AN ANALYTICAL AND NUMERICAL INVESTIGATION OF STREAM/AQUIFER INTERACTION METHODOLOGIES

By

COLT WILLIAM SHELTON

A thesis submitted in partial fulfillment of
the requirements for the degree of

MASTERS OF SCIENCE IN CIVIL ENGINEERING

WASHINGTON STATE UNIVERSITY
Department of Civil and Environmental Engineering

DECEMBER 2011

To the Faculty of Washington State University:

The members of the Committee appointed to examine the thesis of COLT WILLIAM SHELTON find it satisfactory and recommend that it be accepted.

Michael E. Barber, Ph.D., P.E., Chair

Cara J. Poor, Ph.D., P.E.

Akram Hossain, Ph.D., Md.

ACKNOWLEDGMENTS

This research was funded 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. In completion of this Thesis I would like to thank my advisor Dr. Barber for his help and ability to answer questions that I posed throughout my research experience. His expertise and willingness to challenge me to achieve more than I thought possible was appreciated and increased my understanding of the subject matter beyond course work, and started me into the realms of application. I would like to thank my family for their love and support, especially my wife who has brought me dinner and encouragement each step of the way. Without their support the desire to complete such a project would not have been possible. I would like to thank Patrick Maher for his help in finding surface water level data for the dams on the Spokane River and in understanding that data. I would like to thank John Covert for his help in surveying the gage locations on the SVRP and in finding additional data to help determine elevations of those gages. I would also like to thank the authors of the analytical solutions used; Paul Barlow for his help in obtaining the computer code with the ability to simulate a large number of wells, Vitaly Zlotnik for this help in verifying the equation and solute method that I chose, and Mohamed Hantush for his help in developing a computer program in MathCAD that was able to obtain the solution in his paper. I would like to thank Dr. Akram Hossain for his help in understanding the fine points of MODFLOW and Dr. Cara Poor for comments and revisions that added clarity and focus to this Thesis. Lastly, I would like to thank my Lord and Savior Jesus Christ for the faith, peace, and love that have guided me through this process. “Taste and see that the LORD is good. Oh, the joys of those who trust in Him!” Psalm 34:8

AN ANALYTICAL AND NUMERICAL INVESTIGATION OF STREAM/AQUIFER INTERACTION METHODOLOGIES

Abstract

by Colt William Shelton, M.S.
Washington State University
December 2011

Chair: Michael E. Barber

Groundwater and surface water interaction at the stream/aquifer interface produces both local and regional flow patterns that govern the behavior in many groundwater flow systems. Fluctuations from flood waves cause rapid changes in surface water level and play an important role in the transfer of water from the river to the aquifer. Studying an artificial flood wave from the Post Falls Dam during the low flow summer months in the Spokane Valley Rathdrum Prairie (SVRP) aquifer has provided insights on how the method used to describe the interaction affects the heads and seepage rates produced by the flood wave. A data set capturing six minute intervals as the flood wave passed both in the river and in thirty observation wells in the SVRP aquifer will be used for calibration.

Three analytical solutions and two numerical solutions where used to simulate the changes in groundwater head or seepage or both, from the Spokane River to the SVRP aquifer. The three analytical solutions use convolution integrals to simulate the flood wave and solve head in the aquifer or flow in the river in one or two dimensions. The two numerical solutions also use difference approaches; finite difference and finite element. The former models the non-linear Boussinesq equation and the latter the Richards equation.

The results show that each analytical solution produced different values of the hydraulic conductivity of the riverbed sediments, ranging 1.05-29 ft/day. Due to limitations of the assumptions in the analytical models S_y was allowed to increase only to 0.3. The numerical models also produced different estimates of streambed hydraulic conductivities with MODFLOW values 71-75% lower than HYDRUS values of 1.09 and 0.76 for areas 1 and 2 respectively. These results show that the solution method chosen is important in the resulting calibrated values.

TABLE OF CONTENTS

Page

LIST OF TABLES

Page

LIST OF FIGURES

Page

Chapter 1: Introduction

1.1 Introduction to Aquifer/Stream Interaction

As awareness and understanding of the connection between groundwater and surface water increases, the ability of conceptual and mathematical models to accurately reproduce the complex exchanges between these water resources must also increase. The difficulty in simulating the interaction complexity is caused by temporal differences in groundwater and surface water (GW/SW) propagation phenomena (Konikow and Bredehoeft, 1992; Nemeth and Solo-Gabriele, 2003; Constantz, 2008; and Bunner et al., 2009), spatial scale between GW/SW features (Haitjema et al., 2001; Sophocleous, 2002; Kennedy et al., 2008; and Mehl and Hill, 2010), and the mathematical representation of the interaction mechanism (Rushton and Tomlinson, 1979; Nemeth and Solo-Gabriele, 2003; Konrad, 2006; and Rushton, 2007). Attempts to approximate GW/SW interactions range from relatively straight-forward analytical methods to complex numerical solutions.

Many analytical solutions (Hall and Moench, 1972; Moench et al., 1974; Kabala and Thorne, 1997; and Singh et al., 2002) where developed from analogous heat transfer problems presented by Carslaw and Jaeger (1959). Cooper and Rorabaugh (1963) pioneered the way for stream stage changes to be used in convolution integrals. Their work has been furthered by the studies such as Dever and Cleary, 1979; Guo, 1997; Zlotnik and Huang, 1999; Konrad, 2006 and Akylas and Koussis 2007 to include unsteady-state, two-dimensional, streambed sediments, shallