N/A	N/A	N/A
NR1	May	25	1	N/A	N/A	N/A	N/A
NR1	Dec	25	1	N/A	N/A	N/A	N/A
NR1	Mar	50	1	N/A	N/A	N/A	N/A
NR1	Mar	75	1	N/A	N/A	N/A	N/A
NR1	Mar	100	1	N/A	N/A	N/A	N/A
NR1	Mar	150	1	N/A	N/A	N/A	N/A
NR1	Mar	200	1	N/A	N/A	N/A	N/A
NR1	Mar	300	1	N/A	N/A	N/A	N/A
NR1	Jan	25	2	N/A	N/A	N/A	N/A
NR1	Feb	25	2	N/A	N/A	N/A	N/A
NR1	Mar	25	2	N/A	N/A	N/A	N/A
NR1	Apr	25	2	N/A	N/A	N/A	N/A
NR1	May	25	2	N/A	N/A	N/A	N/A
NR1	Dec	25	2	N/A	N/A	N/A	N/A
NR1	Feb	50	2	N/A	N/A	N/A	N/A
NR1	Feb	75	2	N/A	N/A	N/A	N/A
NR1	Feb	100	2	N/A	N/A	N/A	N/A
NR1	Feb	150	2	N/A	N/A	N/A	N/A
NR1	Feb	200	2	N/A	N/A	N/A	N/A
NR1	Feb	300	2	N/A	N/A	N/A	N/A
NR1	Jan	25	3	17.79	24.51%	58.70%	March
NR1	Feb	25	3	16.26	22.66%	30.21%	March
NR1	Mar	25	3	17.82	24.02%	58.69%	May
NR1	Apr	25	3	17.78	23.44%	58.72%	June
NR1	May	25	3	17.79	23.98%	58.69%	July
NR1	Dec	25	3	17.77	22.58%	57.44%	January
NR1	Jan	50	3	35.54	24.49%	58.69%	March

Location	Starting Month	Rate (ft^3/s)	Length of Injection (months)	Maximum Monthly Withdrawal (ft^3/s)	Max Monthly % Return	Average Yearly Return	Max Month
NR1	Jan	75	3	53.26	24.46%	58.68%	March
NR1	Jan	100	3	70.96	24.44%	58.66%	March
NR1	Jan	150	3	106.25	24.40%	58.66%	March
NR1	Jan	200	3	141.33	24.34%	58.66%	March
NR1	Jan	300	3	221.02	25.38%	60.42%	March
NR1	Jan	25	4	18.38	18.38%	58.70%	April
NR1	Feb	25	4	18.37	18.98%	58.78%	May
NR1	Dec	25	4	18.38	18.84%	57.74%	March
NR1	Feb	50	4	36.66	18.94%	58.75%	May
NR1	Feb	75	4	54.94	18.92%	58.74%	May
NR1	Feb	100	4	73.19	18.91%	58.72%	May
NR1	Feb	150	4	109.64	18.88%	58.73%	May
NR1	Feb	200	4	146.02	18.86%	58.76%	May
NR1	Feb	300	4	226.52	19.51%	60.51%	May
NR2	Jan	25	1	12.38	49.53%	66.57%	January
NR2	Feb	25	1	N/A	N/A	N/A	N/A
NR2	Mar	25	1	N/A	N/A	N/A	N/A
NR2	Apr	25	1	N/A	N/A	N/A	N/A
NR2	May	25	1	N/A	N/A	N/A	N/A
NR2	May	50	1	N/A	N/A	N/A	N/A
NR2	May	75	1	N/A	N/A	N/A	N/A
NR2	May	100	1	N/A	N/A	N/A	N/A
NR2	May	150	1	N/A	N/A	N/A	N/A
NR2	May	200	1	N/A	N/A	N/A	N/A
NR2	May	300	1	N/A	N/A	N/A	N/A
NR2	Feb	25	2	N/A	N/A	N/A	N/A
NR2	Mar	25	2	N/A	N/A	N/A	N/A
NR2	Apr	25	2	N/A	N/A	N/A	N/A
NR2	May	25	2	N/A	N/A	N/A	N/A
NR2	May	50	2	N/A	N/A	N/A	N/A
NR2	May	75	2	N/A	N/A	N/A	N/A
NR2	May	100	2	N/A	N/A	N/A	N/A
NR2	May	150	2	N/A	N/A	N/A	N/A
NR2	May	200	2	N/A	N/A	N/A	N/A
NR2	May	300	2	205.42	33.67%	68.10%	June
NR2	Mar	25	3	17.69	23.84%	66.54%	May
NR2	Apr	25	3	17.64	23.26%	66.50%	June
NR2	May	25	3	17.65	23.79%	66.42%	July
NR2	Apr	50	3	35.28	23.26%	66.48%	June

Location	Starting Month	Rate (ft^3/s)	Length of Injection (months)	Maximum Monthly Withdrawal (ft^3/s)	Max Monthly % Return	Average Yearly Return	Max Month
NR2	Apr	75	3	52.92	23.26%	66.48%	June
NR2	Apr	100	3	17.78	5.86%	14.68%	June
NR2	Apr	150	3	105.76	23.24%	66.45%	June
NR2	Apr	200	3	140.95	23.23%	66.42%	June
NR2	Apr	300	3	220.25	24.20%	68.09%	June
NR2	Mar	25	4	18.19	17.98%	66.51%	May
NR2	Apr	25	4	18.18	18.48%	66.45%	July
NR2	May	25	4	18.17	18.32%	66.32%	August
NR2	Apr	50	4	36.35	18.48%	66.44%	July
NR2	Apr	75	4	54.53	18.47%	66.43%	July
NR2	Apr	100	4	72.70	18.47%	66.41%	July
NR2	Apr	150	4	109.02	18.47%	66.39%	July
NR2	Apr	200	4	145.31	18.46%	66.36%	July
NR2	Apr	300	4	225.72	19.12%	68.02%	July
NR3	Feb	25	1	N/A	N/A	N/A	N/A
NR3	Mar	25	1	N/A	N/A	N/A	N/A
NR3	Apr	25	1	N/A	N/A	N/A	N/A
NR3	May	25	1	N/A	N/A	N/A	N/A
NR3	May	50	1	N/A	N/A	N/A	N/A
NR3	May	75	1	N/A	N/A	N/A	N/A
NR3	May	100	1	N/A	N/A	N/A	N/A
NR3	May	150	1	N/A	N/A	N/A	N/A
NR3	May	200	1	N/A	N/A	N/A	N/A
NR3	May	300	1	N/A	N/A	N/A	N/A
NR3	Mar	25	2	N/A	N/A	N/A	N/A
NR3	Apr	25	2	N/A	N/A	N/A	N/A
NR3	May	25	2	N/A	N/A	N/A	N/A
NR3	May	50	2	N/A	N/A	N/A	N/A
NR3	May	75	2	N/A	N/A	N/A	N/A
NR3	May	100	2	N/A	N/A	N/A	N/A
NR3	May	150	2	N/A	N/A	N/A	N/A
NR3	May	200	2	N/A	N/A	N/A	N/A
NR3	May	300	2	207.12	33.95%	71.85%	June
NR3	Mar	25	3	17.93	24.43%	71.03%	May
NR3	Apr	25	3	17.89	23.60%	70.20%	June
NR3	May	25	3	17.91	24.13%	70.09%	July
NR3	May	50	3	35.81	24.13%	70.09%	July
NR3	May	75	3	53.69	24.12%	70.09%	July
NR3	May	100	3	71.59	24.12%	70.08%	July

Location	Starting Month	Rate (ft^3/s)	Length of Injection (months)	Maximum Monthly Withdrawal (ft^3/s)	Max Monthly % Return	Average Yearly Return	Max Month
NR3	May	150	3	107.33	24.11%	70.08%	July
NR3	May	200	3	143.01	24.09%	70.07%	July
NR3	May	300	3	223.38	25.09%	71.79%	July
NR3	Mar	25	4	18.55	18.25%	70.23%	June
NR3	Apr	25	4	18.54	18.85%	70.14%	July
NR3	May	25	4	18.54	18.69%	69.97%	August
NR3	Apr	50	4	37.07	18.84%	70.13%	July
NR3	Apr	75	4	55.61	18.84%	70.13%	July
NR3	Apr	100	4	74.14	18.84%	70.13%	July
NR3	Apr	150	4	111.19	18.84%	70.12%	July
NR3	Apr	200	4	148.23	18.83%	70.10%	July
NR3	Apr	300	4	230.13	19.49%	71.80%	July
NR3[A]	Jan	25	1	0.00	-0.14%	-8.00%	December
NR3[A]	Feb	25	1	0.00	0.00%	-8.05%	January
NR3[A]	Mar	25	1	0.00	0.00%	-7.93%	February
NR3[A]	May	25	1	0.00	0.00%	-7.88%	April
NR3[A]	Jan	50	1	0.00	-0.14%	-7.99%	December
NR3[A]	Jan	75	1	0.00	-0.14%	-8.00%	December
NR3[A]	Jan	100	1	0.00	-0.14%	-8.01%	December
NR3[A]	Jan	150	1	0.00	-0.14%	-8.01%	December
NR3[A]	Jan	200	1	0.00	-0.14%	-8.01%	December
NR3[A]	Jan	300	1	0.00	-0.14%	-7.87%	December
NR3[A]	Jan	25	4	0.00	-0.04%	-7.98%	February
NR3[A]	Dec	25	4	0.00	-0.12%	-8.00%	November
NR3[A]	Dec	50	4	0.00	-0.12%	-8.01%	November
NR3[A]	Dec	75	4	0.00	-0.12%	-8.02%	November
NR3[A]	Dec	100	4	0.00	-0.12%	-8.02%	November
NR3[A]	Dec	150	4	0.00	-0.12%	-8.03%	November
NR3[A]	Dec	200	4	0.00	-0.12%	-8.03%	November
NR3[A]	Dec	300	4	0.00	-0.12%	-7.88%	November
NR4	Apr	25	1	N/A	N/A	N/A	N/A
NR4	Mar	25	1	N/A	N/A	N/A	N/A
NR4	May	25	1	N/A	N/A	N/A	N/A
NR4	May	50	1	N/A	N/A	N/A	N/A
NR4	May	75	1	N/A	N/A	N/A	N/A
NR4	May	100	1	N/A	N/A	N/A	N/A
NR4	May	150	1	N/A	N/A	N/A	N/A
NR4	May	200	1	N/A	N/A	N/A	N/A
NR4	May	300	1	N/A	N/A	N/A	N/A

Location	Starting Month	Rate (ft^3/s)	Length of Injection (months)	Maximum Monthly Withdrawal (ft^3/s)	Max Monthly % Return	Average Yearly Return	Max Month
NR4	Apr	25	2	N/A	N/A	N/A	N/A
NR4	Mar	25	2	N/A	N/A	N/A	N/A
NR4	May	25	2	N/A	N/A	N/A	N/A
NR4	May	50	2	N/A	N/A	N/A	N/A
NR4	May	75	2	N/A	N/A	N/A	N/A
NR4	May	100	2	N/A	N/A	N/A	N/A
NR4	May	150	2	N/A	N/A	N/A	N/A
NR4	May	200	2	N/A	N/A	N/A	N/A
NR4	May	300	2	208.11	34.12%	75.31%	June
NR4	Apr	25	3	18.06	23.81%	73.64%	June
NR4	May	25	3	18.07	24.35%	73.49%	July
NR4	May	50	3	36.13	24.35%	73.50%	July
NR4	May	75	3	54.18	24.34%	73.50%	July
NR4	May	100	3	72.23	24.34%	73.50%	July
NR4	May	150	3	108.29	24.33%	73.51%	July
NR4	May	200	3	144.31	24.31%	73.51%	July
NR4	May	300	3	225.31	25.31%	75.24%	July
NR4	Apr	25	4	18.78	19.09%	73.56%	July
NR4	May	25	4	18.78	18.94%	73.36%	August
NR4	May	50	4	37.56	18.93%	73.36%	August
NR4	May	75	4	56.33	18.93%	73.36%	August
NR4	May	100	4	75.10	18.93%	73.36%	August
NR4	May	150	4	112.63	18.92%	73.37%	August
NR4	May	200	4	150.13	18.92%	73.36%	August
NR4	May	300	4	233.03	19.58%	75.10%	August
NR5	Apr	25	1	N/A	N/A	N/A	N/A
NR5	Mar	25	1	N/A	N/A	N/A	N/A
NR5	May	25	1	N/A	N/A	N/A	N/A
NR5	May	50	1	N/A	N/A	N/A	N/A
NR5	May	75	1	N/A	N/A	N/A	N/A
NR5	May	100	1	50.09	50.09%	75.81%	May
NR5	May	150	1	N/A	N/A	N/A	N/A
NR5	May	200	1	N/A	N/A	N/A	N/A
NR5	May	300	1	164.26	54.75%	77.53%	May
NR5	Apr	25	2	N/A	N/A	N/A	N/A
NR5	Mar	25	2	N/A	N/A	N/A	N/A
NR5	May	25	2	N/A	N/A	N/A	N/A
NR5	May	50	2	N/A	N/A	N/A	N/A
NR5	May	75	2	N/A	N/A	N/A	N/A

Location	Starting Month	Rate (ft^3/s)	Length of Injection (months)	Maximum Monthly Withdrawal (ft^3/s)	Max Monthly % Return	Average Yearly Return	Max Month
NR5	May	100	2	N/A	N/A	N/A	N/A
NR5	May	150	2	98.82	32.40%	75.74%	June
NR5	May	200	2	131.59	32.36%	75.74%	June
NR5	May	300	2	208.38	34.16%	77.47%	June
NR5	Apr	25	3	18.10	23.87%	75.79%	June
NR5	May	25	3	18.11	24.41%	75.64%	July
NR5	May	50	3	36.23	24.41%	75.64%	July
NR5	May	75	3	54.33	24.41%	75.64%	July
NR5	May	100	3	72.43	24.41%	75.65%	July
NR5	May	150	3	108.59	24.39%	75.65%	July
NR5	May	200	3	144.70	24.38%	75.65%	July
NR5	May	300	3	225.90	25.37%	77.39%	July
NR5	Apr	25	4	18.87	19.18%	75.70%	July
NR5	May	25	4	18.87	19.02%	75.49%	August
NR5	May	50	4	37.72	19.01%	75.49%	August
NR5	May	75	4	56.57	19.01%	75.49%	August
NR5	May	100	4	75.43	19.01%	75.50%	August
NR5	May	150	4	113.11	19.01%	75.50%	August
NR5	May	200	4	150.78	19.00%	75.51%	August
NR5	Apr	300	4	233.98	19.82%	77.45%	July
NR5	May	300	4	234.00	19.66%	77.24%	August
PL1	Mar	100	1	49.96	49.96%	57.40%	March
PL1	Feb	100	2	66.13	34.74%	56.81%	March
PL1	Jan	100	3	73.03	25.16%	55.89%	March
PL1	Feb	100	4	76.40	19.74%	55.50%	May
PL2	May	100	1	49.43	49.43%	65.82%	May
PL2	May	100	2	64.85	31.89%	65.78%	June
PL2	Apr	100	3	70.36	23.20%	65.80%	June
PL2	Apr	100	4	72.43	18.40%	65.75%	July
PL3	May	25	1	12.45	49.81%	70.39%	May
PL3	May	50	1	24.86	49.72%	70.37%	May
PL3	May	75	1	37.28	49.71%	70.38%	May
PL3	May	100	1	49.66	49.66%	70.38%	May
PL3	May	150	1	74.36	49.58%	70.38%	May
PL3	May	200	1	99.00	49.50%	70.38%	May
PL3	May	300	1	163.55	54.52%	72.08%	May
PL3	May	25	2	16.40	32.26%	70.33%	June
PL3	May	50	2	32.79	32.25%	70.33%	June
PL3	May	75	2	49.17	32.24%	70.33%	June

Location	Starting Month	Rate (ft^3/s)	Length of Injection (months)	Maximum Monthly Withdrawal (ft^3/s)	Max Monthly % Return	Average Yearly Return	Max Month
PL3	May	100	2	65.54	32.23%	70.32%	June
PL3	May	150	2	98.22	32.20%	70.32%	June
PL3	May	200	2	130.79	32.16%	70.32%	June
PL3	May	300	2	163.55	27.71%	50.40%	May
PL3	May	100	3	71.62	24.13%	70.25%	July
PL3	Apr	100	4	74.20	18.85%	70.30%	July
PL4	May	100	1	49.90	49.90%	73.59%	May
PL4	May	100	2	65.84	32.38%	73.53%	June
PL4	May	100	3	72.22	24.34%	73.45%	July
PL4	May	100	4	75.09	18.93%	73.31%	August
PL5	May	100	1	50.14	50.14%	76.42%	May
PL5	May	100	2	65.96	32.44%	76.35%	June
PL5	May	100	3	72.47	24.42%	76.26%	July
PL5	May	100	4	75.50	19.03%	76.11%	August
SR1	Mar	100	1	49.78	49.78%	61.22%	March
SR1	Feb	100	2	65.58	34.46%	61.18%	March
SR1	Jan	100	3	72.51	24.98%	60.76%	March
SR1	Feb	100	4	75.82	19.59%	60.63%	May
SR2	May	100	1	49.81	49.81%	71.22%	May
SR2	May	100	2	66.02	32.47%	70.95%	June
SR2	Apr	100	3	72.53	23.91%	70.81%	June
SR2	Apr	100	4	75.72	19.24%	70.60%	July
SR3	May	100	1	49.69	49.69%	70.88%	May
SR3	May	25	2	16.42	32.31%	70.83%	June
SR3	May	50	2	32.82	32.29%	70.82%	June
SR3	May	75	2	49.22	32.28%	70.82%	June
SR3	May	100	2	65.61	32.27%	70.82%	June
SR3	May	150	2	98.32	32.24%	70.82%	June
SR3	May	200	2	130.93	32.19%	70.82%	June
SR3	May	300	2	207.40	34.00%	72.53%	June
SR3	May	25	3	17.95	24.20%	70.76%	July
SR3	May	50	3	35.89	24.19%	70.75%	July
SR3	May	75	3	53.83	24.18%	70.75%	July
SR3	May	100	3	71.76	24.18%	70.75%	July
SR3	May	150	3	107.58	24.17%	70.75%	July
SR3	May	200	3	143.36	24.15%	70.75%	July
SR3	May	300	3	223.90	25.15%	72.47%	July
SR3	Apr	100	4	74.40	18.90%	70.80%	July
SR4	May	100	1	49.89	49.89%	73.45%	May

Location	Starting Month	Rate (ft^3/s)	Length of Injection (months)	Maximum Monthly Withdrawal (ft^3/s)	Max Monthly % Return	Average Yearly Return	Max Month
SR4	May	100	2	65.84	32.38%	73.39%	June
SR4	May	100	3	72.22	24.34%	73.31%	July
SR4	May	100	4	75.09	18.93%	73.17%	August
SR5	May	100	1	50.11	50.11%	76.03%	May
SR5	May	100	2	65.84	32.38%	73.39%	June
SR5	May	100	3	72.44	24.41%	75.87%	July
SR5	May	100	4	75.45	19.02%	75.73%	August
NR1-NR5[B]	Mar	125	1	61.96	49.56%	69.05%	March
NR1-NR5[B]	May	125	3	89.47	24.12%	68.87%	July

[A] Denotes the test done with the well near the Spokane River vs. near Lake Pond Oreille as all the other tests.
[B] Denotes the multiple wells test, with each well injecting 25 ft^3/s for a total of 125 ft^3/s pumped and injected.

6.2.2 Infiltration Basins

The difference between injection wells and infiltration basins is the initial time it takes for water to reach the aquifer. In some areas the SVRP aquifer is several hundreds of feet below the ground surface so water placed in infiltration basins would take some period of time to reach the aquifer thereby increasing the lag time compared to direct injection. Because the actual travel time depends on variable saturation relationships, HYDRUS3D was used to estimate vertical travel times. HYDRUS3D is a finite element model that simulates three-dimensional movement of water in variably saturated media (Šimůnek and Šenja, 2007). The program numerically solves Richards' equation for saturated and unsaturated water flow as well as convection-dispersion type equations for heat and solute transport. HYDRUS3D generates a table of water contents, hydraulic conductivities, and specific water capacities from the specified set of hydraulic parameters for each soil type in the flow domain. These values are used to set iteration criteria within the model.

Arguably the largest uncertainty in the model comes from the assumptions surrounding the selection of the appropriate soil hydraulic model (hydraulic conductivity as a function of soil moisture curve) and its' associated parameters. HYDRUS3D allows the user to select from eight different models. Garcia (2010) used soil moisture characteristics at four weather stations distributed throughout the SVRP aquifer to determine that the van Genuchten function was superior to other options for matching the field data. Table 11 summarizes the values for the three van Genuchten parameters in the upper 1.25 meters of soil at the four test locations (the Spokane Valley Fire Department, WA (SVFD), the Consolidated Irrigation District (CID) pump station near the WA/ID state line, the City of Athol, ID (Athol), and the US Forest Service (USFS) facility near Coeur d'Alene, Idaho).

Table 11. van Genuchten soil parameters (Garcia, 2010).

Curve-fitting Parameters		Athol			CID			SVFD			USFS		
		0-20 cm	21-65 cm	66-125 cm	0-20 cm	21-65 cm	66-125 cm	0-20 cm	21-65 cm	66-125 cm	0-20 cm	21-65 cm	66-125 cm
van Genuchten	α	0.023	0.04	0.04	0.023	0.037	0.055	0.025	0.032	0.045	0.028	0.036	0.045
	n	1.41	1.41	1.98	1.71	1.81	2.08	1.81	1.81	1.89	1.51	1.46	2.68
	m	0.2908	0.2908	0.4949	0.4152	0.4475	0.5192	0.4475	0.4475	0.4709	0.3377	0.3151	0.6269

The degree to which these near-surface properties extend down to the water table is currently unknown and consequently introduces uncertainty into the computations. Furthermore, spatial variability of soil properties between the four locations and the final location of the infiltration trenches may create additional discrepancies in the actual infiltration compared to estimated infiltration. Because of the uncertainty associated with current estimates of saturated vertical hydraulic conductivity, sensitivity analyses were conducted by varying the travel time through the unsaturated layer. If infiltration basins are the preferred alternative, additional recommendations will be made on studies needed to reduce the uncertainty.

Similar to injection wells, the locations of the infiltration basins are important because sites close to the river will drain fairly rapidly and sites too far away may drain too slowly and therefore not produce the desired delay into the summer months. Combinations of injection wells and infiltration basins at multiple locations can be used to achieve the appropriate lag and desired increase in summer discharges.

In order to avoid exposing the water to the surface and thus risk having to treat it, the infiltration gallery would consist of a series buried open-bottomed containers. Examples of these high strength modular stormwater infiltration devices are commercially called VersiTank® by the Elmich Company (see Figure 67) or Aquablox® by Aquascape. These crates are buried underground after being covered with a construction fabric to prevent soil from penetrating the openings. Thus they can be located beneath parking lots, parks and open spaces, or along existing road right-of-ways.

Figure 67. Example of infiltration devices (www.elmich.com).

Determining the surface area over which the water should be distributed involved matching the design flow to the desired lag-time and the appropriate infiltration rate. For example, 100 ft^3/s is equivalent to approximately 200 acre-ft/day. Over a one-acre site, this would require 200 feet of infiltration per day. Conversely, over a twenty-acre site, only 10 feet of infiltration per day would be required.

6.2.2.1 Infiltration Basin: HYDRUS3D and HYDRUS2D Setups

The general region surrounding the NR3 injection site (see Figure 68) was chosen as the location of the variable infiltration site. As previously discussed in the direct injection section, this location serves roughly as an average site with respect to the percentage of water returned to the Spokane River. The conditions at this site are shown in Table 12.

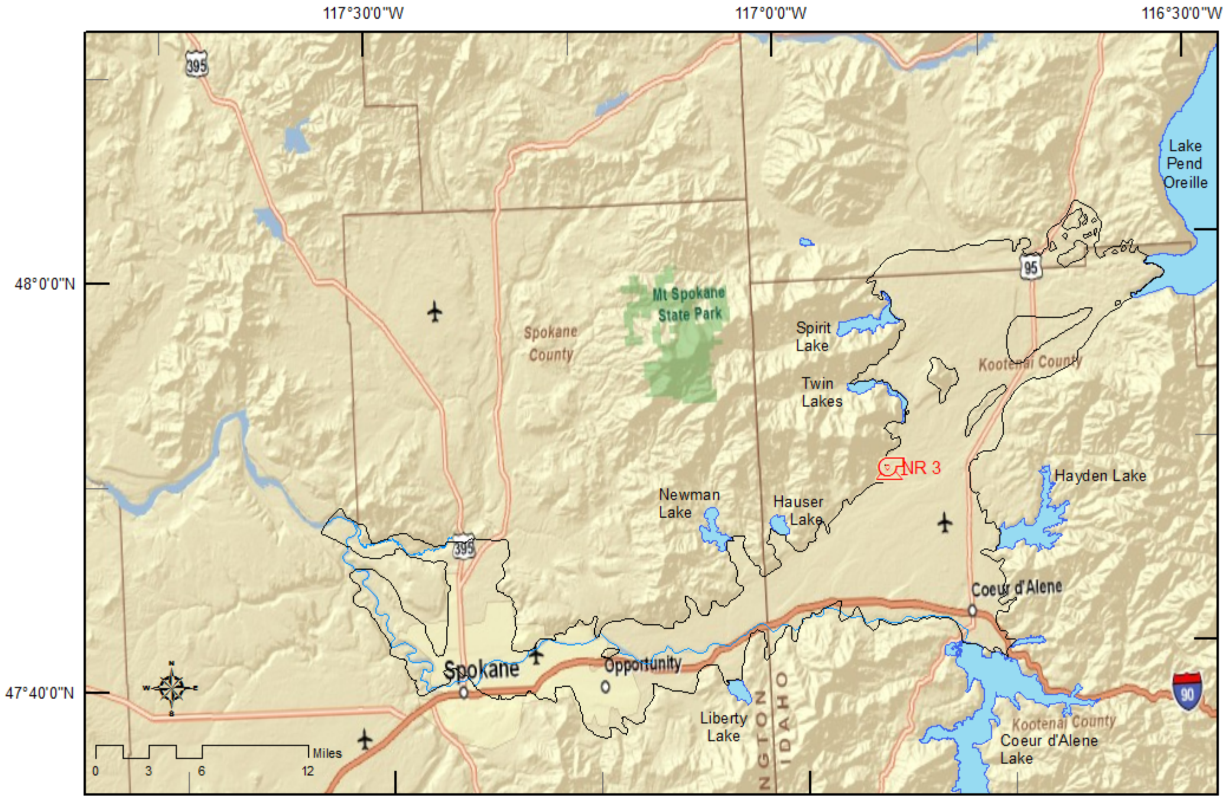

Figure 68. Location of infiltration basin site.

Table 12. Aquifer properties near the northern railroad (NR) sites.

Well Name	Latitude	Longitude	Depth Aquifer Bottom	Average Thickness of Water Table	Hydraulic Conductivity, K	Transmissivity T	Specific Yield Sy
			(ft)	(ft)	(ft/day)	(ft^2/day)	[ft^3/ft^3]
NR1	47.904176	-116.740043	494	148	94	13946	0.21
NR2	47.863687	-116.808549	427	117	17080	2000409	0.19
NR3	**47.820307**	**-116.879078**	**414**	**179**	**12080**	**2167816**	**0.19**
NR4	47.775754	-116.952159	423	270	12080	3257734	0.19
NR5	47.74967	-117.009032	365	224	22150	4951853	0.10

Using this information, a total of 18 scenarios were run to determine the time it would take for water distributed uniformly over an infiltration basin to reach the water table. The bottom of the basin or trench was set at 6.6 feet below the soil surface. This allowed space for the supply pipeline, an Aquablox or VersiTank storage container, and ensured that direct contact with the atmosphere was not possible thus preserving the groundwater status of the water. Figure 69 shows the basin size needed to infiltrate the flows used in the direct injection scenarios. As illustrated, relatively large basins are needed to infiltrate even small rates of flow. The size of the basin was initially determined by the area needed to infiltrate 25 ft^3/s at a uniform velocity of approximately 1 ft/day. This required a surface area of 47.4 acres. The rationale for the 1 ft/day value is explained later in this section.

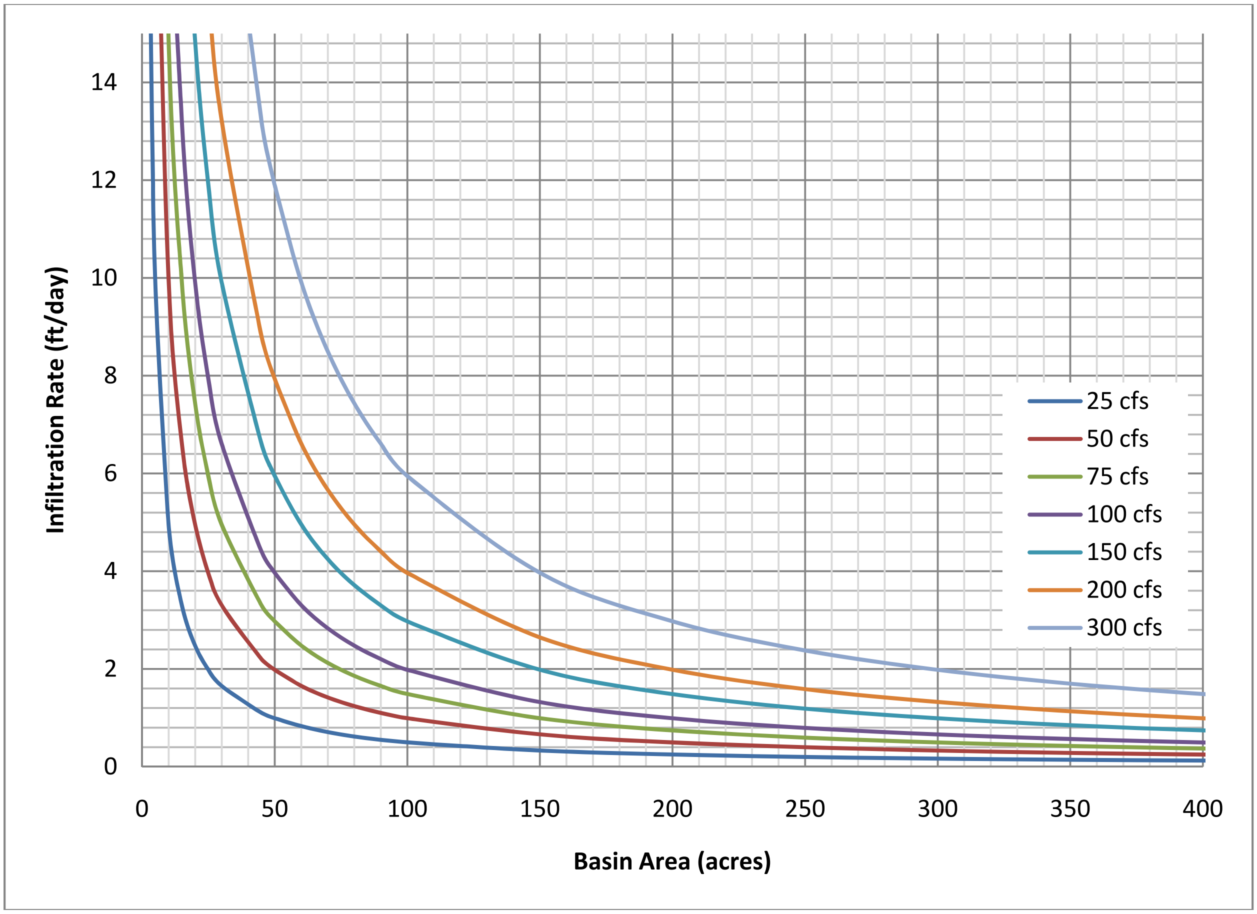

Figure 69. Basin area for given flow and infiltration rate.

Using the 47.4 acres as the infiltration basin size, the effects of three different parameters were investigated:

1) seven direct infiltration rates (25, 50, 75, 100, 150, 200, and 300 ft^3/s),

2) a range of saturated hydraulic conductivities (125 to 32,808 ft/day), and

3) five length to width (L/W) surface area ratios (1 to 10,000).

The tests concerning both the infiltration rates and the L/W ratios used the saturated hydraulic conductivity value surrounding the NR3 site. The test for the range of hydraulic conductivities values used was determined from the calibrated MODFLOW 2000 SVRP aquifer model (Hsieh et al. 2007). It is noted that the range of saturated hydraulic conductivities determined by Hsieh et al. (2007) was 5-22,100 ft/day; however hydraulic conductivities less that 94 ft/day were not considered indicative of the infiltration area. At the other end of the K spectrum, the 32,808 ft/day value was used to help ensure that modeling results out to the 22,100 ft/day rate were accurate. With hydraulic conductivities well beyond the normal range tested, an additional point was determined to be necessary. Modeling results are provided in more detail below.

The L/W ratio of the infiltration gallery is important because it impacts the ability of the water to spread out as it travels through the vadose zone. The more the water can spread out, the lower the moisture content and, subsequently, the lower the unsaturated hydraulic conductivity. Lower K values increase the travel time through the soil column. Table 13 shows the dimensions of the five different L/W ratios used to create the necessary infiltration surface area.

Table 13. Dimensions for Length/Width ratios.

Length	Width	L/W	Length	Width
ft	ft	[ft/ft]	Miles	Miles
1,437.4	1437.4	1	0.272	0.272
4,545.5	454.5	10	0.861	0.086
14,374.0	143.7	100	2.722	0.027
45,454.6	45.5	1000	8.609	0.009
143,740.0	14.4	10000	27.223	0.003

To complete the modeling the infiltration basin width was divided in half assuming an axis of symmetry existed in the middle of the basin. This reduced the number of elements of the model needed to represent the infiltration basin which decreased the time of each run. With the longest run taking over 15 hours to process, the reduction in size was necessary. Subsequently, HYDRUS2D was used to further reduce the size from a fully three dimensional version to a vertical slice of the aquifer cross-section. These two reductions in size allowed for a greater degree of grid refinement then would have otherwise been possible.

The HYDRUS2D model was created with a length of 1,640 ft (500 m) which is about twice the L/W = 1 half basin dimension used. This ensured that the boundary conditions did not adversely impact the solutions. Metric dimensions of the two-dimensional cross section as well as the depth to water table are shown in Figure 70. The infiltration basin began at the left boundary (x = 0) and extended to half width edge located at the far upper right boundary (e.g., x = L = 500 m). The height of the model was 414 ft (126 m) with z = 0 ft at the bottom and z = 414 ft (126 m) at the top. The water table was positioned at z = 177 ft (54 m), which gives an unsaturated zone depth of 237 ft (72 m). Since the bottom of the infiltration basin was 6.6 ft (2 m) below the ground surface, the water from the basin would travel through 230.4 ft (70 m) of unsaturated material. Figure 70 also shows the initial conditions from a linear distribution to constant value. Since the HYDRUS2D model was developed in metric units, the dimensions on Figure 70, Figure 71, and Figure 72 are in meters. These figures also show the setup for Ksat = 12,080 ft/day, injection rate = 1 ft/day, and L/W = 100. This scenario was not chosen because it is the preferred alternative, but rather because it was an average representation of the scenario setups tested.

As illustrated in Figure 71, the boundary conditions were set using a constant head boundary along the left side of the figure (x = 0) which linear transitions from a positive pressure head of 177 ft (54 m) at z = 0 ft to a pressure head of -16.4 ft (-5 m) at z = 193.4 ft (59 m). Above z = 193.4 ft (59 m), the pressure head remains constant at – 16.4 ft (-5 m) up to the ground surface. The infiltration basin is located in the upper right corner of the model domain with the dimension from Table 13 shown for a L/W ratio of 100. The remaining boundaries were assigned as no flow boundaries including the axis of symmetry boundary located on the right hand edge (x = 500 m).

The initial conditions follow the constant head boundary from the aquifer bottom up to the negative 16.4 feet of pressure head (at z = 193.4 ft). From there to the top of the model (z = 414 ft), the pressure head was set at -16.4 ft. This value was allowed to change during the simulation in response to recharge. As such, it was deemed an acceptable starting point for the model. Other initial conditions that allowed the pressure head above z = 193.4 ft to vary linearly to the ground surface where also tested with minimal influence on the results. Figure 72 shows the layout of the initial conditions in the model.

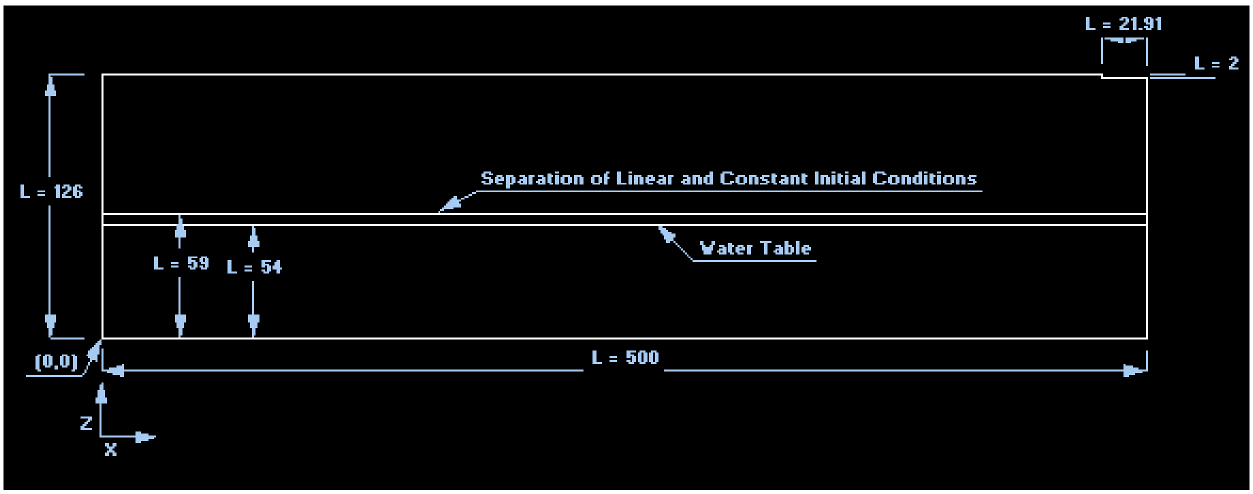

Figure 70. Infiltration basin layout used in HYDRUS2D (lengths given in meters).

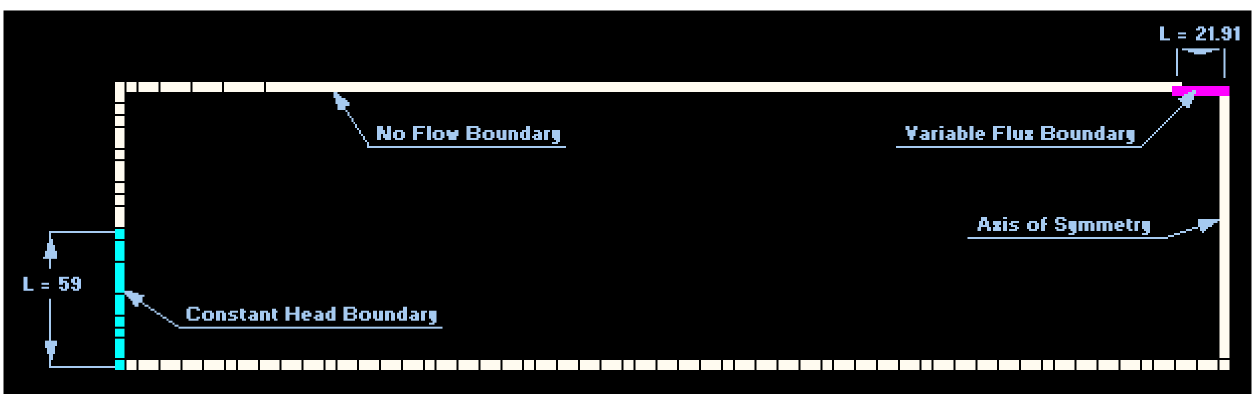

Figure 71. Boundary conditions used in HYDRUS2D (lengths given in meters).

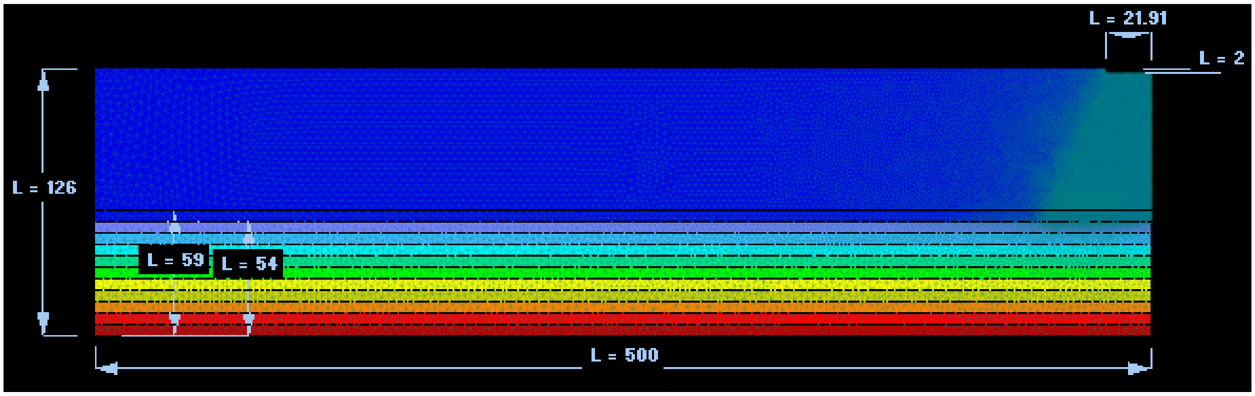

Figure 72. Initial conditions used in HYDRUS2D (lengths in meters).

6.2.2.2 Infiltration Basin Results

The key to understanding our attempt to limit recharge to 1 ft/day lies in the characteristics of soils during the wetting phase. Figure 73 shows the soil hydraulic model (hydraulic conductivity as a function of soil moisture) for the 12,080 ft/day saturated hydraulic conductivity found near the NR3 location. As illustrated in Figure 73, the first half of the curve (theta < 0.20) has very little change in hydraulic conductivity and the K values are relatively small. This causes relatively small amounts of water (e.g., normal precipitation) to move very slowly in the unsaturated zone. As the saturation increases (e.g., theta > 0.20) the K values increase rapidly approaching the saturated values determined in the Hseih et al. (2007) model. This causes the flow to move very quickly and thus the time it takes for water in the infiltration facility to reach the water table decreases significantly. Therefore, to achieve longer lag times a relatively dry soil condition is needed which limits the allowable application rate.

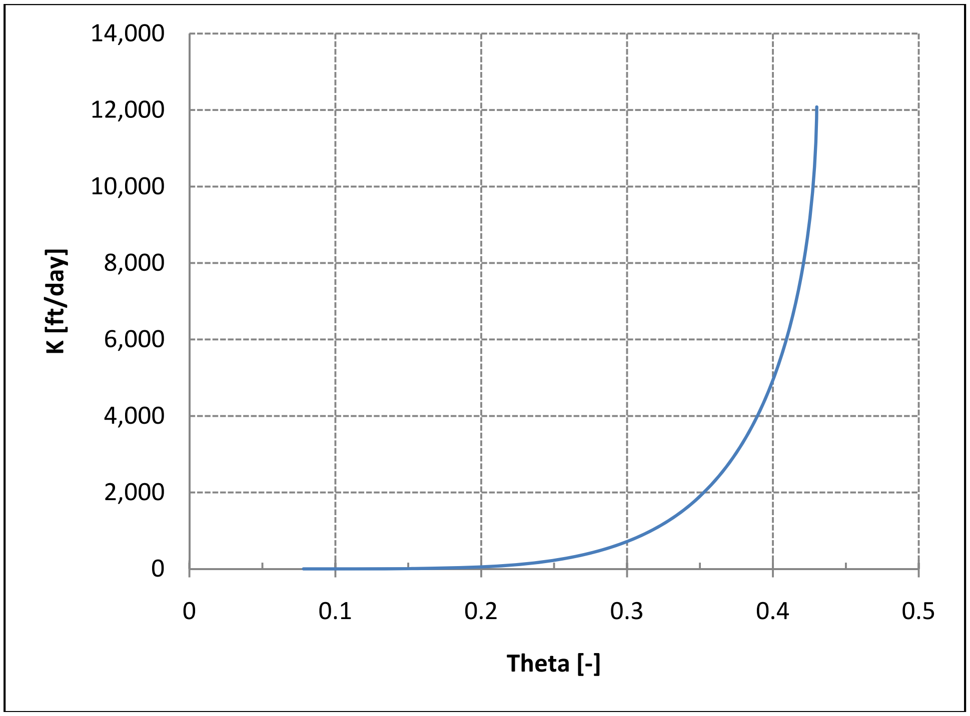

Figure 73. Hydraulic model for soil near NR3 location.

Figure 74 demonstrates how the lag time is impacted by the rate of infiltration water applied to the recharge area. To maintain a significant lag time to the water table (20-30 days), the application rate needs to be approximately 0.75 ft of water per day. In examining Figure 69 in conjunction with Figure 74, the flow needed to achieve a significant lag time (≈ 20 days) is less than about 1 ft/day and the size of the basin needed to accomplish this becomes very large as injection rates increase. For example, at 1 ft/day it would take approximately 200 acres of land to be able to infiltrate 100 ft^3/s. Even the dimensions for the smallest infiltration rate (25 ft^3/s) area are reasonably large (over 47 acres) while the land area needed to accommodate the largest infiltration rate (300 ft^3/s) is much larger yet.

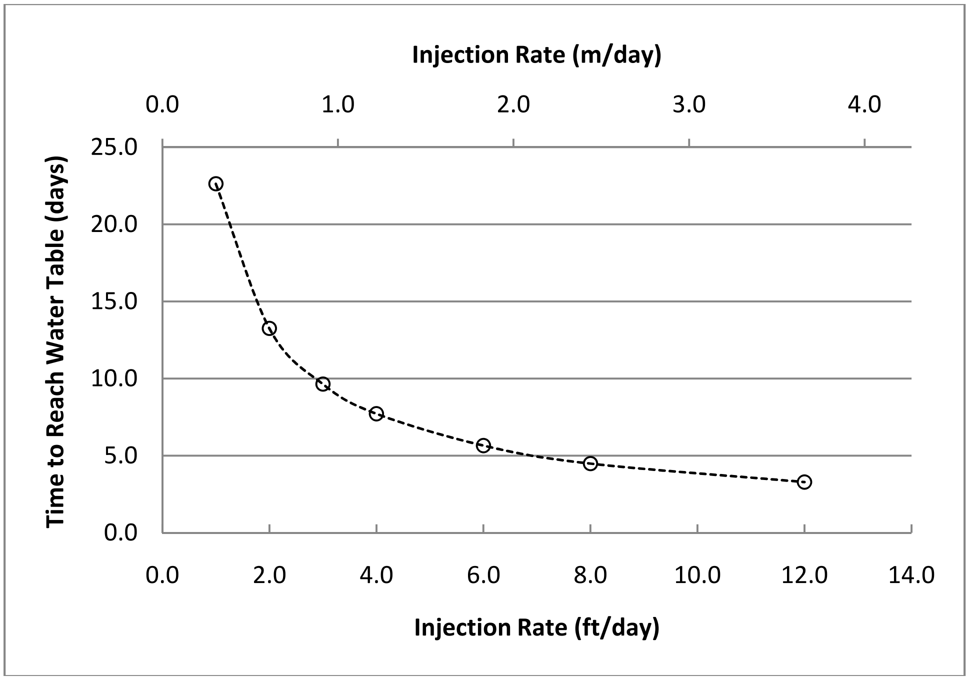

Figure 74. Lag time to reach the water table as a function of injection rate.

From Figure 75 the configuration of the basin dimensions is also important. As the basin move away from a square (L/W = 1) towards an increasingly narrow and long rectangle (L/W = 10,000) the soil beneath the basin becomes less saturated and accordingly the time to reach the water table increases. As indicated it would take a L/W ratio of 10,000 to produce a lag time of 23 days. Table 13 shows that the length of the L/W = 10,000 scenario would be about 27.2 miles, which is almost the distance between the city Spokane and the city Coeur d' Alene (29 miles). The sheer size of this approach would appear to limit the feasibility of infiltration basins as an option for increasing the utility of the injection pipeline.

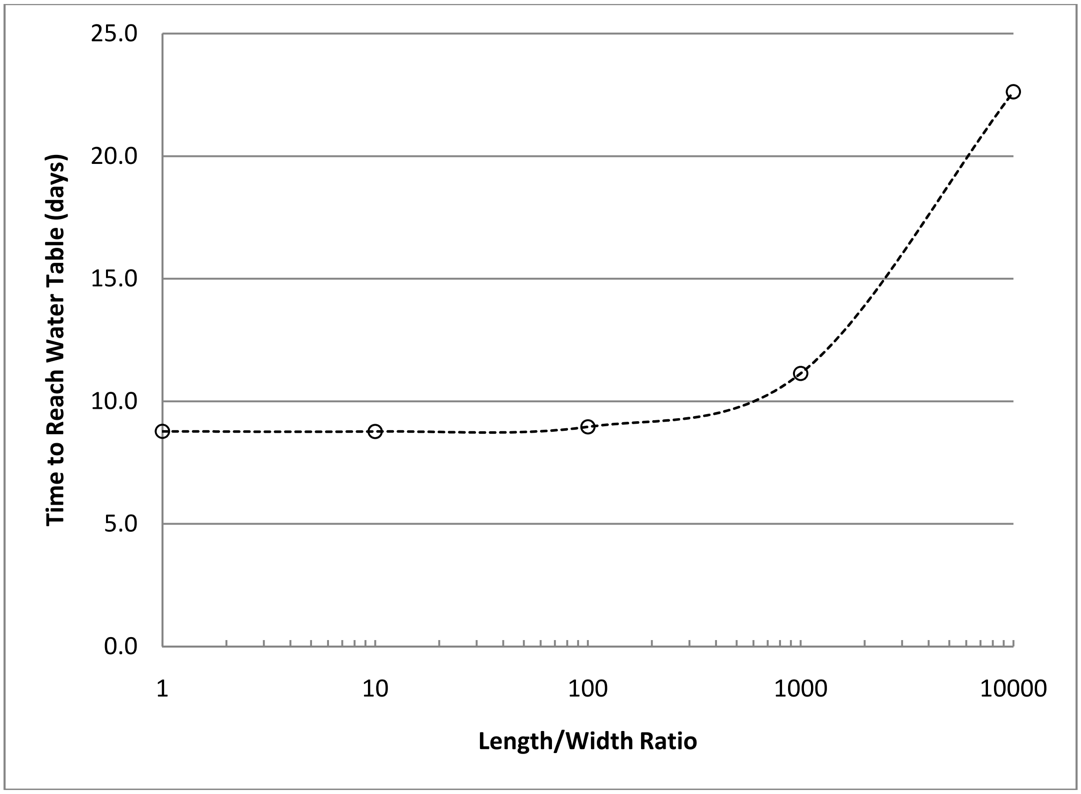

Figure 75. Lag time to reach water table as a function of recharge length to width ratio.

Figure 76 clearly shows the significant sensitivity of the time lag to the saturated hydraulic conductivity. For the range of hydraulic conductivities tested by the HYDRUS2D model, the flows took from approximately 19 days to greater than 120 days to travel through the unsaturated zone down to the water table. As was previously shown in Table 12, the lowest saturated hydraulic conductivity that was tested for the direct injection was 94 ft/day. This is the area referred to as the "Chilco Channel" northeastern region of the SVRP and is shown in light green in Figure 77. From Figure 76 this lower K_{sat} value would potentially allow a significant delay in time to travel through the unsaturated zone and thus enable larger quantities of water to be infiltrated while still allowing for the goal of at least a 30 day lag. However, the geological data in the Chilco Channel area is presently not well understood due to a lack of geologic information and extreme subsurface complexity. Due to this uncertainty, it was decided not to proceed with additional analysis of this area. If this ASR Project merits future expansion, this area could be further explored as a potential option. For the current study, however, the overall conclusion of this analysis is that the large areas needed to achieve the infiltration rates are unpractical for the SVRP ASR project. Therefore, no further analyses were conducted with respect to augmenting storage volumes with possible infiltration basin locations.

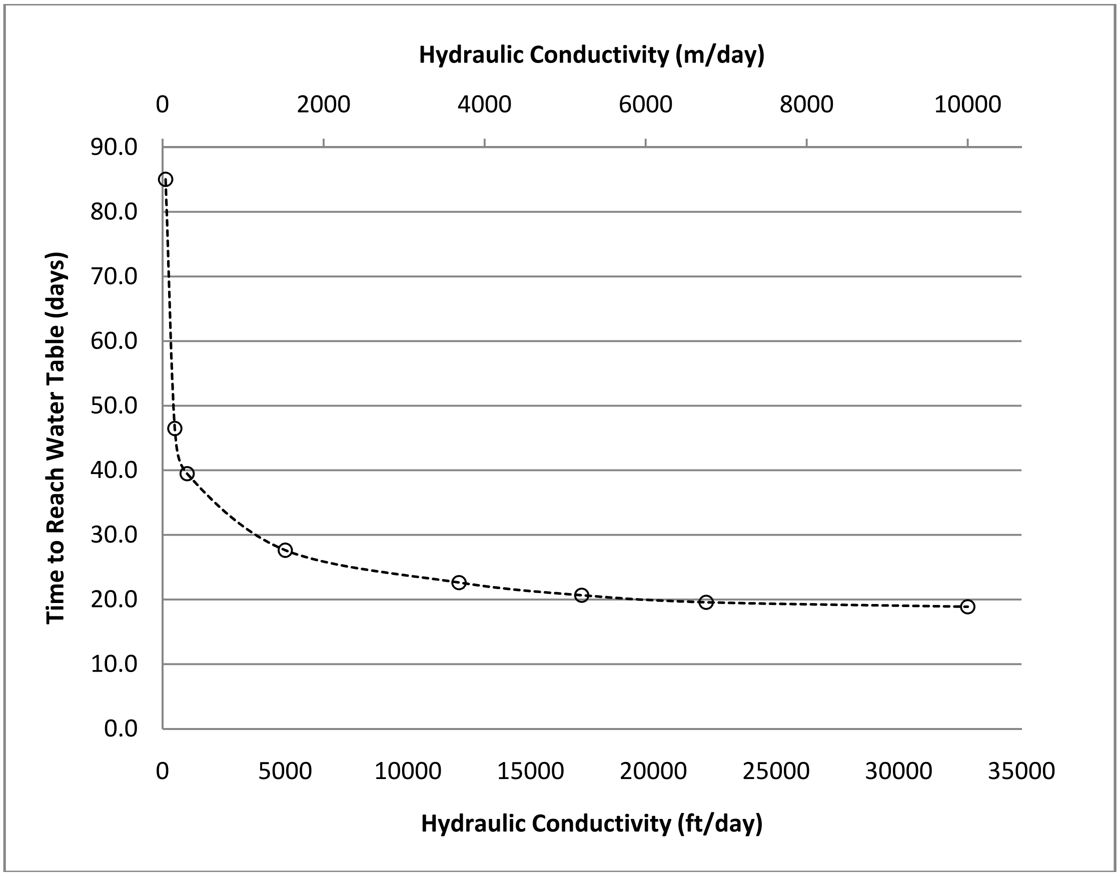

Figure 76. Hydraulic conductivity sensitivity analysis.

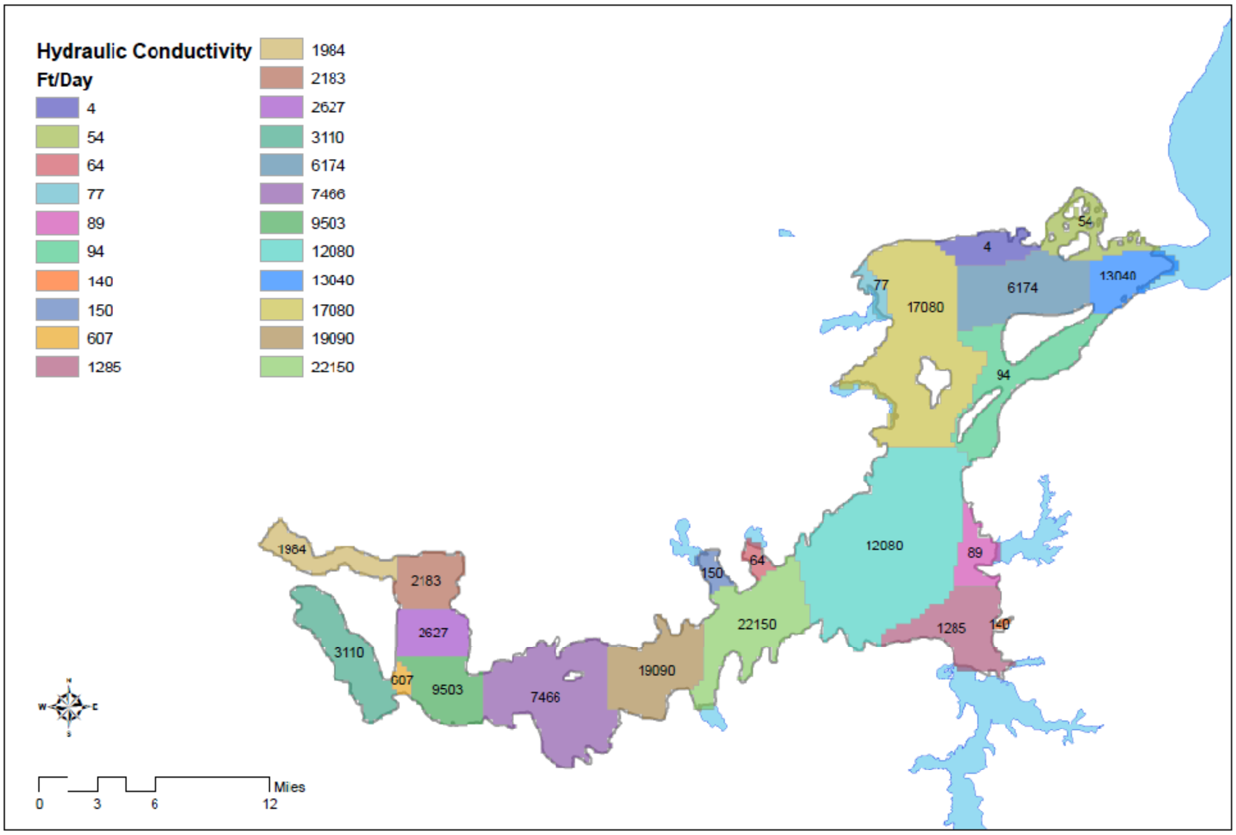

Figure 77. Spatial distribution of saturated hydraulic conductivities in the SVRP study area (from Hseih et al. 2007).

6.3 Water Distribution

After water is extracted from the well fields or surface diversion it must be transported to the injection site. In the Visual MODFLOW-2009 model, there is no direct linkage between extraction and injection, so the route in which the water travels is not an issue. However, in actuality, a large percent of the costs associated with the ASR project will be linked to the distribution route. The analyses conducted to support this phase of the project are described in the following sections.

6.3.1 Spokane Collection and Distribution System

Despite the fact that the groundwater model revealed direct surface water withdrawal is the only viable option for taking water from the Spokane River, shorter distribution routes provided an impetus for further examining this option. The extraction point would be near the intersection of East Wellesley and North Chase Roads approximately 0.5 miles west of the Washington-Idaho state line. After treatment, the water would be pumped east on Wellesley to the North Idaho Road intersection, then north along that road until the route crosses either the Southern Railroad (SR), Power Lines (PL), or Northern Railroad (NR) right-of-ways. All three right-of-way routes cross North Idaho Road at different locations. A summary of the route lengths and ground surface elevations is provided in Table 14. Figure 78 illustrates the lower portions of both the southern railroad (SR) line and the power line (PL) routes. When accessing these sites from Washington, only the lower sites (i.e., 3-5) are pertinent. Because the ground water model results indicated sufficient lag times from these locations, there is no need to go through the added expense of pumping the water further uphill or longer distances.

The groundwater modeling results indicated that there were essentially no differences in return flow quantities or timing with respect to the SR5, PL5, and NR5 sites so cost and access are the only governing factors. As indicated in Table 14, the total distance from the diversion point to SR5 is approximately 19,000 feet (18,988 ft). SR4 is an additional 23,758 ft from SR5. Similarly, the total distance from the diversion point to PL5 is 18,658 ft and to NR5 is 25,328 ft. Assuming equal access to all three routes, these seems to be no reason to pump the water additional lengths in order to use the NR right-of-way. The distances to SR5 versus PL5 and SR4 versus PL4 are fairly comparable in terms of length. Additionally, in terms of elevation or depth to the aquifer, the comparison is close but the overall advantage seems to favor the power line route with PL5 slightly lower than SR5 and PL4 being slightly lower than SR4. This reduces the amount of pumping head required to deliver water to the site. Furthermore, the distances from the ground surface to the aquifer for PL4 and PL5 are 184.0 ft and 129.8 ft, respectively, versus 225.5 ft and 160.1 ft for SR4 and SR5, respectively. For these reasons, the distribution analysis focused on the Power Line route as the preferred alternative.

Table 14. Summary of Spokane River to southern railroad (SR) pipeline properties.

Line Segment	Length (feet)	Elevation (feet)
Diversion Facility	---	2,034
Treatment to Chase/Wellesley	850	2,093
Chase/Wellesley to North Idaho	1,325	2,094
North Idaho to SR Crossing	1,960	2,094
SR Crossing to SR5	14,853	2,144
SR5 to SR4	23,758	2,225
SR4 to SR3	24,312	2,304
SR Crossing to Power Lines	6,490	2,101
PL to PL5	8,033	2,114
PL5 to PL4	24,197	2,180
Power Lines to NR Crossing	3,410	2,112
NR Crossing to NR5	11,293	2,136

The US Environmental Protection Agency's EPANET computer model was used to investigate pumping requirements as functions of discharge rates and pipeline diameters. Specifically, EPANET version 2.0 Build 2.00.12 was downloaded and installed for these analyses. This publically available model is capable of simulating head losses in user defined distribution systems and has been extensively used throughout the world. In this application, losses due to friction were assumed to far exceed localized losses due to valves, joints, tees, and elbows (collectively referred to as minor losses although this should not be meant to infer that they are small).

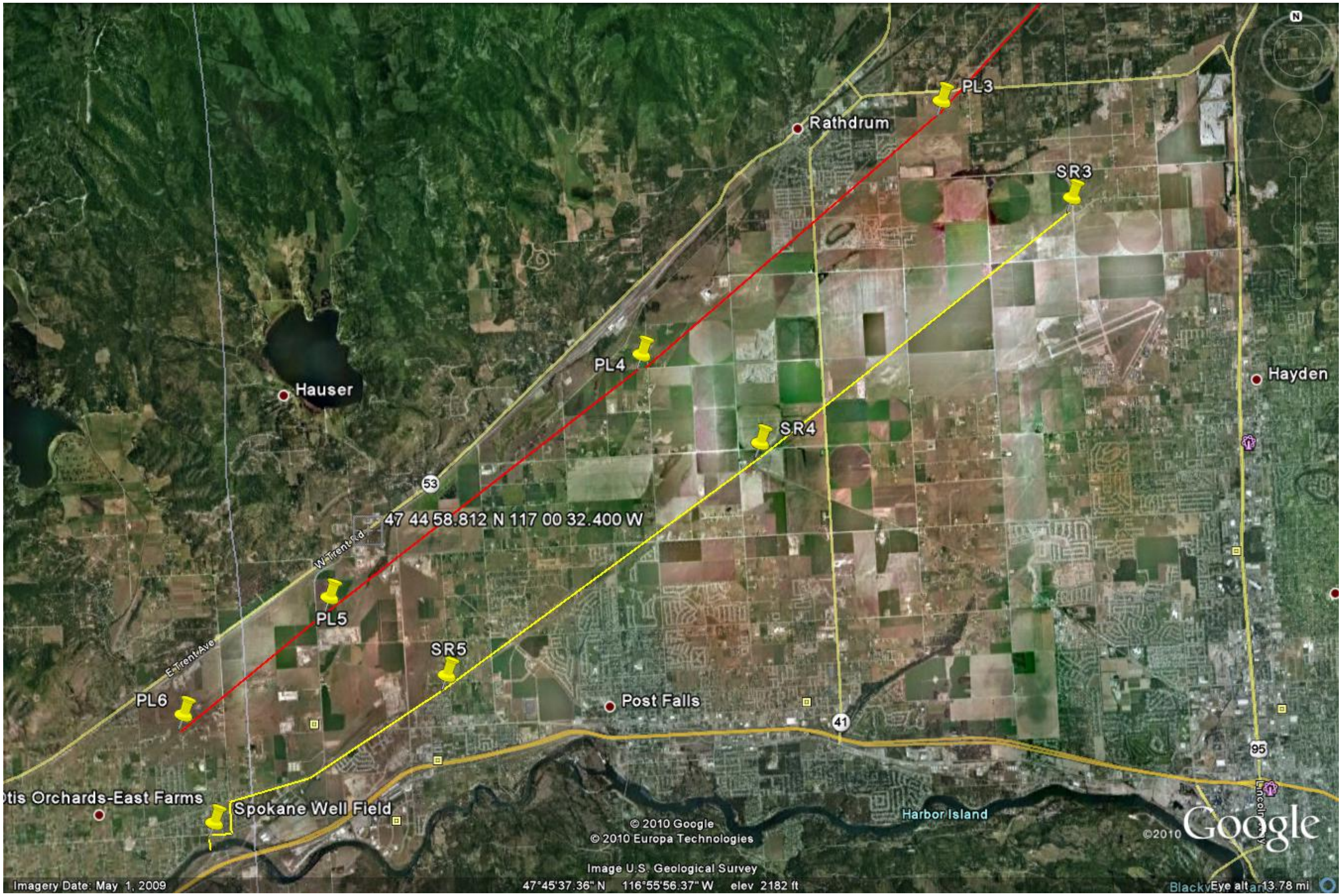

Figure 78. Lower portions of southern railroad (SR) and power line (PL) routes.

The Hazen-Williams friction loss formula was used to estimate head loss throughout the system. This equation can be written in English units as:

$$V = 1.318 * C * R_h^{0.63} S^{0.54} \quad (2)$$

where V is flow velocity (discharge/cross-sectional pipe area) [ft/s], C is the Hazen-Williams coefficient, R_h is the hydraulic radius (inside diameter/4 for circular full pipe flow) [ft], and S is the slope of the energy grade line (head loss/pipe length) [ft/ft].

In this application, the discharge and pipe area (and thus the velocity), the C factor, and the hydraulic radius are all known variables. Substituting head loss divided by pipe length (known parameter) in the S term in Eq. (2) leaves only the head loss term as unknown. EPANET solves this equation for each pipe segment along the flow path.

The value of C depends on the pipeline material and the age of the system. It was assumed that this pipeline would be constructed of cast iron. As indicated in Table 15, published C values range from 64 to 130. An average C factor of 100 was used in this effort although a sensitivity analysis was conducted to evaluate the consequences of this assumption in the Pend Oreille section of the study.

Table 15. Typical values used for Hazen-Williams friction coefficient.

C values for cast iron pipe				
New	10 years old	20 years old	30 years old	40 years old
130	107-113	89-100	75-90	64-83

The EPANET model was run using a variety of flows ranging from 25 to 300 ft^3/s, three injection field sites (PL3, PL4, and PL5), and three injection well diameters (12, 18, and 24 inches). Results of the simulation are presented and discussed in Section 6.3.2.

6.3.2 Pend Oreille Collection and Distribution System

The Lake Pend Oreille collection and distribution system consists of an extraction well field near the lake, a pipeline following one of three potential routes down the SVRP towards Washington, a short turnout at each potential injection location, and a distribution injection well field. The extraction well field would be situated approximately 29,000 feet east of Athol, Idaho although several alternative sites could be similarly developed without material consequences to the overall feasibility of the project. The aquifer depth is approximately 346.75 feet at this location although planned lift would be 365 feet to account for possible drawdown.

The three alternative routes utilizing existing right-of-ways were examined in the Visual MODFLOW-2009 model. The goal of using existing pathways stemmed from our desire to minimize cost and disruption of private property. The routes were along the railroad line across the northern section of the aquifer (NR), a similar railroad line across the southern portion (SR), and the power line (PL) in between the two railroad lines. These locations were shown in Figure 78. The exact route therefore would be more a function of cost and convenience rather than hydraulic requirements. The final cost of the route would depend on the negotiated easement fee.

As explained in the modeling summary, numerous alternative injection locations were examined; eight on the NR line, eight on the SR line, and six along the PL route. The regional Visual MODFLOW-2009 is a block-centered finite difference model with a 1,320 ft x 1,320 ft grid size and is fairly insensitive to exact injection well location. Nevertheless, we elected to use a well field approach for putting the water back into the aquifer. A short 100 foot tee off the main trunk line was used to distribute water to the injection wells. Each injection location was represented by as few as one and as many as twelve wells depending on the flow rate. These wells were assumed to extend from near the surface down to 25 feet below the aquifer surface thus injecting water into the saturated layer. Similar results would likely be obtained if the wells did not penetrate the saturated zone but this assumption avoided the need to consider unsaturated flow.

As described in the previous section, the US Environmental Protection Agency's EPANET computer model was used to investigate pumping requirements as functions of discharge rates and pipeline diameters. In this application, losses due to friction were again assumed to far exceed localized losses due to valves, joints, tees, and elbows (collectively referred to as minor losses although this should not be meant to infer that they are small).

Not all of the alternative injection sites turned out to be feasible in terms of lag times to the river. Generally, any locations west of the WA/ID state line produced lag times that were too short to enhance August/September streamflows. On the NR line, only the sites identified as NR1, NR2, NR3, NR4, and NR5 were truly acceptable. These locations are shown in Figure 79.

NR6 was included in the EPANET analysis as it was located in Washington just west of the state line even though the lag times were very small. The distribution system for this route is summarized in Table 16. Length and elevation data were obtained from Google Earth. The lengths shown in the table represent incremental lengths and are thus additive in order to ascertain total length (i.e., the distance from the well field to NR2 would be 47,315+22,385 = 69,700 ft (13.2 miles)).

Table 16. Summary of northern railroad (NR) pipeline properties.

Line Segment	Length (feet)	Elevation (feet)
Well Field	---	2,390
Well Field to NR1	47,315	2,371
NR1 to NR2	22,385	2,326
NR2 to NR3	23,563	2,231
NR3 to NR4	23,990	2,155
NR4 to NR5	15,601	2,136
NR5 to NR6	9,396	2,125

A screen capture of the EPANET working schematic is shown in Figure 80. It is important to note that the model is not to scale. For instance, each of the turnouts represents a short 100 ft section of pipe but has been drawn exaggerated on the diagram to facilitate graphical representation. Likewise, the injection well fields are drawn as series of circular wells. The model calculates head loss based on user-specified lengths and elevations, and not the diagram. A fixed-head reservoir with an unlimited water supply was used at the upstream end of the pipeline with a pump capable of providing sufficient head to deliver the water to the proper locations. Demands were placed at end nodes sequentially as appropriate to match injections rates with the general goal of 25 ft^3/s per well. The Hazen-Williams friction loss formula was used to estimate head loss throughout the system as was explained in the previous section.

As a reminder, the discharge and pipe area (and thus the velocity), the C factor, and the hydraulic radius are all known variables. Substituting head loss divided by pipe length (known parameter) in for the S term in Eq. (2) leaves only the head loss term as unknown. EPANET solves this equation for each pipe segment along the flow path.

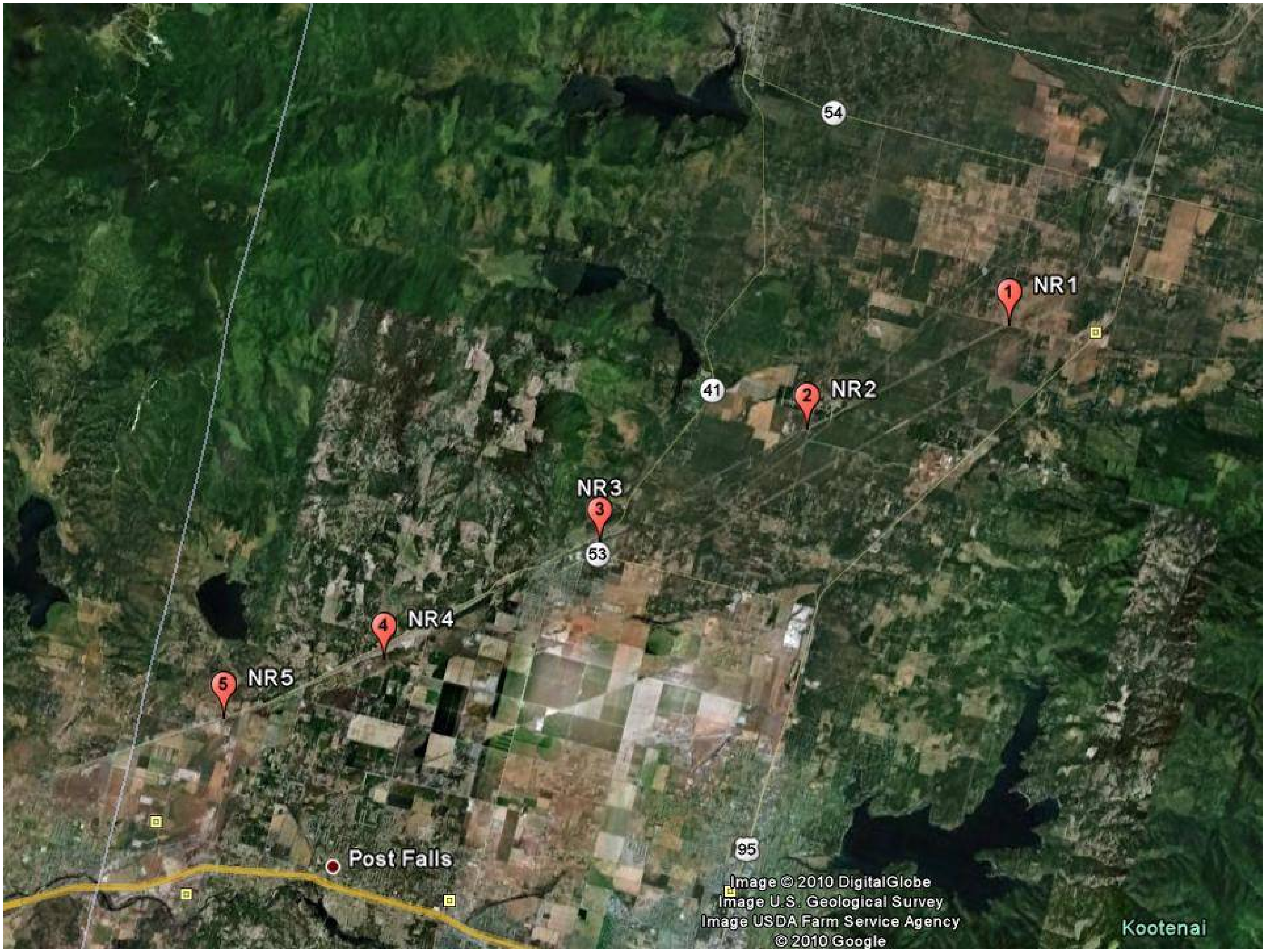

Figure 79. Potential injection well locations along northern railroad (NR) line

Not to scale

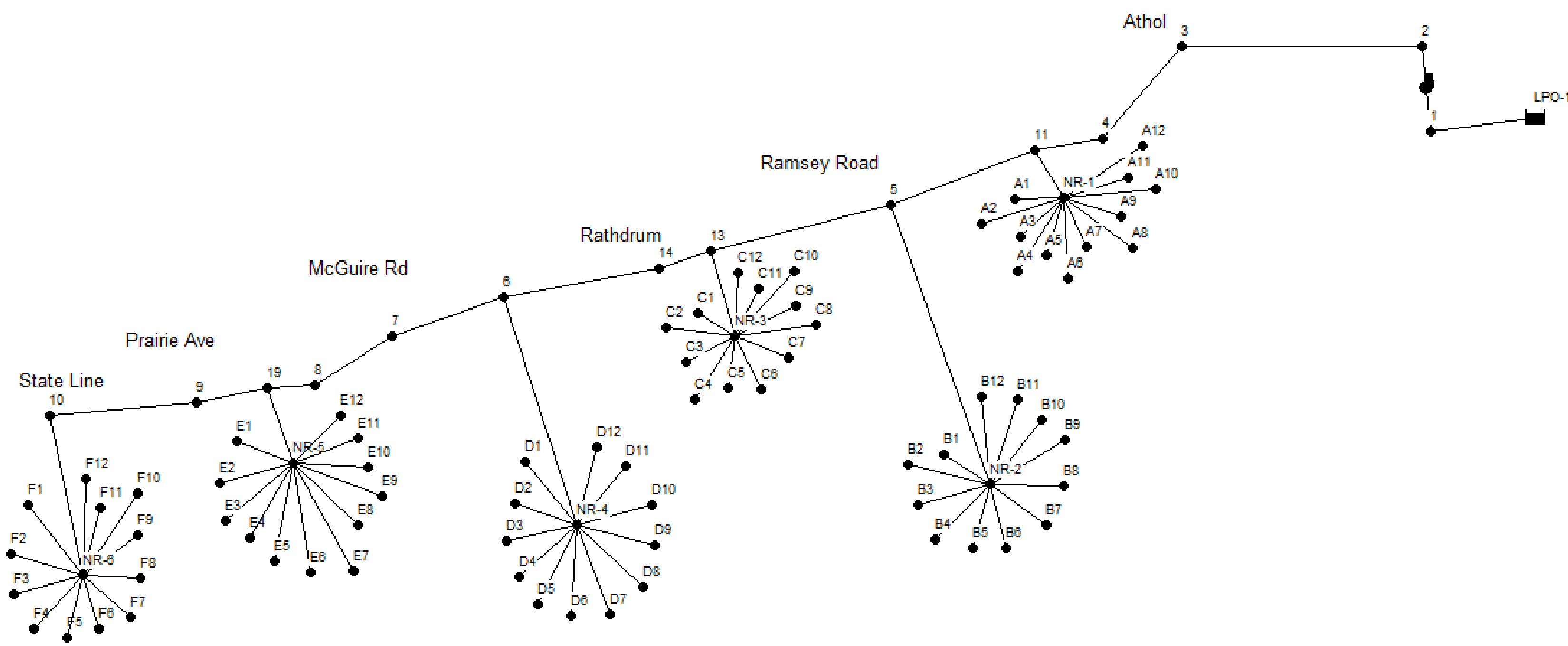

Figure 80. Schematic of possible northern railroad line distribution system.

6.3.3 Summary of EPANET results

<u>PL line from Spokane River Diversion</u>

Three injection well field scenarios (PL3, PL4, and PL5) were examined in association with the Spokane River surface water diversions. PL3 is the furthest point along the Power Line right-of-way investigated in this analysis. The total distance is 66,884 ft (12.7 miles) long according to the route mapped out using Google Earth. The termination point of the pipeline is just east of Rathdrum, Idaho.

Results of the flow, pipe diameter, and well diameter combinations for the PL3 site are provided in Table 17. The shaded boxes in the table represent regions thought to be infeasible because of either excessive head requirements or low flow to diameter (cost) ratios. It was assumed that injection wells were limited to 25 ft^3/s so 100 ft^3/s would require 4 identical wells.

Table 17. Pipe and injection well friction losses from Spokane River to PL3 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to PL3 Location (ft^3/s)						
		25	50	75	100	150	200	300
24	12	915.9						
	18	842.2						
	24	833.2						
30	12	365.6	1,096.4					
	18	291.9	1,022.7					
	24	282.9	1,013.8					
36	12	200.8	501.5	966.9				
	18	127.1	427.8	893.2				
	24	118.1	418.8	884.2	1,504.2			
48	12			302.6	455.4	869.1		
	18			228.9	381.6	795.4	1346.7	
	24		105.4	220.0	372.7	786.4	1337.8	
60	12					349.8	535.8	1,039.5
	18				136.6	276.1	462.0	965.8
	24				127.6	267.2		
72	12							478.1
	18							404.4
	24					111.6	188.2	395.4

PL4 is the midpoint along the Power Line right-of-way investigated in this analysis. The total distance is 42,855 ft (8.1 miles) long according to the route mapped out using Google Earth. For the purpose of this analysis, only friction losses were included in the EPANET model. Locations of valves, bends, and other minor losses were excluded but would require additional head to overcome. Also, additional pumping requirements for processes inside a water treatment plant were not included.

Table 18. Pipe and injection well friction losses from Spokane River to PL4 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to PL4 Location (ft^3/s)						
		25	50	75	100	150	200	300
24	12	608.3						
	18	543.0						
	24	535.0						
30	12	255.4	724.0					
	18	190.1	658.8					
	24	182.2	650.8					
36	12	149.7	342.5	641.0	1,038.6			
	18	84.4	277.2	575.7	973.3			
	24	73.9	269.3	567.7	965.4			
48	12		141.5	215.0	312.9			
	18		76.2	149.7	247.6	513.0	866.5	
	24		68.3	141.8	239.7	505.0	858.6	
60	12				155.8	245.2	364.5	687.5
	18				90.5	180.0	299.2	622.2
	24				80.0	172.0	291.3	614.3
72	12					145.5	194.6	327.5
	18					80.2	129.3	262.2
	24					72.3	121.4	254.3

PL5 is the closest point along the Power Line right-of-way investigated in this analysis. The total distance is 18,658 ft (3.5 miles) long according to the route mapped out using Google Earth.

Table 19. Pipe and injection well friction losses from Spokane River to PL5 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to PL5 Location (ft^3/s) 25	50	75	100	150	200	300
24	12	288.7						
	18	240.3						
	24	234.4						
30	12	134.6	339.3					
	18	86.2	290.9					
	24	80.3	285.0					
36	12	88.5						
	18	40.1	127.3	254.6	428.3			
	24	34.2						
48	12						430.0	
	18		36.5	68.6	111.4	227.2	381.6	
	24		30.6	62.7	105.5	221.3	375.7	
60	12				91.2	130.2	182.3	323.3
	18				42.7	81.8	133.9	274.9
	24				36.8	75.9	128.0	269.0
72	12					86.9	108.5	167.0
	18					38.5	60.1	118.5
	24					32.6	54.2	112.7

The results presented in these three tables illustrate that pumping water from the Spokane River diversion site would likely be possible excluding water quality concerns. Treatment costs are shown below. The results also indicate that while head requirements drop considerably by changing the well diameter of 12 inches to a well diameter of 18 inches, little is gained by expanding the well diameter further from 18 to 24 inches.

NR line from Pend Oreille Area

Seven scenarios, covering a range of flows from 25 to 300 ft^3/s, were evaluated using various pipe and injection well configurations in the EPANET model. The actual flows used are summarized in Table 20 along with typical conversions. Friction losses for various pipe diameters and flow rates are shown in the tables below. These head losses include the distribution system losses and the injection well losses. The injection well losses can be substantial for smaller diameter wells (e.g., 12 inch wells). These losses need to be added to the lift requirement (365 feet) to determine the majority of the overall pumping head requirements for the system. The remaining friction loss that needs to be added is any loss along the lift line(s) and through the well screens at the injection sites. This depends on the design of the extraction and injection well fields. In particular, head losses consist of entrance losses in the screen and filter plus losses arising from flow in the screen and riser. These losses are estimated separately later in this section.

Table 21 summarizes the friction head losses from the extraction well field to the bottom of the injection well at the NR1 location. For example, pumping 25 ft^3/s through an 18-inch water main and into a 12-inch injection well requires approximately 2523 feet of head. Three injection well diameters were used for comparison. As indicated, there is generally a sharp break between the 12 inch and 18 inch diameter wells particularly in comparison to the differences between 18 and 24 inch diameters. Shaded areas in the table indicate regions that were not specifically examined because they would likely be cost prohibitive. For example, using an 18 inch distribution line along the NR1 route with a 24 inch injection well required 2391 ft of head. For this portion of the head loss (excluding lift and friction in the extraction wells), this works out to approximately $225 per acre-ft of injected water using $0.07 per kilowatt, a pump efficiency of 0.85, and a motor efficiency of 0.9. Trying to force 50 ft^3/s through the same size water main would significantly increase head loss and thus pumping costs. In terms of peak return to the river, keeping in mind that the river only sees a fraction of this amount in the low flow months, the cost per ft^3/s would be even higher thus generated concerns over some smaller diameter-high flow options. Likewise, some large diameter-low flow scenarios were excluded due to costs associated with construction. Sections of the table marked with “na” indicate that negative pressures were generated somewhere along the pipeline even with 4000-5000 feet of initial head at the top of the extraction well field.

Table 20. Candidate injection well flows

Flow Rate (ft^3/s)	Equivalent Rates		
	gallons per minute	acre-feet per day	m^3/s
25	11,220	49.6	0.74
50	22,440	99.2	1.48
75	33,660	148.8	2.25
100	44,880	198.3	2.97
150	67,320	297.5	4.45
200	89,760	396.7	5.94
300	134,640	595.0	8.90

Table 21. Summary of pipe and injection well friction losses at NR1 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to NR1 Location (ft^3/s)						
		25	50	75	100	150	200	300
18	12	2523.2						
	18	2404.5						
	24	2391.2						
24	12	724.5	na					
	18	606.7	na					
	24	592.4	2126.4					
30	12	335.0						
	18	217.2	734.5	1535.2	2602.1			
	24	202.9						
36	12	218.3	431.2	760.6				
	18	100.5	313.4	642.8				
	24	86.2	299.1	628.5	1067.5	2256.7		
48	12	156.8	209.2	290.4	398.5			
	18	39.1	91.5	172.6	280.7			
	24	24.8	77.2	158.3	266.4	559.3	949.6	2006.8
60	12	143.5	161.2	188.6	225.0	323.8	455.4	812.0
	18	25.8	43.4	70.8	107.2	206.0	337.6	694.2
	24	11.4	29.1	56.5	92.9	191.7	323.3	679.9
72	12					213.7	267.9	414.6
	18					95.9	150.1	296.8
	24					81.6	135.8	282.5
	30					78.5	132.7	279.4

The same basic set of options was run using EPANET for the NR2 location. Because of increased flow distances, head losses were expected to increase. Based on this, certain areas were discontinued. The summary of results is shown in Table 22. As indicated in the 72 inch diameter line, the break in head requirements is once again between the 12 and 18 inch diameter injection wells.

Table 22. Summary of pipe and injection well friction losses at NR2 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to NR2 Location (ft^3/s)						
		25	50	75	100	150	200	300
18	12							
	18	3530.0						
	24							
24	12							
	18	882.1	3140.2					
	24	869.4	3127.5					
30	12							
	18	296.0	1070.2	2248.8				
	24							
36	12							
	18	136.9	450.2	935.2	1581.4	3332.0		
	24							
48	12							
	18	46.4	123.6	243.0	402.2	833.3	1407.8	na
	24							
60	12							
	18			93.1	146.8	292.2	486.0	1010.9
	24							
72	12					234.8	314.5	530.5
	18					130.2	209.9	425.8
	24					117.4	197.2	413.1

The same basic set of options was run using EPANET for the NR3 location just east of Rathdrum, Idaho. The summary of results is shown in Table 23. The 18 inch diameter main line was dropped from further consideration due to the high head loss estimate obtained previously in Table 22.

Table 23. Summary of pipe and injection well friction losses at NR3 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to NR3 Location (ft^3/s)						
		25	50	75	100	150	200	300
24	12	1253.1						
	18	1170.6						
	24	1160.6						
30	12	486.1						
	18	403.6						
	24	393.6	1412.2					
36	12	256.4	674.4	1324.1				
	18			1241.6				
	24	160.6	583.0	1231.6	2096.0			
48	12	135.3	238.5	398.3	611.1	1187.8		
	18	53.8	156.0	315.8	528.7			
	24	42.8	145.0	305.8	518.6	1095.3	1863.8	
60	12			197.8	269.6	464.0	723.2	
	18				187.1	381.6	640.8	1342.8
	24			105.3	177.1	371.6	630.7	1332.8
72	12					247.3	353.9	642.8
	18					164.8	271.5	560.3
	24					154.8	261.4	550.3

The relatively short injection well casing lengths means that the system is relatively insensitive to the amount of flow being discharged through each well. Although it was initially assumed that 25 ft^3/s could reasonably be discharged through each well, it may be possible to reduce the number of wells and increase the flow in each without significantly increasing the head loss. As an example, six wells at 50 ft^3/s (300 ft^3/s total) were used for NR3, 72 inch main line with 24 inch diameter wells. The head loss increased from 550.3 ft to 558.9 ft. Since Visual MODFLOW-2009 averages over a grid, potential localized problems cannot be adequately evaluated without further field investigations. Consequently, the more conservative 25 ft^3/s maximum was used throughout the analysis.

The same basic set of options was run using EPANET for the NR4 location just west of Rathdrum, Idaho. The summary of results is shown in Table 24.

Table 24. Summary of pipe and injection well friction losses at NR4 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to NR4 Location (ft^3/s)						
		25	50	75	100	150	200	300
24	12							
	18	1464.0						
	24	1454.7						
30	12	557.7						
	18	499.9	1780.3					
	24	492.9	1773.3					
36	12	269.0						
	18	211.2	738.0	1553.3				
	24	204.2	730.9	1546.3				
48	12	116.8	246.6	447.3	714.9	1439.8		
	18	59.0	188.8	389.6	657.1	1382.0		
	24	52.0	181.7	382.5	650.1	1375.0		
60	12			195.3	285.6	530.0	855.8	
	18			137.5	227.8	472.2	798.0	1680.5
	24			130.5	220.8	465.2	791.0	1673.5
72	12					257.6	391.6	754.7
	18					199.8	333.8	696.9
	24					192.8	326.8	689.9

The summary of results for NR5 is shown in Table 25. Because the high head loss in the 24 inch NR4 main line was considered excessive and the loss would increase with distance to the NR5 site, this option was dropped from further consideration beginning in Table 25.

Table 25. Summary of pipe and injection well friction losses at NR5 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to NR5 Location (ft^3/s)						
		25	50	75	100	150	200	300
30	12	617.8						
	18	564.4						
	24	557.9	2068.4					
36	12	290.7	887.5	1811.2				
	18	237.3						
	24	231.4	827.6	1751.3				
48	12	118.4	265.3	492.8	796.0			
	18	64.9			742.5			
	24	58.4	205.4	432.9	736.0	1557.3		
60	12			207.3	309.5	586.5	955.6	
	18			153.9	256.1	533.1	902.1	1902.0
	24			147.4	249.6	526.6	895.7	1895.5
72	12					277.8	429.7	841.0
	18					224.4	376.2	787.6
	24					217.9	369.8	781.1

Table 26. Summary of pipe and injection well friction losses at NR6 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to NR6 Location (ft^3/s)						
		25	50	75	100	150	200	300
30	12	647.0						
	18	602.3						
	24	596.9	2150.0					
36	12	296.7	935.7					
	18	252.0	891.0					
	24	246.6	885.6	1874.6				
48	12							
	18	67.5	224.9	468.4	793.0	1672.3		
	24							
60	12							
	18			162.8	272.2	568.7	963.9	2034.4
	24							
72	12							
	18					238.2	400.8	841.2
	24							

Sensitivity Analysis for Hazen-Williams C Factor

To briefly examine the potential impact of the Hazen-Williams C coefficient, a sensitivity analysis was conducted using one of the 75 ft^3/s - NR4 scenarios. As illustrated previously in Table 24, the head loss was 137.5 feet when C was equal to 100, the distribution pipeline was 60 inches in diameter, and the injection well was 18 inches in diameter. If the value of C was reduced by 20% to 80 and all other parameters remained constant, the friction head loss would increase to 207.9 feet (+51.2%). Conversely, if the C value was increased by 20% to 120, the friction head loss would decrease to 98.1 feet (-28.7%). This nonlinear variation in head loss must be considered. Furthermore, since most studies indicate that C will decrease over time, the system needs to be flexible enough to handle additional pressure if needed in the future. We believe the C value of 100 is a reasonable average value for this analysis but that flexibility should be maintained in the design so that additional pumps can be added if needed in the future.

Multiple Injection Locations

Rather than have all the flows injected into the aquifer at the same location, the EPANET model was used to determine the head required to provide equal amounts of flow to five locations along the NR pipeline (NR1, NR2, NR3, NR4 and NR5). For example, a flow rate of 25 ft^3/s means that 125 ft^3/s will be injected in total but the flow rate will decrease incrementally at each injection location. For purposes of comparison, the analysis assumed a constant diameter distribution line and an 18 inch diameter injection well. Using this flow rate and injection well diameter, a 48 inch distribution pipe required a total head of 646.7 ft. The total amount of friction loss needed in each scenario is shown in Table 27. As indicated the ability to provide flow and head to the five locations is limited to 75 ft^3/s per station (375 ft^3/s total). The higher flows on the right side of the table were not calculated because of extremely high head demands or pipe diameters exceeding 6 feet. Moreover, the drawdown caused by extracting more water than this from the well field became a concern. At some point, if more water is needed, the discussion of multiple pipelines and well fields should commence as this would likely improve reliability over a single huge facility and allow staged construction based on growing regional demands.

Table 27. Summary of MAXIMUM pipe and injection well friction losses at multiple locations.

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to each NR1, NR2, NR3, NR4, and NR5 site (ft^3/s)						
		25	50	75	100	150	200	300
30	18	na						
36	18	2599.6						
48	18	646.7	2312.1					
60	18	223.8	785.5	1654.8				
72	18	97.2	328.2	685.9	na			

Variable Distribution Pipeline Diameters

In the previous analysis it was assumed that the distribution pipeline diameter remained constant even as the flow decreased between injection locations. Additional scenarios were examined to determine the increase in head required if the diameters were slowly reduced in the direction of flow. The options shown in Table 28 do not encompass all possible pipe diameter combinations but rather provide an indication of possibilities depending on construction cost versus long-term operation and maintenance costs. Comparison of these values with those presented in Table 27 indicate the potential trade-offs available. For example, a discharge of 25 ft^3/s per node through a constant 72-inch pipe required 97.2 feet of friction head loss. The third option shown in Table 28 requires 273.0 feet of head loss for a pipeline 72- to 36- inches in diameter.

The flow of 100 ft^3/s per node (500 ft^3/s total) produced head losses in excess of the 2,700 feet available at the top of the extraction wells. This caused the EPANET model to exhibit a fatal warning message and halt execution. Therefore, that space it is shown as “na” in Table 28.

Table 28. Summary of pipe and injection well friction losses at multiple locations with variable main pipeline diameters and 18 inch injection well (in feet).

Main Pipe Diameter (inches)		Flow Rate to each NR1, NR2, NR3, NR4, and NR5 (ft^3/s)			
		25	50	75	100
Extraction to NR1	72	73.8	216.8	438.1	na
NR1 to NR2	60	113.3	365.1	754.8	
NR2 to NR3	60	135.5	454.5	948.2	
NR3 to NR4	54	152.2	525.2	1102.4	
NR4 to NR5	54	155.2	537.9	1130.2	
Extraction to NR1	72	73.8	216.8	438.1	
NR1 to NR2	54	141.3	466.1	968.7	
NR2 to NR3	54	180.7	617.8	1294.2	
NR3 to NR4	48	213.5	746.3	1571.0	
NR4 to NR5	48	219.4	769.5	1620.8	
Extraction to NR1	72	73.8	216.8		
NR1 to NR2	54	141.3	466.1		
NR2 to NR3	48	214.1	738.2		
NR3 to NR4	48	246.8	866.6		
NR4 to NR5	36	273.0	963.3		
Extraction to NR1	72	73.8	216.8		
NR1 to NR2	48	195.3	660.9		
NR2 to NR3	48	268.0	932.9		
NR3 to NR4	36	403.8	1467.2		
NR4 to NR5	36	439.3	1563.3		

Extraction and Injection Well Losses

Friction losses in screen and riser sections can be estimated using the Hazen-Williams formula (USACE, 1992) in the form of:

$$H_f = \frac{303 * V^{1.85}}{C^{1.85} * d^{1.167}} \qquad (3)$$

where V is the pipe velocity (ft/s), C is the Hazen-Williams coefficient (100), d is the pipe diameter (ft) and H_f is the head loss (ft/100 ft of pipe). Table 29 indicates the head losses associated with various well diameters for the entire 365 ft of lift line anticipated.

Table 29. Head losses in screen and riser sections.

Well Diameter (inches)	Flow Rate (ft^3/s)	Additional Head Loss (ft)
12	10	24.4
	20	88.0
	25	133.0
	30	186.4
18	10	5.4
	20	19.4
	25	29.7
	30	41.6
24	20	6.8
	25	10.2
	30	14.3
	40	24.4
	50	36.9
30	20	3.0
	25	4.5
	30	6.3
	40	10.7
	50	16.1

The values shown in Table 29 should be viewed as approximate. Without detailed design of screens and filter materials it is not feasible to analyze these losses exactly. Actual values will depend on details that are beyond the goal of this project. An exploratory well should be developed before final design of the pumping facilities is completed. In fact, ASCE standard guidelines for aquifer recharge projects (2001) recommend a complete field investigation test program prior to the preliminary design phase.

Power Line (PL) Route from Pend Oreille Area

As previously mentioned, the northern rail road line was one of three possible routes for the distribution line. The power line (PL) route is shown in Figure 81. Six locations were originally examined for possible injection however PL1 and PL2 were dropped from this phase because aquifer geometry resulted in significant amounts of injected flows feeding the extraction well field (flowing northeast) rather than recharging the aquifer and subsequently the Spokane River. The lengths and elevations of the injection locations are presented in Table 30. The distance to the PL4 location (21.8 miles) is very similar to the NR4 location (22.2 miles). These two locations were used as points of comparison since hydrologically the NR4/SR4 locations provided the necessary lag response to deliver water during the low flow August-September period.

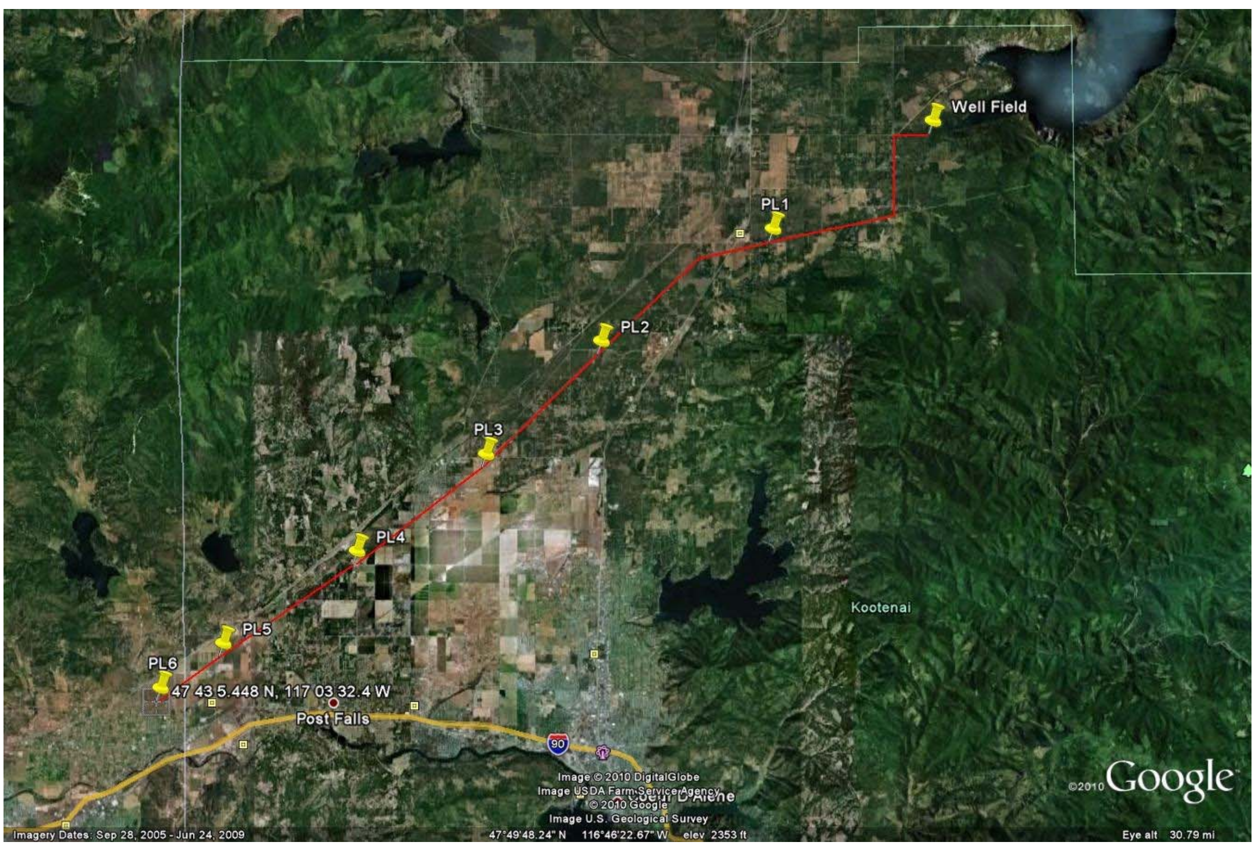

Figure 81. Potential injection well locations along power line (PL).

Table 30. Summary of power line (PL) pipeline properties.

Line Segment	Length (feet)	Elevation (feet)
Well Field	---	2,390
Well Field to PL1	35,966	2,404
PL1 to PL2	31,250	2,281
PL2 to PL3	23,736	2,215
PL3 to PL4	24,029	2,180
PL4 to PL5	24,197	2,114
PL5 to PL6	11,567	2,093

Elevation profiles along the route were also examined. As illustrated in Figure 82, the profiles among the NR and PL lines are quite similar so pump head requirements and well depths were expected to be very compatible.

As a comparison to the northern railroad route, the EPANET model was setup and run to deliver water to the PL4 location (see Figure 83). As noted in the groundwater modeling results, PL1 and PL2 were unacceptable injection site locations due to geological considerations and PL6 was too close to the river to have adequate lag time so only sites PL3, PL4 and PL5 were feasible. PL4 is nearly the midpoint between the ends.

Keeping all factors regarding pipe material properties, depth below groundwater table, and minor losses, Table 31 illustrates the friction losses for transporting water from the well extraction field near Lake Pend Oreille to the PL4 site. In examining the values presented for NR4 in Table 24 and using the friction losses for 100, 200, and 300 ft^3/s as example cases, the 60 inch diameter pipeline and 18 inch injection well field for NR4 requires approximately 229, 798, and 1680 feet, respectively. Similarly, 224, 782, and 1648 feet of head are required for comparable flows along the PL route. Therefore, the routes are very similar in terms of anticipated pumping costs.

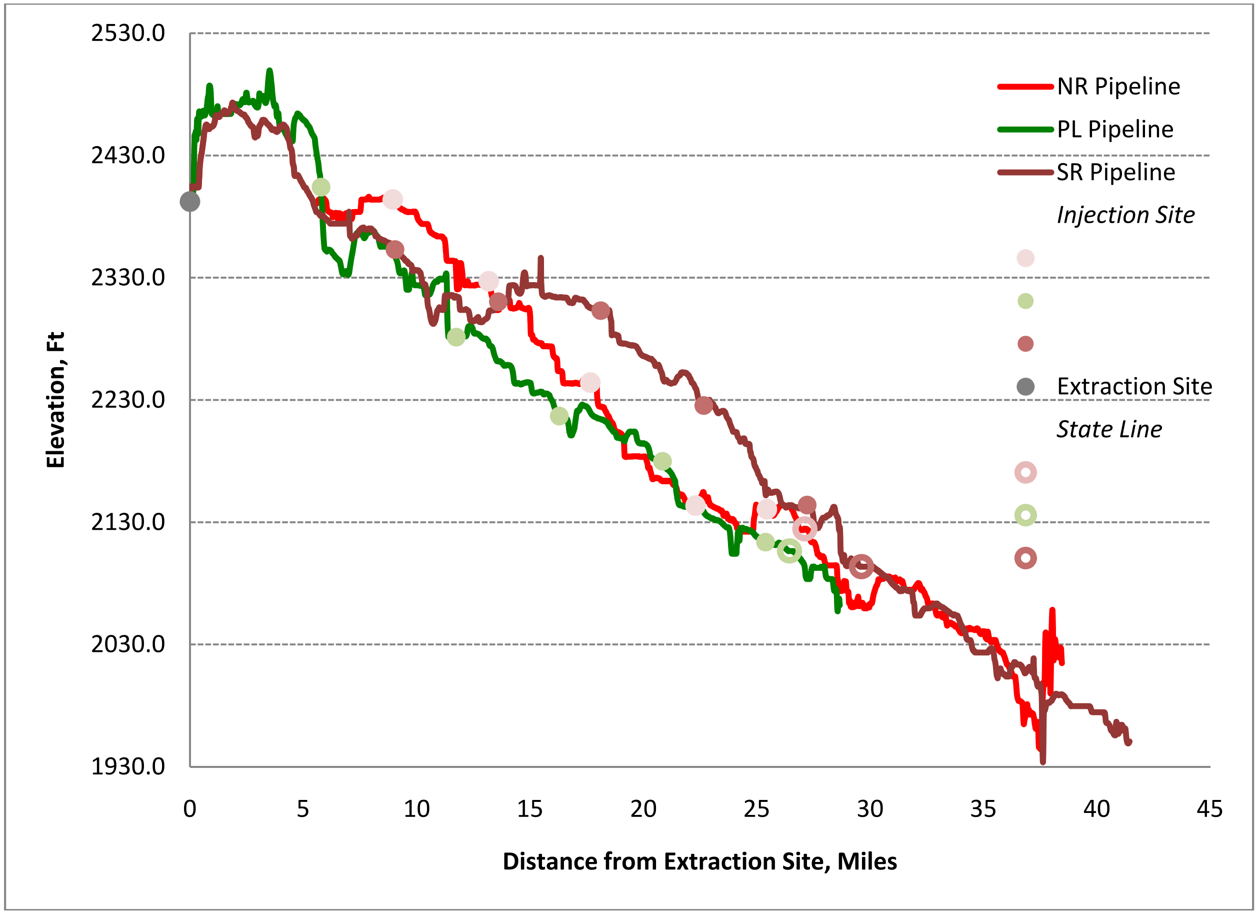

Figure 82. Elevation profile along the three potential alternative routes.

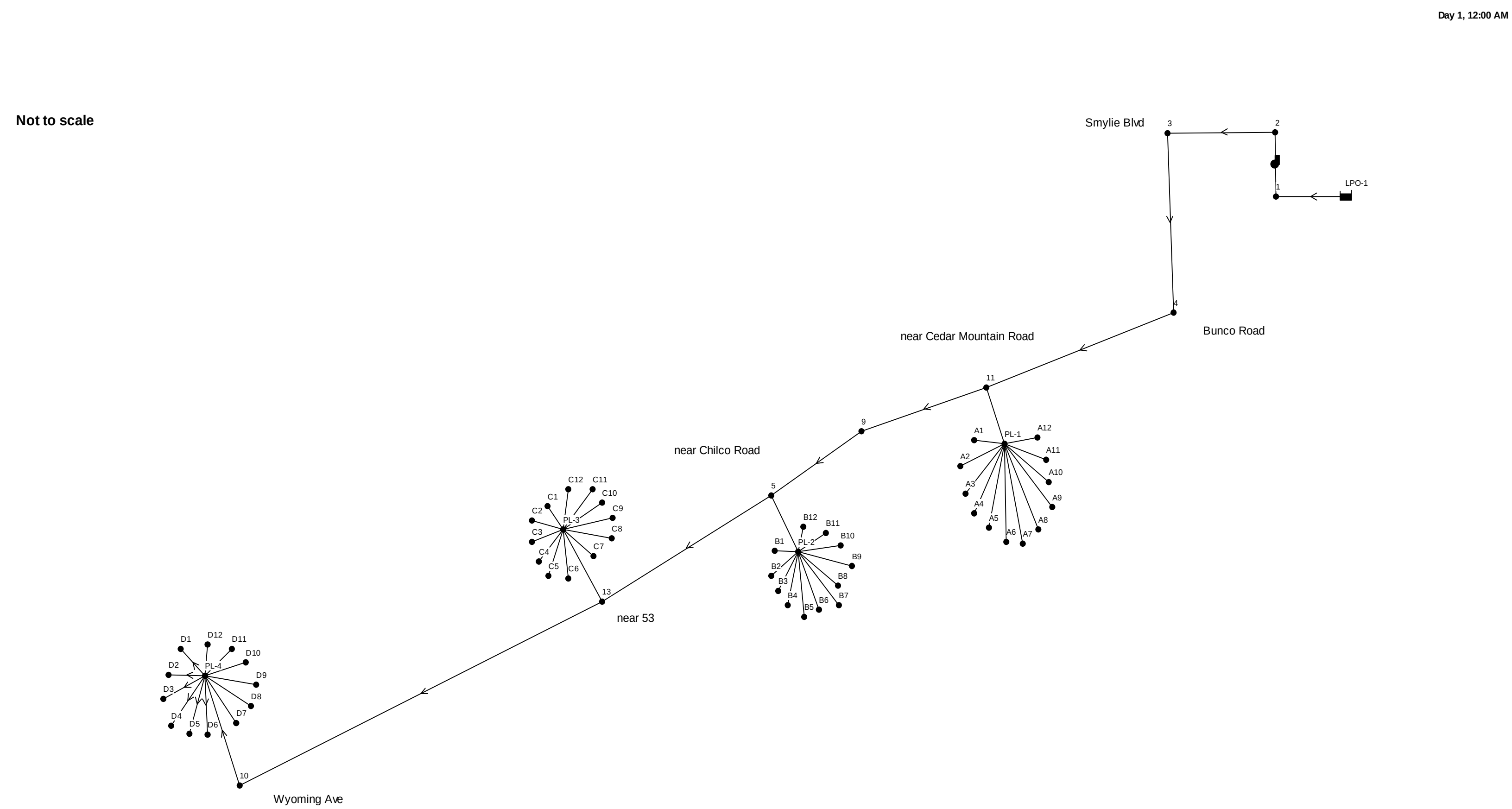

Figure 83. Schematic of potential power line (PL) distribution route.

Table 31. Summary of friction losses from Pend Oreille well field to PL4 (in feet).

Pipe Diameter (inches)	Injection Well Diameter (inches)	Flow Rate to PL4 Location (ft^3/s)						
		25	50	75	100	150	200	300
36	12	265.0						
	18	207.2						
	24	200.2	716.8	1,516.4				
48	12		243.1	440.0	702.4	1,413.2		
	18			382.2	644.6	1,355.4		
	24		178.3	375.2	637.6	1,348.4		
60	12			192.9	281.4	521.1	840.5	1,705.9
	18			135.1	223.6	463.3	782.7	1,648.1
	24			128.1	216.5	456.3	775.7	1,641.1
72	12					253.9	385.3	741.4
	18					196.1	327.5	683.6
	24					189.1	320.5	676.6

Southern Railroad (SR) Route from Pend Oreille Area

As indicated in Figure 82, the southern railroad route is higher in elevation in the areas of the SVRP aquifer most likely to be used as injection locations (SR3-SR5). The locations of the perspective injection locations are shown in Figure 84. Groundwater modeling results indicated that, similar to the PL route, the SR1 and SR2 locations would not be acceptable geologic locations for injections. The SR3-SR5 sites would require deeper injection wells. While potentially feasible, pipeline lengths are similar (see Table 32) and there are no other compelling reasons to elect this route unless right-of-way constraints exist at either of the other two routes. Because of the similarity to the other routes, no EPANET scenarios were explicitly run for this potential route.

Table 32. Summary of southern railroad (SR) pipeline properties.

Line Segment	Length (feet)	Elevation (feet)
Well Field	---	2,390
Well Field to SR1	47,714	2,352
SR1 to SR2	24,082	2,312
SR2 to SR3	23,919	2,304
SR3 to SR4	24,312	2,225
SR4 to SR5	23,758	2,144

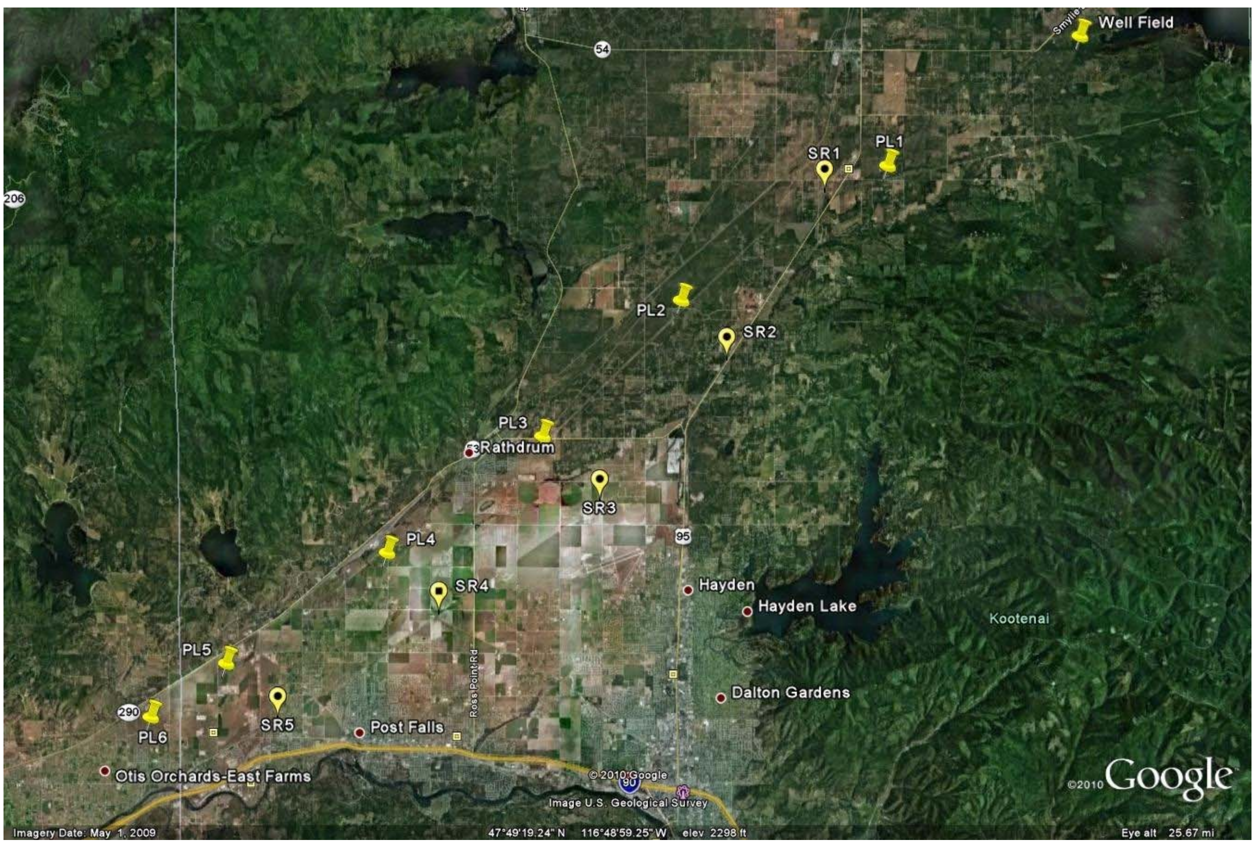

Figure 84. Layout of southern railroad (SR) route.

6.4 SVRP Groundwater Model Limitations

There are several potentially significant assumptions built in to the bi-state MODFLOW model that were subsequently used in the updated Visual MODFLOW-2009 model. The benefits of using a regionally agreed upon model far outweigh the limitations but they are nevertheless worth highlighting.

- The original model was developed using existing data from 2005. New information suggests some changes in recharge may be warranted. Additional studies are ongoing regarding the natural drainage from Lake Pend Oreille that may change model assumptions.

- The monthly time step used in the groundwater model prevented finer scale analysis without building in some assumptions.

- The lack of vertical unsaturated hydraulic conductivity (K) data required approximation of the K versus soil moisture relationship needed in the infiltration modeling.

- The period of operation was defined by the original bi-state study.

- No groundwater quality modeling meant that the impact of return flows on TMDL requirements used boundary conditions reflecting current conditions rather than expected future conditions. Phosphorus concentrations, in particular, were set at the original background levels with no change due to additional flows.

6.5 Interaction with State of Idaho

As part of the project, representatives from the Idaho Department of Water Resources (IDWR) were provided a draft copy of this report. Subsequently, a follow-up conference call was conducted to provide them with an opportunity to provide feedback. As part of their own planning efforts, Idaho has projected increased summer demands on the Spokane River as a result of future Idaho growth and so this project was presented as a potential option for them to consider.

Overall, IDWR remained non-committal regarding both the project and the feasibility of a third-party obtaining an Idaho water right for the appropriate high flow periods determined by the investigation. From a technical perspective, since we used the bi-state MODFLOW model, IDWR did not express any concerns regarding results. Furthermore, while they did indicate that aquifer storage and recovery was not a recognized water management option in Idaho, they also pointed out that storing water as mitigation for water diversions was a strategy used elsewhere in the state.

Ownership of water rights is also a complex issue although in general any entity can hold a water right in Idaho. The water right can be in the name of an individual, group of individuals, organization, corporation, or government agency. While private ownership of instream flow rights is possible on a temporary basis through the water banks, only the Idaho Water Resources Board can apply for and hold new appropriations for instream flow water rights. Consequently, the water would likely have to be shown to provide a beneficial use beyond instream flow enhancement. Additional legal perspectives would have to be obtained to clarify this issue.

The tentative extraction well field would be near or in Farragut State Park. IDWR did not know of any legal reason this would not be a valid location although they were relatively quick to point out that the entire water right process would likely undergo significant public scrutiny. Opportunity for significant cost savings exist if the extraction wells could be located near Lake Pend Oreille where the depth to groundwater is less.

6.6 Right-of-Ways

The placement of the distribution pipeline has been proposed along three different paths; the northern railroad route, the power line route, and the southern railroad route. The northern railroad route follows the Burlington North Santa Fe (BNSF) Railroad line between Athol, Id and Spokane, WA. The power line route follows the Bonneville Power Administration and Avista Utilities Corp power lines along a comparable path. The southern railroad route follows the Union Pacific mainline between Athol and Spokane. There has been some talk about relocating the Union Pacific line but no final decision has been made at this time so the current alignment was used.

In order to install a pipeline near a BNSF railroad line (northern railroad, NR), a completed application for a permit to access BNSF's property is required. This application includes a $600.00 non-refundable processing fee and two sets of drawings of the area requesting to be occupied. Pipeline application processing instructions can be obtained from the BNSF website. Commercial general liability insurance, business automobile insurance, workers compensation and employer's liability insurance, railroad protective liability insurance and pollution legal liability insurance (if necessary) are also all required to construct a pipeline near the railroad. The permit response time is generally around 45 to 60 days after the application is received.

The power line (PL) corridor contains one Avista Utilities 230 kV line (from Rathdrum to Cabinet St.) and two Bonneville Power Administration lines (from Lancaster Rd. to Taft Ave. 500 kV and Bell to Noxon, 230 kV). Avista does not own any of the property along the corridor however it does have a series of easements which allow them to install, operate and maintain their power line. Their easements are generally not exclusive but they do reserve the right to make sure any other utility installations do not interfere with the integrity of their structures. To further inquire about right-of-way costs of a pipeline installation, detailed construction plans must be submitted to both parties involved.

Union Pacific requires similar plans for non-flammable substances encroaching on their railroad right-of-ways. Plans for proposed pipelines shall be submitted to and meet the approval of the chief engineer of the railroad or his authorized representative before work is begun and all work on railroad right of way, including the supporting of the track or roadbed, shall be subject to his inspection and direction.

The goal of these alignments is to utilize existing right-of-ways as much as possible to reduce the easement costs. If additional right-of-way is required, the cost can be quite substantial and the time to negotiate with multiple individuals can be long. Many factors impact the cost but estimates are between $0 and $195 per linear foot of water pipeline (sometimes more for oil/gas systems). For rural (non-industrial) properties, this is closer to the $30-$65/ft range for a 72-inch pipeline with a 65 foot wide construction right-of-way.

7.0 Cost Analysis

The regional bi-state MODFLOW model has been used to demonstrate the technical viability of the SVRP aquifer storage and recovery project, however, the overall feasibility of the ASR project will ultimately depend on economic cost compared to alternatives. Therefore, after updating the model to Visual MODFLOW-2009, cost analysis of each final alternative design scenario was conducted. ASCE (2001) recommends including the costs of land acquisition, right-of-way acquisitions, planning, engineering, construction, operation and maintenance, contingency, permit and legal, replacement, and decommissioning.

Cost estimates, which included capital (construction) costs and operation & maintenance (pumping and repair) costs, were completed for each promising scenario. The capital costs were those associated with the water diversion and delivery system, construction of the injection wells and/or infiltration basins, land or right-of-way costs, treatment costs, and permitting costs. O&M costs included pumping expenses, personnel, and long-term upkeep. Environmental and time factors were also included in the analysis. To the maximum extent possible, we attempted to obtain local estimates. It should be noted, however, that the size of the infrastructure related to water quality treatment and the larger pipeline diameters are not often (if ever) built in the area so national comparisons were necessary.

7.1 Pipeline Construction

There are several ways of estimating pipeline costs that are routinely used but companies such as R.S. MEANS often provide good starting points as do similar construction activities at other locations. The difficulty is that most construction activities involve unique aspects that are difficult to ascertain from the literature or project descriptions. Factors such as road crossings, pavement replacement, water table and geologic considerations to construction, and utilities reconstruction can significantly increase the price of construction. In examining the literature and bid jobs from around the region and country, we were able to estimate the costs of the various components of the project. As mentioned previously, this would be a relatively large project by regional standards so there are few local examples to draw on. Thus, much of the estimates are from other locations.

El Paso County, Colorado (2002) used a composite of bid tabulations and found that for welded steel pipe the in-place cost for pipe larger than 24 inches was:

$$\text{Cost per linear foot} = 10.1667 \text{ x pipe diameter} - 163 \tag{4}$$

The Canadian Energy Pipeline Association used similar relationship in proposing:

$$\text{Cost per linear foot} = 7.759 \text{ x pipe diameter} \tag{5}$$

For a 60-inch diameter pipe, the two estimates work out to be $447 per foot and $466 per foot, respectively. Likewise, for a 48-inch diameter pipe, the respective costs would be $325 and $372 per foot and for a 36-inch diameter pipe the costs would be $203 and $279 per foot.

The City of Greeley, Colorado is in the process of constructing a 30 mile long, 60-inch water line for conveying 77 ft^3/s (50 MGD) for approximately $80,000,000. This works out to a cost of $505 per foot which is approximately 10% higher than the estimated equations. Of course local conditions such as right-of-way costs, pavement replacement, elevation grades, utilities relocation, and other construction obstacles can have a tremendous impact on costs.

The City of Thornton, Colorado installed 5.5 miles of 42-inch diameter pipe for $5,200,000 which is equal to $179 per foot compared to $264 and $326 respectively from the estimator equations. However, a 5.5 mile, 78-inch diameter raw water pipeline in Texas cost approximately $668 per foot compared to $630 and $605 per foot so the equations do not consistently over or under predict cost.

For local comparison, the City of Spokane requested bids for two 36-inch diameter projects in 2008 and 2009. One project received bids for approximately 17,000 feet of pipe at an overall cost ranging from $151 to $222 per foot. The second project received bids for nearly 21,900 feet of pipe ranging from $127 to $177 per foot. The composite average of $169 per foot is nearly 30% less than the average of the two estimation equations provided above. Whether or not this discount would scale to larger pipe diameters would likely depend somewhat on the capabilities of the regional construction firms and the general economy at the time of construction. After examining these ranges, we elected to use the average of Equations (4) and (5) for estimating purposes.

This resulted in the pipeline costs shown in Table 33, which are in line with typical reported project costs but well below what several other agencies propose using for planning purposes. For example, values for transmission systems used for rural construction of pipelines by the St. Johns River Water Management District (Florida) are also shown in the table. The US Bureau of Reclamation (USBR) used a flat rate of $10/inch diameter/linear foot of pipeline in their 2006 evaluation of the Boise/Payette Water Storage Assessment Report. Although SVRP construction would likely be easier than in the USBR project area, it was impossible to completely dismiss the difference in price estimates. Conversely, HDR reported prices from $2.75 to $4.50/in/ft for several pipelines in the Corpus Christi, Texas area including a 54 inch, 29 mile long water pipeline. In a 2007 Master Plan study for Las Cruces, New Mexico CDM had values of $5.70 to $5.10/in/ft of 36 and 42 inch diameter lines. Therefore, while we used the average equation values previously described, the discrepancy in prices was noted and used in the final analysis in a sensitivity analysis where we increased and decreased the pipeline costs to demonstrate potential ranges in costs.

Table 33. Estimated costs for distribution pipeline.

Diameter (inches)	Cost		St. Johns River Water Management District $/linear ft
	$/in/ft	$/linear ft	
24	$5.57	$133.61	$181.74
30	$6.25	$187.39	$326.09
36	$6.70	$241.16	$369.57
48	$7.26	$348.72	$451.30
60	$7.60	$456.27	$639.13
72	$7.83	$563.83	$860.00

7.1.1 Spokane Collection and Distribution System Cost

The preferred route for this system is likely either from the Spokane River to the power line (PL) option or the Spokane River to the northern railroad line (NR) as previously discussed. These options assume a direct diversion from the river without need for a diversion dam or other instream structure other than the intake. The construction costs by pipe diameter for the PL and NR options are shown in Table 34 and Table 35, respectively. These costs need to be compared to long-term pumping costs to get a complete picture of which combination of options provides the best value.

Table 34. Cost of pipeline from Spokane River to power line injection locations.

Pipeline Information	PL5	PL4	PL3
Pipeline Length (ft)	18,658	42,855	66,884
Pipeline Diameter			
36 inches	$4,499,600.	$10,335,000.	$16,129,900.
48 inches	$6,506,400.	$14,944,300.	$23,323,600.
60 inches	$8,513,100.	$19,553,500.	$30,517,200.
72 inches	$10,519,900.	$24,162,700.	$37,710,900.

Table 35. Cost of pipeline from Spokane River to northern railroad injection locations.

Pipeline Information	NR5	NR4	NR3
Pipeline Length (ft)	25,328	40,929	64,919
Pipeline Diameter			
48 inches	$8,832,300.	$14,272,600.	$22,638,300.
60 inches	$11,556,400.	$18,674,700.	$29,620,700.
72 inches	$14,280,600.	$23,076,800.	$36,603,000.

For comparison purposes, the costs of delivering water from the river to the PL5 versus NR5 site would be 35.75% less for all pipeline diameters. However, the costs for PL4 versus NR4 and PL3 versus NR3 are higher by 4.5% and 3%, respectively. The costs along the SR route are similar to the PL route values (e.g., a 60 inch pipeline would be $8,633,700, approximate $120,000 more in construction). The costs vary only slightly by site with costs within 2% of the comparable sites for all three locations (SR5, SR4, and SR3).

7.1.2 Pend Oreille Collection and Distribution System Cost

Using the same methodology previously described, cost estimates for the Lake Pend Oreille well field to the injection sites along the power lines (PL) and northern railroad (NR) routes were examined. As indicated in Table 36 and Table 37, the pipelines from the Lake Pend Oreille well field would be considerably longer than the Spokane River pipelines and therefore cost proportionally more money to construct. The differences between the PL and NR routes are fairly insignificant compared to the overall cost and uncertainty so either route would likely be acceptable. Complete analyses of the total system cost versus the amount of water delivered are presented later in this chapter.

Table 36. Pipeline costs from Lake Pend Oreille wells to power line injection locations.

Pipeline Information	PL3	PL4	PL5
Pipeline Length (ft)	90,952	114,981	139,178
Pipeline Diameter			
48 inches	$31,716,500.	$40,095,800.	$48,533,700.
60 inches	$41,498,800.	$52,462,500.	$63,502,900.
72 inches	$51,291,000.	$64,829,200.	$78,472,063.

Table 37. Pipeline costs from Lake Pend Oreille wells to northern railroad injection locations.

Pipeline Information	NR2	NR3	NR4	NR5
Pipeline Length (ft)	69,700	93,263	117,253	132,854
Pipeline Diameter				
48 inches		$32,522,500.	$40,888,100.	$46,328,400.
60 inches	$31,802,100.	$42,553,200.	$53,499,100.	$60,617,400.
72 inches	$39,298,600.	$52,584,000.	$66,110,200.	$74,906,400.

The number of significant figures shown in the four preceding tables is not meant to imply certainty in estimates but merely for a means of comparison. Recall that if the USBR figure of $10/in/ft was used, the 72-inch pipeline to NR4 cost estimate would be $84,422,160 rather than the $66,110,200 value shown in the table.

7.1.3 Construction Contingencies

A search of construction plans found that allowances for cost overruns due to unforeseen events are generally planned. Allowances of 5 to 10% of the construction costs are typical values with amounts up to 25 to 35% reserved in difficult construction environments. As this project would be constructed on the SVRP aquifer, uncertainties such as bedrock or excessively high groundwater levels are not anticipated. Nevertheless, we elected to estimate contingencies at 15% to reflect uncertainties and price fluctuations over time.

7.2 Water Treatment (if necessary)

Since a groundwater well field in the Spokane area turned out to be physically infeasible, the possibility of direct river diversions were explored. Cost estimates were prepared for water treatment of Lake Pend Oreille and the Spokane River. According to Federal Regulations (40 CFR 144.12), surface water that is injected into the groundwater aquifer is governed by EPA's Underground Injection Control program and must be treated to drinking water standards. Idaho administers the Underground Injection Control Program under Idaho Code Title 42 Chapter 39 and IDAPA 37.03.03, administered by IDWR. Injection wells cannot directly or indirectly cause negative impacts to groundwater resources (Codrington, 2009). Due to the relatively low concentrations of contaminants, Lake Pend Oreille water can be treated using conventional water treatment technologies. The Spokane River has high concentrations of heavy metals and total suspended solids; therefore, a more advanced iron and manganese treatment process as well as additional sediment removal is necessary. These cost estimates are for treating the water to drinking water standards, not groundwater quality standards. Washington State has a more stringent policy on injection wells revolving around anti-degradation of groundwater. To meet these groundwater quality standards, surface waters would need to be treated to the background water quality of the SVRP aquifer. The aquifer water quality may be higher than drinking water standards, which would require additional removal of constituents and cost even more than conventional treatment.

Six cost estimates were completed for three flow rates (25, 50, and 100 ft^3/s) and both types of treatment (conventional and advanced). A cost estimate for 500 ft^3/s was not completed due to the limited amount of information on water treatment plants of that size. There are very few treatment plants that treat that much water, therefore we were not able to obtain a reliable cost estimate. Cost estimates were determined using equations and spreadsheets developed by McGivney and Kawamura (2008). The equations accounted for economy of scale, local tax rates, and inflation rates. All cost components, such as design, construction management, equipment and materials cost, labor, mobilization and demobilization, administration costs, and insurance, were included. Cost estimates do not include the cost for purchasing the land required for the treatment plant. Table 38 shows treatment costs for both types of treatment at 25, 50, and 100 ft^3/s. Treating 25 and 50 ft^3/s of Spokane River water would cost more than Lake Pend

Oreille water. However, according to the methodology employed, treating 100 ft^3/s of Spokane River water would cost less than Lake Pend Oreille water. This may be due to the extremely large scale; the treatment tanks required for conventional treatment become very large at flows approaching 100 ft^3/s. Alternatively, and perhaps more likely, is that extrapolation of the equations used to estimate costs may not be valid at this scale. It is rare for municipalities to build treatment plants larger than 40 MGD, which makes extrapolation of the equations questionable.

Operation and maintenance costs are also shown in Table 38. These costs include manpower, chemicals, electricity, maintenance repairs and periodic replacement of equipment, and other supplies and services that are necessary to operate a water treatment plant. Actual costs vary significantly depending on the cost of electricity and the availability of chemicals and equipment. This is only a preliminary estimate to provide an idea of annual costs for the treatment part of the project.

Table 38. Cost Estimates for both water sources at 25, 50, and 100 ft^3/s.

	25 cfs (16.2 MGD)	50 cfs (32.3 MGD)	100 cfs[1] (64.5 MGD)
Conventional Treatment - Lake Pend Oreille			
Construction	$ 40.4 million	$ 56.6 million	$ 114 million
Annual Operation and Maintenance	$800,000	$1.2 million	$1.5 million
Iron & Manganese Removal - Spokane River			
Construction	$ 46.6 million	$ 67.9 million	$ 113 million
Annual Operation and Maintenance	$1.0 million	$1.2 million	$1.3 million

[1] Equations used to estimate costs may not be valid for this flow rate. Treatment plants are typically not this large.

For verification of these cost estimates, we compared costs with other treatment plants that have been recently constructed. The 24 MGD Santan Vista Water Treatment Plant in Arizona cost $46.6 million to construct and $1.3 million annually to operate. The 42 MGD Brushy Creek Regional Water Treatment Plant in Texas cost $63 million to construct (operation and maintenance costs were not available). The 92 MGD San Juan-Chama drinking water project in New Mexico cost $160 million. These construction and operation and maintenance costs are similar to our estimates in Table 38. For the possible 200 ft^3/s and 300 ft^3/s scenarios, we elected to simply multiply these costs by factors of 2 and 3, respectively. While this neglects the economy of scale that would likely be achieved, the potential savings were thought to be within the accuracy of the estimates.

Regardless of the exact cost of the larger treatment plants, it appears that the costs would prevent direct surface withdrawals from being a likely alternative. Therefore, we elected to focus on subsurface extraction and injection without exposing the water to the surface.

7.2.1 Groundwater Source Treatment

The Rathdrum Prairie Sensitive Resource Aquifer was designated a sole source aquifer in 1978 (Federal Register, Vol. 43, No.28-Thursday, February, 9, 1978). The designation concluded that if contamination to the aquifer occurred, it could create a significant public health hazard as this aquifer is the sole source of drinking water for much of the area. Subsequent to the sole source aquifer designation by the U.S. Environmental Protection Agency, Idaho designated it a sensitive resource aquifer with additional water quality requirements. The entire project area is within this Sensitive Resource Aquifer Area designation, and is subject to the most stringent requirements under Idaho's ground water quality rule (IDAPA 58.01.11).

Treatment may be necessary for ground water to ground water transfers such as contemplated in this study. Site specific temporal and spatial assessments of source and receiving area chemical condition are necessary to determine what, if any, treatment requirements might be necessary. Such studies are beyond the scope of this report. Because ground water throughout the SVRP is of generally high quality, this study assumes no treatment will be necessary for groundwater source to receiving area transfers.

7.3 Well Drilling and Pumping

7.3.1 Extraction and Injection

A local well drilling company (H_2O Well Service, Inc) provided a drilling cost estimate of $10/diameter inch/foot of depth plus approximately 10% mobilization cost. This is equivalent to $11/diameter inch/foot of depth. The estimate is only for the drilling, casing, well screen, and other expenses associated with finishing a well would cost extra although without material effect on the outcome or overall cost. Table 39 provides the drilling cost information at each location considered viable along the NR, PL, and SR routes. The trade-off that must be considered is in the cost of the well versus the cost of pumping. Since the well drilling costs are one-time expenses, it may be cost effective to use a larger diameter well to save pumping costs. The calculations performed with the EPANET model indicated a fairly significant decrease between the 12-inch and 18-inch diameters.

Table 39. Summary of well drilling cost ($/well).

Location	Extraction	NR1	NR2	NR3	NR4	NR5	NR6
Depth (ft)	365	377	335	264	185	171	143
Diameter (inches)							
12	$48,180	$49,764	$44,220	$34,848	$24,420	$22,572	$18,876
18	$72,270	$74,646	$66,330	$52,272	$36,630	$33,858	$28,314
24	$96,360	$99,528	$88,440	$69,696	$48,840	$45,144	$37,752
30	$120,450	$124,410	$110,550	$87,120	$61,050	$56,430	$47,190
Location				SR3	SR4	SR5	
Depth (ft)				325	249	185	
Diameter (inches)							
12		--	--	$42,900	$32,868	$24,420	--
18		--	--	$64,350	$49,302	$36,630	--
24		--	--	$85,800	$65,736	$48,840	--
Location				PL3	PL4	PL5	
Depth (ft)				236	209	155	
Diameter (inches)							
12		--	--	$31,152	$27,588	$20,460	--
18		--	--	$46,728	$41,382	$30,690	--
24		--	--	$62,304	$55,176	$40,920	--

7.3.2 Pumps and Pump Station

The scenario envisioned for the Pend Oreille well field is that individual pumps will be used to extract water from wells at a rate of 25 ft^3/s (11,220 gpm) per well. Therefore, to supply 100 ft^3/s to the distribution pipeline, four individual wells will be required. Each of these pumps will be required to supply 365 feet of static lift and nearly 30 ft of well screen losses for a total of approximately 400 feet of head at the design discharge. From a location near the top of the well, the water will be combined into the main distribution line and a pump station will be required to deliver the water to the injection well field. The friction head needed varies by pipe diameter, flow rate, and pipeline length. The flow rate could be as high as 300 ft^3/s and the head could be almost 1,900 feet. At extremely high heads, it may be advisable to have more than a single pump station at some location down the line to avoid excessive pressures in the pipe at the discharge side of the pump.

In order to size a pump to the pipeline specifications described above, a commercial web site called pump-flo.com was used. This web site uses specific flow rates and head values to generate pump curves of various manufacturers. Using this information it was decided that a vertical turbine pump would be an adequate choice for our project. Two manufacturers were determined to be sufficient choices for the initial pump, the American Marsh 480 series and Fairbank Morse 7000 series.

In a 2002 Colorado River Conveyance Feasibility Study, Boyle proposed the following equation for estimating pump stations costs without contingencies:

$$Cost = 46{,}000\left(\frac{448.8*Q}{100}\right)^{0.75}\left(\frac{H}{300}\right)^{0.66} \tag{6}$$

where H is the head [ft] and Q is the flow rate [ft^3/s].

Feasibility studies conducted by several other consulting firms and government agencies have adopted an even simpler way of estimating cost based on the horsepower required by the station.

$$\text{Cost} = K_1 \text{ x horsepower} \tag{7}$$

where K_1 is a constant ranging from $700 to $2,500 per horsepower.

The USBR used the horsepower equation and a K_1 value of $2,000 to estimate pumping costs in their 2006 investigation of the Boise/Payette Water Storage Assessment Report. While relatively straightforward to implement, these two equations produce very divergent results particularly at high heads or high flow rates. Using the calculator on the WSU web site: http://irrigation.wsu.edu/Content/Calculators/General/Required-Water-Pump-HP.php, flow rate and heads were converted into total horsepower requirements assuming 80% pump efficiency and 85% motor efficiency. Both equations were then plotted as a function of head for a discharge of 25 ft^3/s (see Figure 85). While the horsepower equation is only slightly below the Boyle equation at 50 feet of head, the equations quickly diverge at high heads. Likewise, at high flows (see Figure 86) a K_1 value of $1,250 is required to intersect the lowest point but again the cost using Equation (7) rapidly accelerates to 2 or 3 times Equation (6).

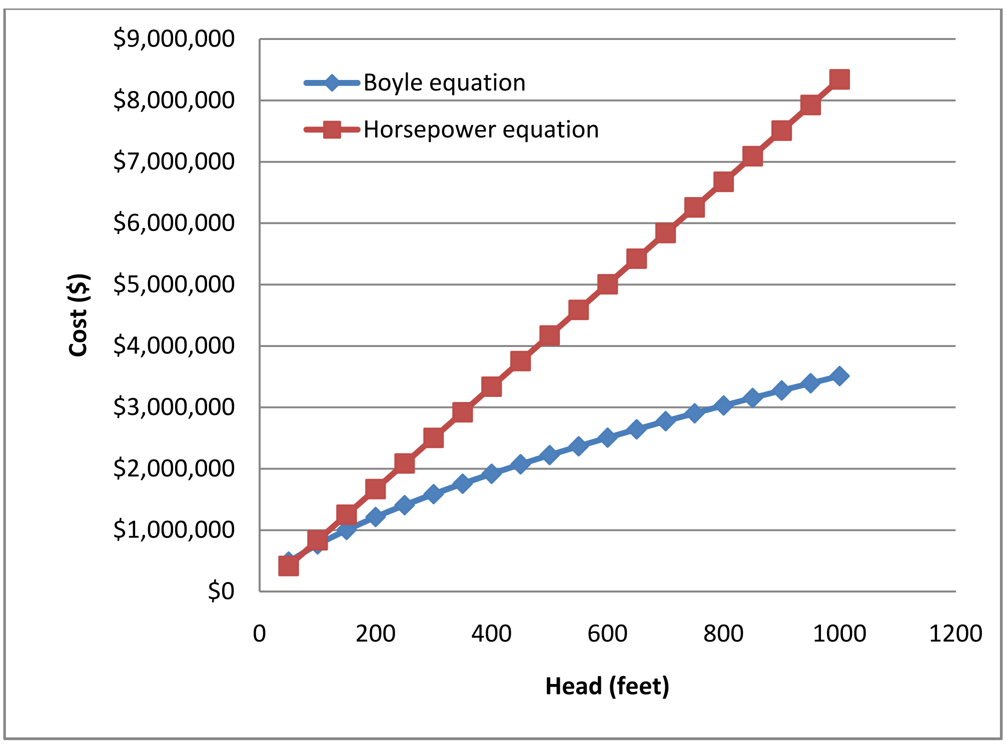

Figure 85. Estimated pump station costs for a 25 ft^3/s flow rate and $2,000 K_1 value.

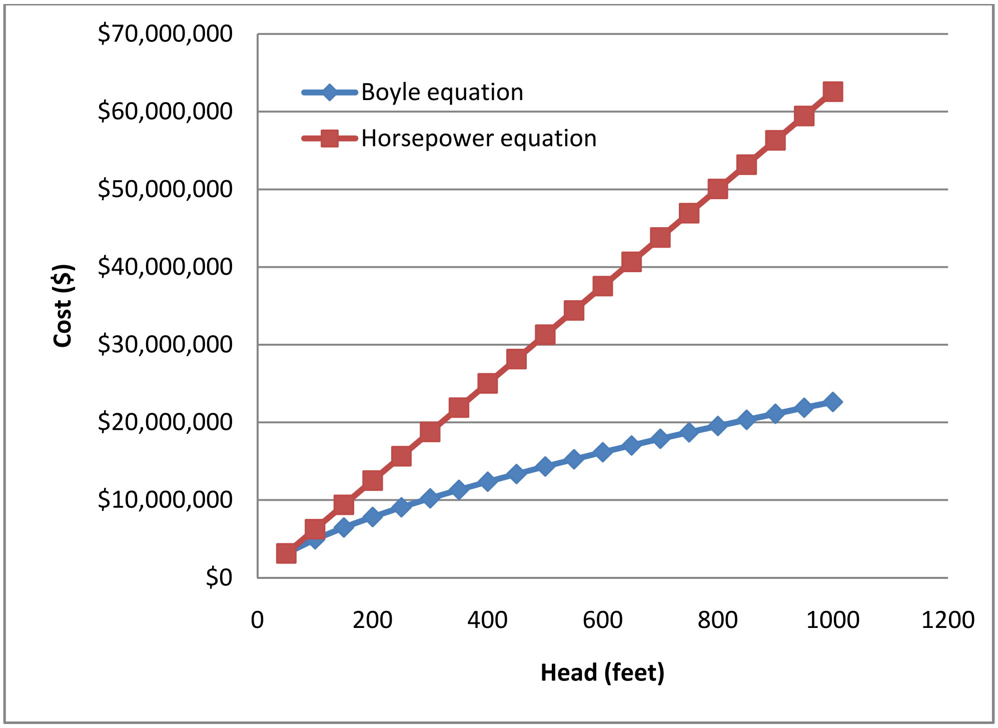

Figure 86. Estimated pump station costs for a 300 ft^3/s flow rate and $1,250 K_1 value.

CDM (2007) also estimated costs of several smaller pumping stations (heads from 80 to 267 feet and flows from 1 to 3 ft^3/s) using equation (7) with 2010 cost estimate of $1,688 per horsepower which seems to suggest that this equation is widely used. Looking at the trend lines in Figure 85 and Figure 86, however, reveals that there is no economy of scale factored in to the horsepower equation. In other words, for a constant head, the cost of a 50 ft^3/s plant is twice that of a 25 ft^3/s facility.

To account for the large-scale nature of this design, we elected to use the average value of Equations (6) and (7) for the estimate. The pumping stations represent fairly significant portions of the overall cost and should be closely examined in the preliminary design phase of the project.

The other concern related to this was the size of the pressure increase within the pipe. In this analysis, the maximum pressure head was assumed to be 1000 feet of water. If the delivery pipe system required more pressure than this, it was assumed that two pump stations would be constructed at appropriate locations (e.g., far enough apart that friction loss significantly reduced the pressure in the pipeline before adding additional pump head). For example, if the system required a total of 1,200 feet of friction loss, the price was determined for two (2) stations each producing 600 feet of head.

To estimate the cost of the extraction pumps, we obtained a written price quote from Peerless Pump in Indianapolis, Indiana. These are high flow (25 ft^3/s), high head (400 ft) pumps beyond the typical range of most well pumps. The quote, obtained in fall 2010, was substantially higher than a ballpark number tossed out by a different vendor. The Peerless estimate was $1,356,604 per pump compared to an assumed $400,000 initially used. This required us to recalculate our original cost estimates, however, because Peerless Pump put more time and effort into coming up with the quote, we felt that it was considerably more accurate than the previous value.

7.3.3 Infiltration

Originally, it was hypothesized that we could increase the viability of the ASR project by exploiting a shallow, underground infiltration gallery and letting vertical drainage provide some of the lag time necessary. This could be implemented as a stand-alone project or as a means of extending the pumping period back into winter months. Utilizing an underground infiltration gallery would require additional pipeline, excavation, right-of-way or area, geotextile liner, and underground storage/infiltration units. However, it may have provided for a much longer pumping period thus reducing the cost per quantity of flow delivered.

We began investigating costs before the unsaturated flow modeling results were available. An estimate for the underground storage/infiltration units was prepared based on a 2010 price list provided by AquaBlox® for their product. Each unit consists of 8 panels: a top and bottom, two sides, and four cross braces/ends. The smaller unit is a 26.5 inch long by 16 inch wide by 9.5 inch high structure thereby providing 2.94 ft^2 of drainage area. In bulk, the top and bottom sections cost \$7.13 per piece, the sides cost \$3.11, and the cross braces \$1.99 per piece. The total cost per unit is approximately \$28.50 plus shipping. To keep fines and other debris from clogging the pore spaces, the units would be covered by a geotextile membrane liner. The cost of these is relatively modest, at approximately \$0.70 per square foot.

The storage/infiltration units have a load bearing capability of 38 pounds per square inch (psi) so there are various places where they could be installed. Beneath sidewalks, along existing roadway right-of-ways, underneath parking lots or parks, or in dedicated open spaces are all feasible.

The costs of excavation and backfilling need to be included for this option. Like most other construction-related costs, these can be extremely site specific depending on the substrate encountered. In 2007, the Oregon Department of Transportation (ODOT) reported excavation costs averaging \$17.06 cubic yard (yd^3) based on 19 projects and 83,200 yd^3 of material. The City of Spokane recently received bids for earthwork at the Qualchan Golf Course related to a bank stabilization project where excavation costs varied between \$6.00 and \$20.00/yd^3, which is in line with the ODOT report. In addition, backfilling costs were estimated to be between \$10.00 and \$20.00/yd^3. The average cost of excavating and backfilling came to approximately

$29.00/yd^3. Without site-specific investigation, it was decided that this would be a reasonable value to use for this estimate.

When the unsaturated flow modeling results from HYDRUS2D indicated that the maximum rate of flow would be approximately 0.75 ft/day/ft^2 of area, the idea of infiltration became infeasible. The amount of surface area required to produce significant flow increases in the Spokane River became a barrier to adopting this in any area with high hydraulic conductivity so future economic analysis were suspended.

7.4 Operation and Maintenance Costs

Once the project construction is complete, there will be annual costs associated with keeping the facility in good working order. These costs include personnel, training, inspection, monitoring, repair and replacement, and energy. While the single largest portion of the operation and maintenance (O&M) costs is likely to be the energy cost of pumping the water (see Section 7.4.1), costs such as repair and replacement cannot be totally overlooked. Based on an assumed 20 year design life, repair and replacement of mechanical and electrical components was estimated to cost approximately 8% of the original cost (3% repair and 5% replacement). Since we assumed a design life of 30 years, the replacement value was reduced from 5 to 3.3 % and an O&M cost of 6.3% on all construction costs except the water treatment facility was used. O&M costs for the water treatment facility were computed separately.

7.4.1 Pumping Costs

The cost of operating the well pumps depends on several underlying assumptions regarding the overall well and pipeline design. The U.S. Department of Energy suggests the following equation for estimating the frictional loss portion of pumping costs ($ per hour of pumping):

$$Cost = \frac{1}{1706} \frac{k\ f\ L\ Q^3}{D^5\ e_p\ e_m} \tag{7}$$

where f is the friction factor (typically 0.015 to 0.0225), L is the pipe length in feet, and D is the inside pipe diameter in inches. The 1706 is a conversion factor that accounts for the mixed units used in this equation (gpm, inches, feet, seconds, and hours).

Because the friction factor (f) is a function of the Reynolds Number, it varies with temperature and velocity (Moody Diagram) and is therefore difficult to determine when evaluating multiple scenarios. Other ways of estimating costs avoid the iteration approach. According to the Engineering ToolBox web-based calculator, the cost ($ per hour of pumping) can be estimated using:

$$Cost = 0.746 \frac{k\ H\ Q}{3960\ e_p\ e_m} \tag{8}$$

where k is the electricity cost in dollars per kilowatt-hour (kWh), H is the total head in feet, Q is the discharge in gallons per minute (448.8 gpm = 1 ft^3/s), e_p is the pump efficiency, and e_m is the motor efficiency.

According to the U.S. Energy Information Administration (2010), the average industrial user in Idaho is charged \$0.0471/kWh or 4.71 cents per kWh versus 7.78 cents per kWh for a residential customer. In Washington, average rates are similar for industrial and residential users at 4.16 and 7.73 cents per kWh, respectively. As of February 2010, Avista charged residential users in Idaho and Washington approximately 8.09 and 7.78 cents per kWh, slightly above the state average. Assuming the project purchases its electricity from Avista and is classified as an industrial user rather than a commercial user (commercial rates are somewhere between industrial and residential depending on energy used), a conservative rate of \$0.05/kWh would seem appropriate for this analysis regardless of whether the facility was located in Idaho or Washington. However, Avista has also recently requested the public utilities commissions in Idaho and Washington to approve rate increases of 13.1 and 13.4% respectively in the two states. Therefore, a rate of \$0.055/kWh was used in the following calculations. However, as pumping cost is directly (linear) proportional to the rate (k), other electricity prices are relatively easy to calculate.

Pumping efficiency is calculated using the following equations:

$$e_p = \frac{Horse\ Power\ Out}{Horse\ Power\ In} = \frac{H\ Q\ W\ S_g\ /33{,}000}{HP\ \ pump\ curve} \tag{9}$$

where W is the weight of water (8.33 lbs/gallon at 68°F), S_g is the specific gravity (approximately 1.0 at 68°F), and 33,000 is the factor for foot pounds per minute. For this analysis, an e_p value of 0.80 was used.

Similarly, the efficiency of the motor (e_m) is less than 100% although not generally as variable as the pump efficiency. As an approximation based on a review of several web-based documents, a value of 0.85 was used for motor efficiency.

To complete the calculation in equation (8), a pump curve from a specific pump is needed. This involves making an assumption on the type of pumps that will be installed. Deep well vertical turbine pumps are commonly installed. These pumps are centrifugal pumps comprised of a bowl assembly, a column and shaft assembly, a discharge assembly, and a driver (motor). These types of pumps have a useful life of approximately 15 years and efficiencies in the 75-85 percent range (Bankston and Baker, 1994).

Using this information and Equation (8), daily pumping costs ($/day) were estimated for the NR4 site and summarized in Table 40 assuming 18-inch extraction and injection wells. The trade-off between pipe size (e.g. construction cost) and pumping cost (operation) is clearly evident. For example, 100 ft^3/s through a 48-inch pipe costs $17,262 per day while increasing the diameter to 60 inches reduces the pumping cost to $10,216 per day. Since 100 ft^3/s is equivalent to 198 acre-ft/day, the energy cost per acre-ft changes from approximately $87 to $52. However, this will be replaced by additional up front construction costs.

Table 40. Daily pumping costs for 18 inch wells from LPO well field to NR4 ($/day).

Pipe Diameter (inches)	Flow Rate to NR4 Location (ft^3/s)						
	25	50	75	100	150	200	300
24	$7,626.						
30	$3,671.	$17,848.					
36	$2,486.	$9,295.	$23,978.				
48	$1,862.	$4,788.	$9,648.	$17,262.	$43,739.		
60			$6,551.	$10,216.	$21,341.	$39,149.	$102,175.
72					$14,643.	$23,912.	$53,746.

One of the assumptions built into Table 40 is that both extraction and injection wells are constructed with 18-inch diameters. To illustrate the sensitivity of these assumptions, variations in well diameters were tested and are presented in Table 41. The example is for a 60-inch pipeline carrying 100 ft^3/s from the Lake Pend Oreille well field to NR4. The differences between the 12 and 18 inch diameters are significant and likely worth the added construction costs depending on the number of days pumped each year.

Table 41. Sensitivity of pumping costs versus extraction and injection well diameter for 60-inch pipeline from LPO to NR4 site.

Well	Well Diameter (inches)	Pumping Costs ($/day)	Change from Base Case
Extraction Injection	18 18	$10,216/day	--
Extraction Injection	12 18	$11,912/day	+$1,696/day
Extraction Injection	24 18	$9,896/day	-$320/day
Extraction Injection	18 12	$11,165/day	+$949/day
Extraction Injection	18 24	$10,102/day	-$114/day
Extraction Injection	12 12	$12,860/day	+$2,644/day
Extraction Injection	24 24	$9,782/day	-$434/day

Comparisons of NR4 pumping costs to the NR2, NR3, NR5, NR6, and PL4 sites were also investigated using several flow scenarios. Assuming 18-inch diameter extraction and injections wells, the pumping costs for these locations are summarized in Table 42. Trade-offs between injection rate, location, pipeline construction cost, pumping costs, and the flow returning to the Spokane River are presented later in this chapter.

Pumping costs from the Spokane River to the injection sites are typically much lower than those from the Lake Pend Oreille well field due to the much shorter distances and the lack of static lift required to extract the groundwater from the aquifer. Also, the pipeline route is moving up gradient so PL5 is closer to the diversion point (shorter pipeline) than PL4. Assuming 18-inch diameter injections wells, the pumping costs for these locations are summarized in Table 43. Head losses through the water treatment plant were not factored into this analysis.

Table 42. Daily pumping costs at alternative injection sites from LPO well field.

Pipe Diameter (inches)	Site	Flow Rate to Location (ft^3/s)					
		25	75	100	150	200	300
48	NR3	$1,840		$15,155			
	NR5			$18,664			
	PL4		$9,563	$17,057	$43,084		
60	NR2					$28,908	
	NR3			$9,549	$19,111	$33,989	$85,548
	NR5		$6,753	$10,681	$22,841	$42,566	$113,080
	NR6			$10,945	$23,717		
	PL4		$6,521	$10,148	$21,122	$38,647	$100,579
72	NR2					$19,845	$40,398
	NR3				$13,774	$21,867	$47,020
	NR5				$15,241	$25,304	$58,212
	NR6				$15,581	$26,111	$60,851
	PL4				$14,544	$23,705	$53,091

Table 43. Daily pumping costs at alternative injection sites from Spokane River diversion.

Pipe Diameter (inches)	Site	Flow Rate to Location (ft^3/s)						
		25	50	75	100	150	200	300
36	PL3	$521	$3,511	$10,994				
	PL4	$346	$2,275	$7,086	$15,974			
	PL5	$165	$1,045	$3,134	$7,029			
48	PL3			$2,818	$6,263	$19,581	$44,204	
	PL4		$625	$1,843	$4,064	$12,629	$28,442	
	PL5				$1,828	$5,593	$12,526	
60	PL3				$2,242	$6,797	$15,165	$47,552
	PL4				$1,485	$4,431	$9,821	$30,635
	PL5				$701	$2,014	$4,395	$13,535
72	PL3							$19,911
	PL4					$1,974	$4,244	$12,910
	PL5					$948	$1,973	$5,834

7.5 Total System Cost

An EXCEL® spreadsheet was created to analyze the various options on an Equivalent Uniform Annual Cost (EUAC) basis. This procedure examined the one-time construction costs and the long-term operation and maintenance costs to determine the total cost per unit of water delivered. The spreadsheet calculated the present worth of cost (see EXCEL PV command) of the O&M costs, added the construction costs, and then calculated the EUAC. The analysis assumed a 30-year design life for the pipeline and a 4.5% State of Washington tax-exempt general obligation bond rate. The bond rate was slightly higher than the 4.23% 30-year bond rate the State received last year reflecting uncertainty in future interest rate direction.

There are several ways to measure flow benefits; 1) total amount of water stored, 2) change in instantaneous stream flow at a point, and 3) total inflow volume over some finite period of time. Since the SVRP aquifer is fairly deep beneath the surface, most additional water that is stored will eventually drain to the river or Long Lake unless captured by groundwater withdrawals in the system. Therefore, the first method of looking at total stored volume has some merit. However, water returning in winter months or peak flow months may not have the same value as water in summer and early fall periods. In the extreme, this requires knowledge of the return flows on a daily basis. While this is beyond the capability of the bi-state Visual MODFLOW-2009 model, it is possible to examine river flow changes in a specific month (e.g., August) in which case the second method can be used. Finally, since there are downstream reservoirs in the system, flows over a longer period may be useful for long-term augmentation. The third method sums several monthly inflow amounts (e.g., August, September, and October) to look at a total volume.

Because of the uncertainty regarding which water uses this project would ultimately be used to satisfy, we elected to report results based on all three of these methods.

<u>*Total amount of water stored*</u>

Conservation principles require that an overall water balance be maintained so any water injected into the aquifer would either build up storage or drain to the outlet. As a result, it is beneficial to look at the total storage amount as one metric. If the only factor being considered was the total amount of water stored, however, then the solution would favor the shortest pipeline due to reduced construction and pumping costs regardless of whether the water flowed back towards Lake Pend Oreille or discharged into the Spokane River in January. Therefore, it is important that this is not the only evaluation conducted. Nevertheless, the information is useful for comparison purposes and is discussed below.

A total of thirty-two (32) scenarios originating from the Lake Pend Oreille well field and twenty-two (22) from the Spokane River direct diversion were examined in this analysis. Scenario variations consisted of different pipeline diameter, injection location, and pumping rate combinations. The various combinations are shown in Table 44. The short-hand designations are written as origination - injection location - pipeline diameter-injection well diameter – discharge. For example, the first entry of LPO-NR4-24-18-25 means: pumping from the Lake Pend Oreille well field, injecting at the NR4 location, with a 24-inch distribution line, an 18-inch injection well, and a discharge of 25 ft^3/s. The base case runs were conducted assuming 30 days of pumping and injection well diameters of 18 inches. The price is greatly impacted by the number of operational days so combinations of different pumping durations were also included in the analysis. As a result, several of the scenarios have multiple options.

The range of annualized base costs for the Lake Pend Oreille base case (30 day pumping) option was from \$679 to \$2,663 per acre-foot stored as illustrated in Figure 87. The economy of scale is clearly evident in that the first three data points represent 25, 50, and 75 ft^3/s options which are much higher than the 100 to 300 ft^3/s options. The average cost of all 32 options was \$1,429 per acre-foot stored. The least expensive options occur at the NR2 injection location which was the shortest pipeline distance examined due to return flow considerations. The \$679 value was for the LPO-NR2-72-18-300 scenario. Costs increased in the downstream direction away from the well field with a \$929 value for the comparable LPO-NR3-72-18-300 scenario. Excluding the NR2 and NR3 locations, the lowest cost was \$1,095 per acre-foot stored for the LPO-PL4-72-18-300 scenario.

In comparison, the lowest cost option for the Spokane River diversion was \$1,905 per acre-foot stored (scenario SR-PL5-72-18-300). The shorter pipeline distance and reduced pumping costs could not overcome the water treatment costs for a 30 day pumping period.

Table 44. Lake Pend Oreille well field and Spokane River direct diversion scenarios.

Full Cost Scenarios		
LPO-NR4-24-18-25	LPO-NR5-48-18-100	LPO-NR3-72-18-200
LPO-NR4-30-18-50	LPO-PL4-48-18-100	LPO-NR5-72-18-200
LPO-NR4-36-18-75	LPO-NR3-60-18-100	LPO-PL4-72-18-200
LPO-NR4-48-18-100	LPO-NR5-60-18-100	LPO-NR3-72-18-300
LPO-NR4-60-18-100	LPO-PL4-60-18-100	LPO-NR5-72-18-300
LPO-NR4-60-18-200	LPO-NR3-60-18-200	LPO-PL4-72-18-300
LPO-NR4-72-18-200	LPO-NR5-60-18-200	LPO-NR4-60-12-100
LPO-NR4-60-18-300	LPO-PL4-60-18-200	LPO-NR4-60-24-100
LPO-NR4-72-18-300	LPO-NR3-60-18-300	LPO-NR2-60-18-200
LPO-NR3-48-18-100	LPO-NR5-60-18-300	LPO-NR2-72-18-200
	LPO-PL4-60-18-300	LPO-NR2-72-18-300
SR-PL5-36-18-100	SR-PL3-48-18-200	SR-PL5-72-18-200
SR-PL5-48-18-100	SR-PL3-60-18-200	SR-PL3-60-18-300
SR-PL5-60-18-100	SR-PL4-48-18-200	SR-PL3-72-18-300
SR-PL4-36-18-100	SR-PL4-60-18-200	SR-PL4-60-18-300
SR-PL4-48-18-100	SR-PL4-72-18-200	SR-PL4-72-18-300
SR-PL4-60-18-100	SR-PL5-48-18-200	SR-PL5-60-18-300
SR-PL3-48-18-100	SR-PL5-60-18-200	SR-PL5-72-18-300
SR-PL3-60-18-100		

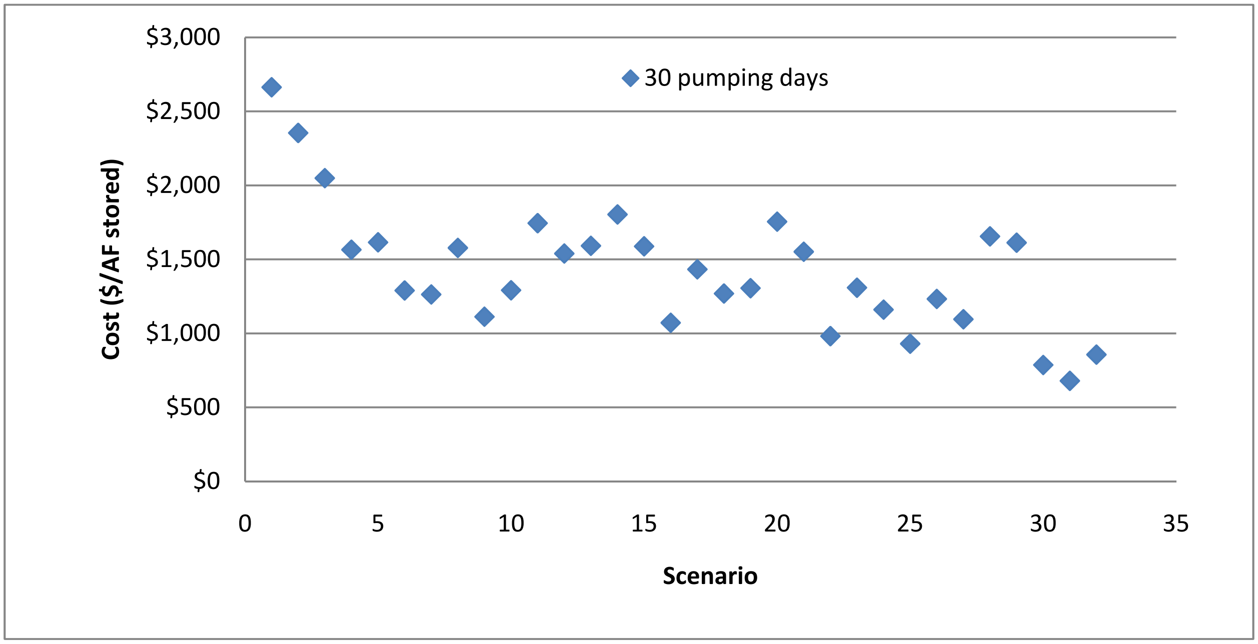

Figure 87. Cost of Lake Pend Oreille alternatives.

The economic impact of three injection well diameters (12, 18 and 24 inches) was examined but overall results were not particularly sensitive to this parameter. The three similar Lake Pend Oreille well field options used to examine the economic impact of injection well diameter (LPO-NR4-60-12-100, LPO-NR4-60-18-100, and LPO-NR4-60-24-100) cost $1,655, $1,615 and $1,612 per stored acre-foot, respectively. The price break, albeit quite modest, occurs between the 12-inch and 18-inch diameters. We decided to focus our analyses on the 18-inch diameter injection wells.

Because the infrastructure costs are fixed once a system is in place, a significant economy of scale exists among all of the scenarios. Using the LPO-NR3-72-18-300 option as an example, Figure 88 illustrates the decrease in cost as the pumping period is extended from one month (30 days) to four months (120 days). If the facility can be used for 4 months, then the cost per stored acre-foot is $292 compared to $929 for the 30 day scenario. At the 300 ft^3/s flow rate, the 120 day duration would result in approximately 71,400 acre-feet of water being stored in the SVRP aquifer.

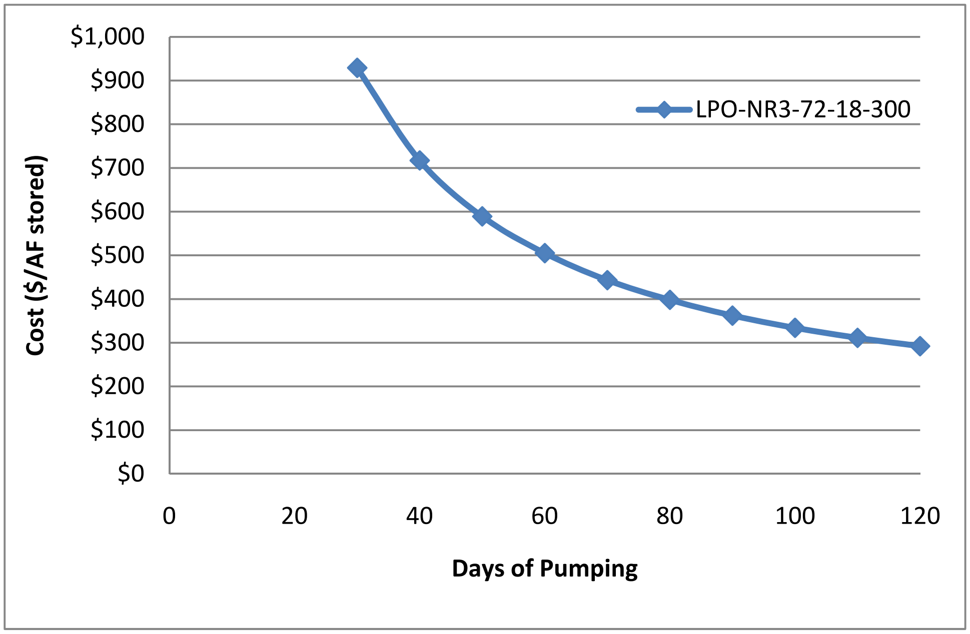

Figure 88. Project water cost as a function of pumping period.

A similar decrease can be seen at the Spokane River facility. The cost of water for the SR-PL5-72-18-300 scenario is reduced from $1,905 to $484 per acre-foot stored if 300 ft^3/s is diverted for a four month period of time.

The peak summer return flow increases the closer the injection site is to the Spokane River as the water has less opportunity to spread out across the SVRP aquifer. However, both the construction and O&M costs of extending the pipeline increase and the so the water may become more expensive if the increase in flow doesn't adequately offset the additional costs. The goal is not necessarily to find the least expensive water but rather to find the least cost option for providing the amount of water needed.

Change in average August stream flow and total inflow volume over August-October time frame

The value of water from the SVRP ASR project is a function of the timing of the return flow to the Spokane River. The costs per August acre-foot of water or August through October acre-foot total water return are considerably higher than the stored (injected) cost per acre-foot. For example, the LPO-NR4-24-18-25, 4 month injection running April through July cost per stored acre-foot is $771 compared to the $9,655 per August acre-foot of return flow or the $3,582 per August-October acre-foot of return flow. Table 45 summarizes the annual cost of water for several of the low flow (25-75 ft^3/s) scenarios. The rates per acre-foot shown in the table appear quite high even for municipal water. Recall that 1 acre-foot is equivalent to 325,800 gallons so $9324/AF is $28.60 per 1000 gallons. If storage reservoirs downstream could be operated in such a way as to extend the beneficial use period beyond October or earlier in June or July, then the costs per acre-foot could be significantly reduced.

Table 45. Low-flow cost comparison for diversions initiating at LPO well field.

	Injection Rate					
	25 ft^3/s		50 ft^3/s		75 ft^3/s	
Scenario	Aug	Aug, Sep, & Oct	Aug	Aug, Sep, & Oct	Aug	Aug, Sep, & Oct
LPO-NR4-24-18 April – 1 m	103 $38,463	267 $14,838				
LPO-NR4-24-18 April – 2 m	223 $18,825	580 $7,238				
LPO-NR4-24-18 April – 3 m	350 $12,648	922 $4,801				
LPO-NR4-24-18 April – 4 m	483 $9,655	1302 $3,582				
LPO-NR4-24-18 May – 1 m	120 $33,077	314 $12,641				
LPO-NR4-24-18 May – 2 m	248 $16,928	656 $6,399				
LPO-NR4-24-18 May – 3 m	380 $11,670	1035 $4,285				
LPO-NR4-30-18 May – 1 m			240 $29,259	628 $11,182		
LPO-NR4-30-18 May – 2 m			496 $15,237	1312 $5,460		
LPO-NR4-30-18 May – 3 m			760 $10,672	2071 $3,916		
LPO-NR4-36-18 May – 1 m					360 $25,462	942 $9,731
LPO-NR4-36-18 May – 2 m					743 $13,305	1968 $5,023
LPO-NR4-36-18 May – 3 m					1140 $9,324	3106 $3,422

A common theme evident in Table 45 is that economy of scale is necessary in order for the cost per acre-foot to be manageable. While the three month LPO-NR4-36-18 scenario is still a rather expensive $9,324/acre-foot of August water, it is over 20% cheaper than the three month 25 ft^3/s LPO-NR4-24-18 alternative. With this in mind, we decided to focus efforts on large flows and pipelines. Table 46 summarizes the results for a number of flow/destination/pipeline diameter combinations for flows ranging from 100 to 300 ft^3/s. Similar to the previous table; the results shown here exhibit a strong correlation to pumping duration.

Table 46. High-flow cost comparison for diversions initiating at LPO well field.

	Injection Rate					
	100 ft^3/s		200 ft^3/s		300 ft^3/s	
Scenario	Aug	Aug, Sep, & Oct	Aug	Aug, Sep, & Oct	Aug	Aug, Sep, & Oct
LPO-NR3-48-18 May – 1 m	437 $17,615	1206 $6,383				
LPO-NR3-60-18 May – 1 m			874 $14,628	2411 $5,303	1315 $17,783	3627 $6,448
LPO-NR3-72-18 May – 1 m			874 $13,382	2411 $4,851	1315 $12,617	314 $4,574
LPO-NR3-48-18 May – 2 m	864 $9,436	2437 $3,345				
LPO-NR3-60-18 May – 2 m			1727 $7,993	4867 $2,836	2633 $9,856	7429 $3,493
LPO-NR3-72-18 May – 2 m			1727 $7,152	4867 $2,538	2633 $6,855	7429 $2,430
LPO-NR3-48-18 May – 3 m	1267 $6,805	3698 $2,332				
LPO-NR3-60-18 May – 3 m			2529 $5,875	7385 $2,012	3856 $7,418	11266 $2,539
LPO-NR3-72-18 May – 3 m			2529 $5,152	7385 $1,764	3856 $5,059	11266 $1,731
LPO-NR3-48-18 April – 4 m	1663 $5,458	4768 $1,904				
LPO-NR3-60-18 April – 4 m			3321 $4,781	9521 $1,668	5062 $6,158	14527 $2,146
LPO-NR3-72-18 April – 4 m			3321 $4,121	9521 $1,437	5062 $4,132	14527 $1,440

LPO-NR4-72-18 May – 3 m					4624 $5,014	12604 $1,839
LPO-NR5-72-18 May – 3 m					5373 $4,765	13464 $1,902
LPO-PL4-48-18 May – 3 m	1514 $6,748	4131 $2,473				
LPO-NR2-60-18 May – 1 m			788 $12,967	2234 $4,574		
LPO-NR2-72-18 May – 1 m			788 $11,898	2234 $4,197	1185 $10,256	3361 $3,616
LPO-NR2-60-18 May – 2 m			1531 $7,241	4442 $2,496		
LPO-NR2-72-18 May – 2 m			1531 $6,513	4442 $2,245	2337 $5,719	6789 $1,969
LPO-NR2-60-18 April – 3 m			2266 $5,275	6475 $1,846		
LPO-NR2-72-18 April – 3 m			2266 $4,663	6475 $1,632	3460 $4,213	9899 $1,473
LPO-NR2-60-18 April – 4 m			2949 $4,357	8675 $1,481		
LPO-NR2-72-18 April – 4 m			2949 $3,792	8675 $1,289	4573 $3,461	13473 $1,175

The cost per acre-ft of water delivered to the injection site for the LPO-NR2-72-18-300 scenario pumping over a 4 month duration (122 days) beginning in April is $218 compared to $679 for a 30 day pumping period. As illustrated in Table 46, this value escalates significantly when examining the August or August through October return flow benefit periods. This is not suggesting that flows in other times of the year are not beneficial. For example, on average, there is an additional 4,244 acre-feet of water that enters the system in July which may be useful in many years. If July is included in the beneficial use period, then the cost per acre-ft is reduced from $1,175 to $893 for this scenario. Likewise, late June and November flows are enhanced by this project. To help illustrate the potential benefit, the 1996-2005 average monthly Spokane River return flows from the LPO-NR2-72-18-300 scenario with pumping beginning in April each year is presented in

Figure **89**. The values shown are in acre-feet per day. To convert to ft^3/s, a multiplier of approximately 0.5 is used (e.g., 100 AF/day = 50 ft^3/s). The total additional inflow to the Spokane River for this scenario is 44,335 acre-feet. However, as shown in

Figure **89**, although the benefit peaks in August, the flow is spread over the entire year. The total cost per acre-foot is $357 if there are beneficial ways to utilize the flows throughout the year.

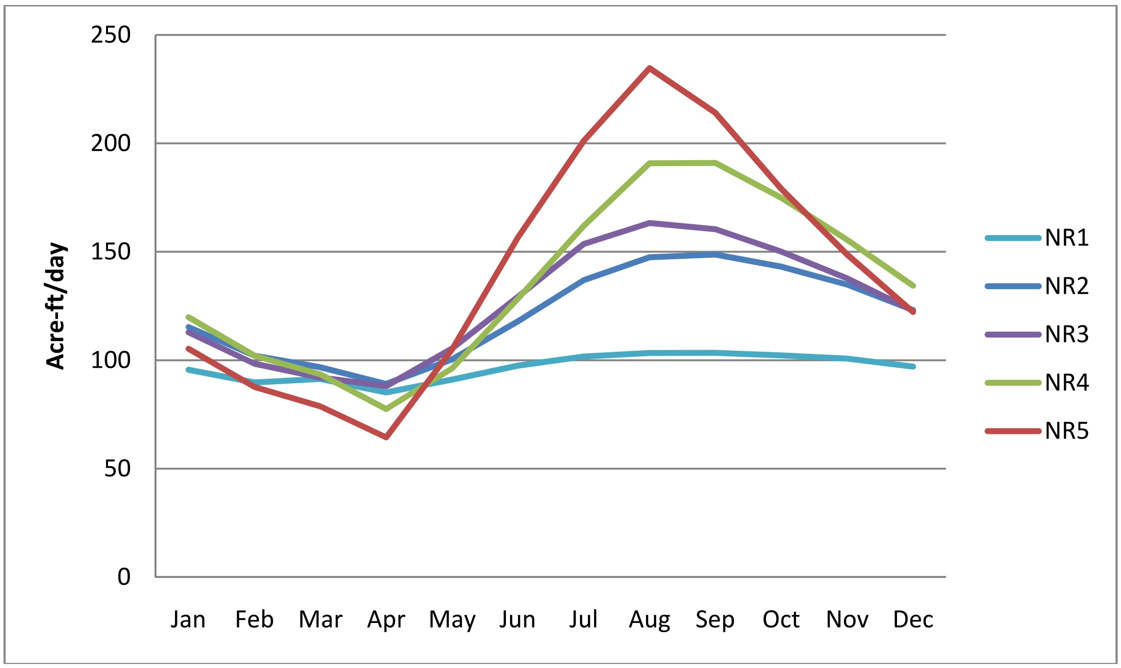

Figure 89. Monthly average Spokane River return flows (1996-2005) for a 4 month, 300 ft^3/s injection at NR1-NR5 sites (for beginning month see Table 8).

As previously discussed, the costs per acre-foot shown above include annualized construction and O&M costs. In general, the construction costs are far greater than the O&M costs. For instance, examining the LPO-NR2-72-18-300 case and a 4 month pumping period, the total construction costs are estimated to be approximately $88,000,000 and the annual O&M costs are nearly $10,500,000. If construction costs were excluded, the annual cost of operation for providing August through October water falls from $1,175 to $366 per acre-foot.

The operation cost increases with the extraction period. Table 47 indicates the number of days used in the calculations based on the extraction period. The construction and operation and maintenance costs for several typical scenarios are presented in

Table **48**. While the O&M costs increase with days of operation, the overall cost per acre-foot of water decreases since the O&M costs are less than the construction costs on an annualized basis.

An important point regarding the trade-off of pipeline versus pump station construction cost is illustrated in

Table **48**. The construction cost estimate of LPO-NR3-60-18-300 is considerably more than LPO-NR3-72-18-300 ($167M versus $122M) in spite of the fact that the pipe diameter is increases by one foot in the latter case. Close inspection of the costs revealed the reason for the apparent discrepancy. While the pipeline cost for the larger diameter pipe does increase by over $10M in LPO-NR3-72-18-300, there is a significant decrease in pumping station requirements as the friction loss between the two pipeline designs decreases from 1,342 feet in the 60 inch pipe to 560 feet in the72 inch pipe. Consequently, since pumping station costs were based on required head (which is 58% less for the larger diameter pipeline), the overall project cost is less expensive. The same phenomenon can be seen at the NR2 site comparing the 200 ft^3/s scenarios with 60 and 72 inch diameter pipes.

A reasonable question might be whether or not there would be additional cost savings going to an even larger diameter pipeline (e.g., 84 or 96 inch diameter). In examining the literature, there are a few projects that specified this size of pipe but the range of costs were too large to provide reliable estimates. A bid estimate for Class 200, C900 PVC pipe reported a price under $50 per foot for the pipe plus installation while the Freeport Regional Water Authority in Sacramento, California constructed a 6.7 mile long 84 inch steel pipeline at a total cost of nearly $58,500,000 ($1,653/foot) including roads, stoplights, and other infrastructure. Since it was not possible to identify pipeline only costs, we elected not to pursue larger diameters at this time.

Table 47. Actual operational days per extraction period.

Extraction Periods	Total Days of Operation
April or June	30
May or July	31
April + May May + June June + July	61
April + May + June	91
May + June + July	92
April + May + June + July	122

Table 48. Total construction and O&M costs.

Scenario	Construction Costs	Annual Operation Cost per Extraction Period (days)			
		30	61	91	122
LPO-NR2-60-18-200	$74,940,477	$5,588,000	$6,485,000	$7,352,000	$8,248,000
LPO-NR2-72-18-200	$70,427,417	$5,032,000	$5,647,000	$6,243,000	$6,858,000
LPO-NR2-72-18-300	$87,630,000	$6,733,000	$7,985,000	$9,197,000	$10,449,000
LPO-NR3-60-18-100	$73,813,000	$4,937,000	$5,233,000	$5,519,000	$5,815,000
LPO-NR3-60-18-300	$166,676,000	$13,067,000	$15,719,000	$18,285,000	$20,937,000
LPO-NR3-72-18-300	$122,041,000	$9,099,000	$10,557,000	$11,967,000	$13,425,000
LPO-NR4-24-18-25	$30,009,000	$2,119,000	$2,356,000	$2,585,000	$2,821,000
LPO-NR4-30-18-50	$52,004,000	$3,812,000	$4,365,000	$4,900,000	$5,454,000
LPO-NR4-48-18-100	$70,697,000	$4,972,000	$5,507,000	$6,025,000	$6,560,000
LPO-NR4-72-18-200	$115,028,000	$7,964,000	$8,705,000	$9,423,000	$10,164,000
LPO-NR4-72-18-300	$146,619,000	$10,849,000	$12,516,000	$14,128,000	$15,794,0000
LPO-NR5-72-18-300	$162,779,000	$12,001,000	$13,806,000	$15,552,000	$17,357,000
LPO-PL4-60-18-300	$198,393,000	$15,516,000	$18,634,000	$21,651,000	$24,769,000
SR-PL3-72-18-300	$464,299,000	$12,391,000	$13,008,000	$13,606,000	$14,223,000

Capital construction and operation & maintenance costs are also limiting factors governing the feasibility of the ASR Project. While the costs of the different alternatives are discussed in this report, it could be that cost limits the rate and scope of an ASR project. It was beyond the scope of this project to determine users' willingness to pay for the water delivered by the project.

8.0 Potential Benefits Analysis

8.1 Additional Water Availability for Economic Growth and Aquatic Habitat Protection

The primary benefits of additional water in the Spokane River are the mitigation potential against further declines in stream flow due to continued growth in the SVRP region and as replacement water for projects downstream. In general, benefit estimates were quantified as additional water supply obtained during any month where demands (or projected demands) are not currently being met. Specifically, we examined the increase in available diversions for withdrawal along the Spokane and Columbia Rivers. In other words, water returning to the system during high flow periods was not considered a benefit.

8.2 Ancillary Benefits

In addition to the direct economic benefits afforded to new diversions along the Spokane and Columbia Rivers, this ASR project would also have ancillary benefits. The flows would augment low streamflows in the Spokane River and thus improve aquatic habitat and recreational opportunities. It was also initially hypothesized that the additional summer groundwater return flows would result in reduced instream temperatures and that the combination of cooler temperatures and additional flow would help reduce algal blooms in Long Lake. Furthermore, most of the additional summer flows would be run through Avista's hydropower facilities on the lower Spokane River when excess generating capacity is available. This chapter examines the potential impacts of these ancillary benefits.

8.2.1 Surface Water Quality Modeling Results

The purpose of this section of the benefits analysis is to analyze the effect of SVRP aquifer injections on phosphorus, nitrogen, and dissolved oxygen concentrations, temperature, and stream flow of the Spokane River at Long Lake. The CE-QUAL-W2 water quality model used by Ecology to help determine dissolved oxygen concentrations and phosphorus limits was modified to include groundwater flow changes and run to evaluate water quality responses

(Moore and Ross, 2010). Other than provide for additional groundwater seepage along the reaches, no changes to boundary conditions, initial condition, reaction rates or any other parameter were made.

There are a number of different CE-QUAL-W2 models created and calibrated for the Spokane River TMDL study (Portland State University, 2008) including four different scenarios (referred to as A, B, C, and D) provided by the Washington State Department of Ecology. Scenario A was created with only the background constituent concentrations. Scenarios B, C, and D incorporated elevated levels of phosphorus and CBOD. Input files for all four scenarios are available from Chris Berger's webpage through Portland State University (2008). However, since the base case (Scenario A) represented existing conditions, this was the only scenario used in the model. Scenario A was modified to include Visual MODFLOW-2009-generated return flows from four distinct pumping periods. One four month pumping period beginning in April was conducted. Four other runs beginning in May and having durations ranging from one to four months were also conducted although the four month pumping in May would extend into August it allowed for direct comparison to the April run. Comparison of the April and May runs showed no significant differences in constituent levels at segment 151 for the duration of the model period thus additional runs with various pumping periods were not conducted. Each period assumed an injection rate into the aquifer of 300 ft^3/s. The injection well field was located at 47°44'58.8"N latitude and 117°00'32.5"W longitude (NR5 – Grid Location 160,120). Output from the Visual MODFLOW-2009 groundwater model was used to alter the groundwater input files provided in the CE-QUAL-W2 surface water model. For this analysis, the modified models were named according to the pumping and injection duration. For example, Scenario A.1 is the original Scenario A input file modified by adding groundwater inflow from the one month (May) pumping period. Similarly, Scenario A.2 is the original Scenario A with two months of pumping (May and June). The water quality model output data focused on streamflow, nitrogen as ammonium, phosphorus as phosphate, dissolved oxygen, and temperature of the water entering Long Lake. All water quality modeling periods lasted from March 15 to October 31 of 2001.

<u>Groundwater Distribution</u>

To change the groundwater input files in the CE-QUAL-W2 model, additional return flows from the aquifer due to the various injection periods were required. The Visual

MODFLOW groundwater model output results are reported as additional monthly average flows out of the aquifer and into the Spokane River. However, because the groundwater flow input files for the CE-QUAL-W2 model require values for each Julian day, the monthly flows needed to be distributed over a daily time step. To distribute the flows, the original total volume of water exchanged between the river and aquifer each day was determined. Values were positive when water moved from the aquifer to the river. Conversely, values were negative when water moved from the river to the aquifer. Thus, the absolute value of this volume was used. This was then divided by the total volume interacting over the whole month to find the percentage of the total monthly volume interacting on that particular day. This percentage was multiplied by the additional volume that occurred on that day due to the injections and added to the original groundwater volume to obtain the new daily groundwater volume. After a time unit conversion, the value was in the correct format for CE-QUAL-W2. Using this method, the additional monthly flow was distributed such that it was proportional to the original water movement between the aquifer and the river.

Zone Overlap

The Spokane River mainstem is broken into twelve branches (reaches) in the original USACE CE-QUAL-W2 model files, starting at Coeur d'Alene Lake outlet at Post Falls Dam and moving downstream to Long Lake Dam. The Visual MODFLOW-2009 groundwater model, however, separates the river into five zones (see Table 49) over the same approximate distance. In order to maintain twelve branches in the CE-QUAL-W2 model, the groundwater flows in the five zones of the VM-2009 model must be properly distributed across the twelve branches. For example, Table 49 shows that 91.67% of the cells in the Sullivan branch of the VM model overlap with branch 4 of the CE-QUAL-W2 model and the remaining 8.33% of the cells overlap branch 5. Each column always adds up to 100% so that each VM-2009 reach is fully distributed over the CE-QUAL-W2 branches. Branches 1, 6, and 7 of the VM-2009 model and branch 12 (Long Lake) of the CE-QUAL-W2 model are not used because they do not overlap each other. The portion (1.56%) of the injections going to Branch 12 of the CE-QUAL-W2 model from Branch 5 of the VM-2009 model is added to Branch 11 because there is no groundwater input file for Long Lake in the CE-QUAL-W2 model. Once all of these percentages are obtained, they can each be multiplied by the total additional volume due to injections for each month, which results in the distributed monthly volume. This total monthly volume is then multiplied by the percentage of the original monthly volume occurring daily (see Groundwater Distribution section for explanation) to find the additional volume due to injections. For CE-QUAL-W2 Branches 5 and 8, corresponding total monthly volumes are added together before multiplying by the percentage of the original monthly volume occurring daily. By adding this number to the original daily volume, the new daily groundwater flow is obtained and can then be updated in the CE-QUAL-W2 model. Figure 90 shows a flowchart of the whole process including distributing the groundwater flows and the zone overlap. The left branch represents data from Visual MODFLOW while the right branch represents data given in the CE-QUAL-W2 model.

Table 49. Visual MODFLOW-2009 zone distribution across CE-QUAL-W2 branches.

		Zones from Visual MODFLOW-2009 Model				
		Greenacres	**Sullivan**	**Spokane Gage**	**Long Lake**	**Post Falls**
		2	**3**	**4**	**5**	**8**
Zones from CE-QUAL-W2 Model	**1**	0	0	0	0	93.94
	2	59.38	0	0	0	6.06
	3	40.63	0	0	0	0
	4	0	91.67	0	0	0
	5	0	8.33	18.37	0	0
	6	0	0	34.69	0	0
	7	0	0	42.86	0	0
	8	0	0	4.08	7.81	0
	9	0	0	0	43.75	0
	10	0	0	0	14.06	0
	11	0	0	0	34.38	0
	12	0	0	0	1.56	0

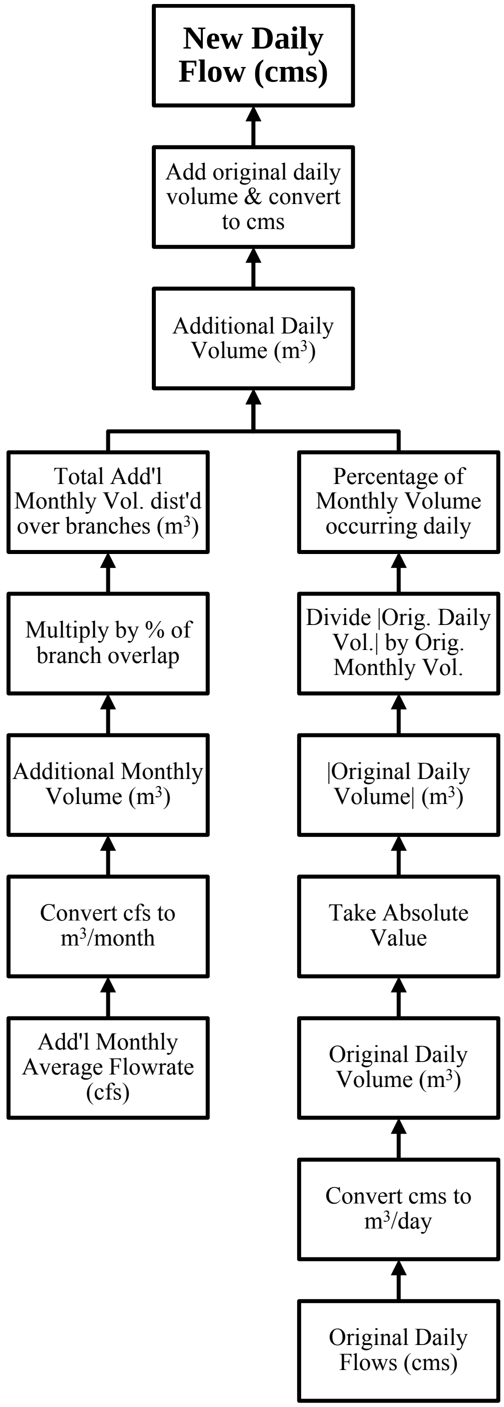

Figure 90. Calculation breakdown for new daily groundwater flows.

Streamflow Results

As a result of the injected water at NR5, discharges in the Spokane River increased. The resulting streamflow is examined at Segment 151 of the CE-QUAL-W2 model, just downstream of Nine Mile Falls Dam at the upstream end of Long Lake. Figure 91 shows the flow values at the beginning of Long Lake for each pumping period when flow corrections are applied to Scenario A. Although the water quality model actually begins on March 15, 2001, Figure 91 begins on June 19 because flows were always identical to the original Scenario A. Until this point, the lag time from the injection well to the river did not result in material changes to early season flows. We recognize that this is an artifact of the relatively-short time period that the original water quality model was operated. However, inclusion of previous year ASR streamflow increases would have required modification of the initial conditions of the TMDL model which we did not want to do. Furthermore, since warm weather-low flow changes were relatively modest and the proportion of the May/June streamflow would be very small, we elected to ignore the previous years' pumping increase.

The water quality model also attempted to use injection well return results from the furthest upstream site (NR1 – Grid Location 207,75) to examine the potential impact of injection well location. The CE-QUAL-W2 model used the return flows generated by the Visual MODFLOW-2009 model assuming a 2-month injection of 300 ft^3/s at the NR1 location. The lag time and back flow to Lake Pend Oreille resulted in no significant change in streamflows during the modeling period. Because this injection well site was the farthest one from the river and showed no difference in streamflow, only NR5, the closest one to the river, was used in the model. Because of its close proximity to the river, injections at NR5 would represent the best case scenario for water quality improvement.

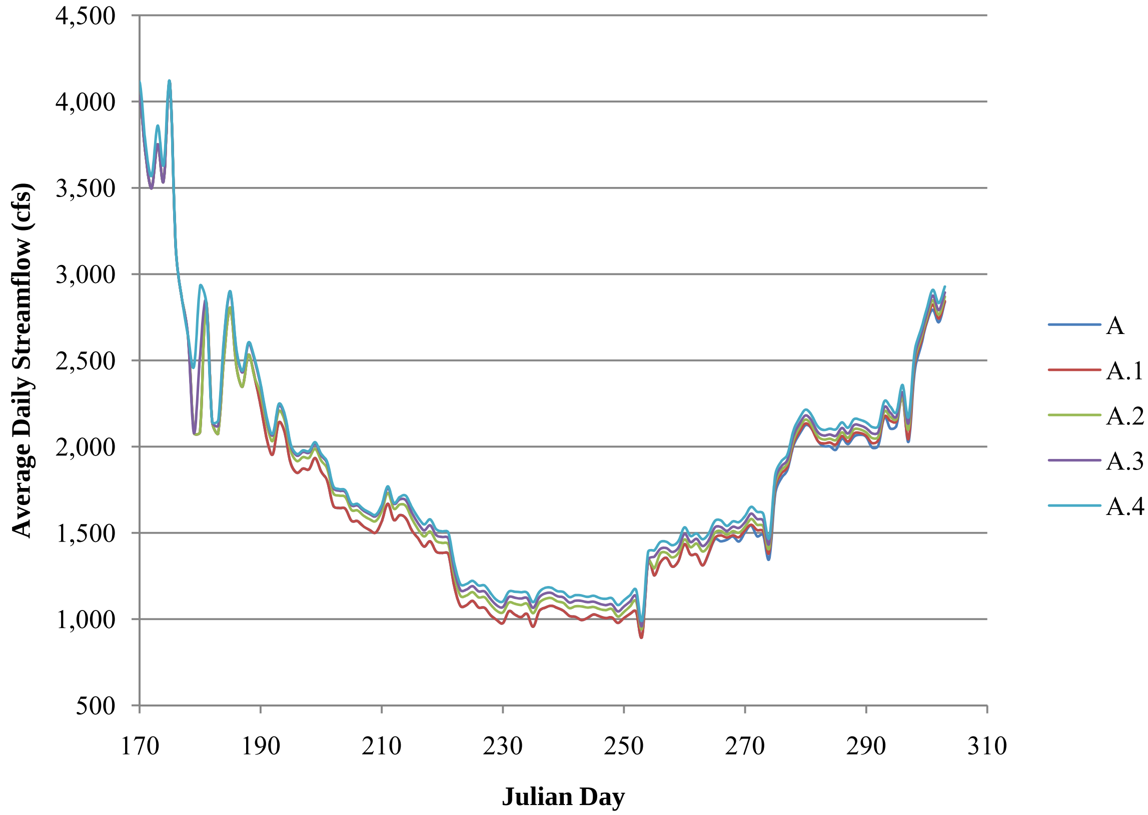

Figure 91. Average daily streamflows at upstream end of Long Lake for each scenario from Julian day 170 to 310 (June 19 to October 31).

Constituent Results

The river constituents observed in this report include dissolved oxygen, phosphorus as phosphate, nitrogen as ammonium, and temperature. All levels are from the upstream end of Long Lake at Segment 151 of the CE-QUAL-W2 model. Average values for each scenario are presented in Table 50. Two other characteristics of these constituents examined were the percent of time during the model period that a certain constituent showed improvement over the original Scenario A results and the average amount by which that constituent was improved upon. These results are shown in Table 51 and Table 52, respectively. Improved phosphorus, nitrogen, or temperature conditions are indicated by decreases in values, while improved dissolved oxygen is indicated by an increase in values. For comparison, Table 53 shows the percent of each model period that constituent concentrations were worsened. Any remaining percentages for each model are attributed to times when constituent concentrations are unchanged.

Table 50. Average constituent concentrations for all scenarios.

Parameter	Scenarios				
	A	A.1	A.2	A.3	A.4
Dissolved Oxygen (mg/L)	9.93	9.77	9.73	9.74	9.74
Phosphorus (mg/L)	0.0030	0.0033	0.0034	0.0035	0.0035
Nitrogen (mg/L)	0.0236	0.0214	0.0211	0.0211	0.0212
Temperature (^oC)	12.51	12.61	12.62	12.59	12.57

Table 51. Average improvement over Scenario A.

Parameter	Scenarios			
	A.1	A.2	A.3	A.4
Dissolved Oxygen (mg/L)	0.19	0.17	0.17	0.18
Phosphorus (mg/L)	0.0004	0.0006	0.0006	0.0007
Nitrogen (mg/L)	0.0060	0.0067	0.0071	0.0074
Temperature (^oC)	0.39	0.41	0.44	0.46

Table 52. Percent of model period constituents showed improvement over Scenario A.

Parameter	Scenarios			
	A.1	A.2	A.3	A.4
Dissolved Oxygen	21	22	23	23
Phosphorus	32	28	28	27
Nitrogen	57	58	59	60
Temperature	42	43	44	45

Table 53. Percent of model period constituents showed decline over Scenario A.

Parameter	Scenarios			
	A.1	A.2	A.3	A.4
Dissolved Oxygen	58	63	65	66
Phosphorus (mg/L)	66	71	71	72
Nitrogen (mg/L)	40	40	39	39
Temperature (^oC)	55	56	55	54

Initially it was assumed that additional water would help the Spokane River. To understand and interpret the results, close examination of the water quality model was necessary. The additional flows entering the river due to injection well returns are represented by increased distributed tributary inflows (groundwater seepage) in the water quality model. These flows are evenly distributed over each vertical layer, and vertical mixing may be induced causing higher concentrations of phosphorus and nitrogen at times. However, perhaps the most significant factor in the given scenarios was that the background groundwater concentrations in each branch of the river were higher than their respective river constituent concentrations. As our goal was not to change the original calibrated TMDL model, the boundary conditions at the stream-aquifer interface remained constant. When the only parameter changed is the groundwater flow (which is always increased), a simple mass balance would reveal that loads to the river increase slightly. Improvement for each constituent tended to occur during the beginning of each day and declined in the late morning throughout the afternoon as photosynthesis and the additional nutrient loading became more of a factor than just increase flow at night. This helps to explain why constituent level improvement varied over the model period and often caused the situation to worsen. Constituent levels could likely improve if the background groundwater concentrations in each branch of the river were lower than their respective river constituent concentrations.

In reality, the characteristics of the water being extracted from near Lake Pend Oreille are likely different than those of the existing water that enters the Spokane River. Whether or not increased flow caused by the injection well would reduce or increase the groundwater concentrations was not part of this study but should be examined in the future. Reactions taking place after reinjection and during the water's travel time to the river are relatively unknown. For this same reason, none of the groundwater temperature files are altered either. It is because of this uncertainty that the groundwater constituent concentrations are not altered from their original Scenario A input values.

8.2.2 Additional Hydroelectric Generation

There are times during low flow months where excess turbine capacity exists at hydroelectric facilities on the Spokane River. At these times, additional flow in the Spokane River due to the injections can be directly translated into energy generation at the five hydroelectric facilities in the study area. These include Upriver, Upper Falls, Monroe Street (not modeled in CE-QUAL-W2), Nine Mile Falls, and Long Lake Dams. Upriver Dam is operated by the City of Spokane while the others are operated by Avista Utilities. Dam specifications were obtained from the Washington State Department of Ecology's Inventory of Dams in the State of Washington (2008). Dams outside the immediate study area, including dams downstream on the Columbia River and two dams on the Spokane River (Post Falls and Little Falls Dams), were excluded from the analysis. Post Falls Dam, the most upstream dams, is above where additional water enters the river and consequently will not benefit from the additional return flows. Little Falls Dam is downstream and could benefit depending on operation of upstream reservoirs which was beyond the scope of this study. Large facilities on the Columbia should also be able to use this water but determining this increase in power generation was beyond the scope of our analysis.

Power, in horsepower generated during a timestep, for each dam was calculated as:

$$P = (\gamma\ Q\ \Delta h\ e)/(550.25) \qquad (10)$$

where γ is the unit weight of water at 50°F [62.4 lb/ft^3], Q is the flow rate through the turbines [ft^3/s], Δh is the height of water above the turbines [feet], and e is the efficiency of the turbine.

Because the height of water above the turbines and the efficiency can vary based on different flow rates and storage, this combined factor was solved for using the flow and power capacities of the turbines. Then, for each new power calculation, this maximum "Δh*E" term was multiplied by the flow rate and unit weight of water before being divided by 550.25 to convert to horsepower.

Once horsepower was calculated and converted to megawatts, energy (E) production in kilowatt hours during a certain timestep was calculated as:

$$E = 1000 * P * t \qquad (11)$$

where P is the power generated during the timestep [MW] and t is the timestep [hrs].

Once the energy production during each timestep was calculated, they were added together over the whole model period to determine the total energy produced. For Upriver, Upper Falls, Nine Mile, and Long Lake, the actual turbine flow was obtained from the given input files for Scenario A to calculate the actual energy produced over the model period in 2001. Since there was no input file for the turbine flow at Monroe Street Dam, the potential flow was used to calculate the energy. Potential flow refers to the water that could have passed through the turbines based on the flow and power capacities of the turbines. The streamflow output file at Division Street Bridge was used for this because it contained the nearest upstream data to Monroe Street Dam. A summary of the average power output is presented in Table 54.

Because potential flow through the turbines at Monroe Street Dam was used to calculate energy production instead of the actual flow, the energy produced by Scenarios A.1 through A.4 is not much higher than the actual energy production (see Table 55). A turbine flow capacity of 2,800 ft^3/s at Monroe Street Dam was used to regulate the river flow upstream of Monroe Street, which resulted in the maximum energy the facility could have generated during the model period rather than the actual energy generated.

Table 54. Average power output (MW) over the model period at each dam in the study area.

Dam	Scenarios				
	A	A.1	A.2	A.3	A.4
Upriver	5.6	6.3	6.3	6.3	6.3
Upper Falls	7.0	7.5	7.6	7.6	7.7
Monroe Street	10.5	10.6	10.7	10.8	10.8
Nine Mile	12.8	13.5	13.6	13.7	13.8
Long Lake	39.5	41.2	41.2	41.3	41.5

Table 55. Energy generation (MWh) for the model period at the five hydroelectricity facilities in the study area.

Dam	Scenarios				
	A	A.1	A.2	A.3	A.4
Upriver	30,902	34,829	34,907	34,974	35,041
Upper Falls	38,626	41,429	41,756	42,081	42,380
Monroe Street[1]	58,100	58,481	58,937	59,383	59,802
Nine Mile	71,151	74,787	75,290	75,719	76,160
Long Lake	219,007	227,681	227,681	228,137	229,358

[1]For Monroe Street Dam the potential flow through the turbines was used instead of actual flow.

The annual power generation at the retail cost of $0.078/kWh for each of the four scenarios is quite significant due to the compounding effect of the five dams. Subtracting the power generation for each scenario from the base case (A) shown in Table 55 and looking only at the incremental generation yields the revenue projections shown in Table 56. This projection assumes that water remains in the river and passes through all five hydroelectric facilities. However, even if this flow is replacement flow for other regional diversions, the amount would represent the offset in lost power production from the new diversions.

Table 56. Incremental power production (MWh) and additional hydropower revenue.

Dam	Scenarios			
	A.1	A.2	A.3	A.4
Upriver	3,927	4,005	4,072	4,139
Upper Falls	2,803	3,130	3,455	3,754
Monroe Street[1]	381	837	1,283	1,702
Nine Mile	3,636	4,139	4,568	5,009
Long Lake	8,674	8,674	9,130	10,351
TOTAL Revenue	$1,510,000	$1,617,000	$1,751,000	$1,941,000

9.0 Conclusions and Recommendations

The technical and economic viability of storing peak season flows in the vicinity of the SVRP aquifer for improving summer low flow conditions in the Spokane River and downstream were examined and documented in this report. The study used a combination of groundwater, pipe flow, and water quality modeling and economic analysis. Cost estimates were derived from local data wherever possible although comparisons to national projects of similar scale were also conducted.

A few key observations:

- Based on the regional MODFLOW model and analyses conducted in this study, it appears to be technically feasible to use Lake Pend Oreille water to enhance SVRP levels to improve flow conditions in the Spokane River.
- The economics of the project improves with volume and pumping period with the lowest cost option resulting from 300 ft^3/s of diversions over a 4 month period.
- It is not viable to extract Spokane River water due to excessive water treatment costs both in terms of treatment, operation, and land acquisition costs.
- Groundwater extractions near the Spokane River caused deficits in aquifer levels that resulted in recirculation of resources rather than additional water so this option was ruled out.
- While all three potential pipeline routes were possible, the paths along the northern pipeline (NR) and power line (PL) routes look the most promising because injection locations were nearer the ground water table.
- Direct injection is preferable to infiltration due to the excessive amounts of land required for an adequate infiltration gallery.
- Benefits for peak summer hydropower generation were identified.
- The impact of additional flows on water quality remains unclear as insufficient data on groundwater concentrations of phosphorus exists and the kinetics of transformation within the SVRP are not well understood.

Preferred Alternative

Because of the acute demand for water in the watershed and the economy of scale demonstrated by the study results, longer pumping with high discharge rates are more cost effective than short pumping periods or low flows. Consequently, there are two alternatives that stand out above other options. Scenarios LPO-NR2-72-18-300 and LPO-NR3-72-18-300 both appear to the best of the options considered in terms of cost per acre-ft delivered to the Spokane River. On average, LPO-NR2-72-18-300 provides 4,573 acre-feet of flow during August and 13,473 acre-feet during August-October for costs of $3,461 and $1,175 per acre-foot, respectively. Likewise, LPO-NR3-72-18-300 provides 5,062 acre-feet of flow during August and 14,527 acre-feet during August-October for costs of $4,132 and $1,440 per acre-foot, respectively. So, while NR3 provides 10.7% more August flow and 7.8% more August-October flow, the costs are 19.4% and 22.6% higher, respectively. While, it is tempting to convert the flow volumes into ft^3/s (e.g., 4,573 AF = 74.4 ft^3/s), the monthly time step makes such conversion somewhat problematic. These values represent average conditions not instantaneous quantities on a specific day.

The region is projected to need this much water and perhaps more to offset growth and climate change issues so the question regarding preferred alternative boils down of the publics' willingness to pay for water. Overall it appears that the LPO-NR2-72-18-300 provides a considerable amount of water (nearly 90% of the NR3 site) at a construction cost of nearly $90 million compared to NR3 site where construction costs are estimated at $122 million. For this reason, it appears that the NR2 site would be the preferred alternative.

A critical component of cost is the assumed beneficial period. Eventually, all of the water injected into the SVRP emerges through wells or groundwater discharges into the Spokane or Little Spokane Rivers. There may be benefits outside the August-October time period that should be accounted for in the analysis. For example, if we recover all of the water injected over a 122 day pumping window at the LPO-NR2-72-18-300 location, the cost per acre-foot drops to $220. Obviously, this is considerably lower than projected costs for water during the low flow period. The value depends on complex storage and operation decisions made downstream.

Another consideration is the distribution route from the extraction field to the injection field. While the northern railroad route was chosen as the alignment, the costs associated with the power line route (PL) are very similar to the NR line. The PL2 site should be located a bit closer to the state line to account for uncertainties in model parameters but the incremental costs should be rather small. More investigation of the geology in the region of the injection location would be required prior to any permanent installation so this concern should be easy to satisfy.

Other routes could work just as well but they would require negotiations with multiple landowners so the process could be lengthy. Model results showed little variation in streamflow as the injection sites were moved north to south along transects of the aquifer. In part this is the monthly stress-period and in part this is the nature of the high hydraulic conductivities.

Potential Next Steps

The plan may need to be revised to account for concerns over right-of-ways, flow, and cost. The feasibility of using existing right-of-ways must be explored in more than a passing conversation with the railroads and power companies. A funding mechanism for both the upfront construction costs and annual O&M costs would have to be devised. Subsurface exploration and assessment of injection and extraction areas needs to be conducted. Engineering design of the extraction, distribution, and injection infrastructure needs to be performed to verify cost estimates. The potential impacts (both positive and negative) of additional groundwater recharge on the quality of discharges seeping back to the river need to be better understood. Water quality studies would have to be conducted at the injection site to make sure water quality standards and treatment assumptions would be satisfied. All of these issues would be addressed in an environmental impact statement analysis that would take place at an appropriate place in the process.

On the modeling front, there are several opportunities to expand the certainty of model predictions. The bi-state model was originally developed with monthly stress-periods over the 1990 to 2005 time period. A more operational model would be developed by recalibrating to a weekly stress-period. Moreover, a considerable amount of new data has been collected since the original model was developed so updating recharge, incorporating future climate conditions, and recalibrating the model would be beneficial.

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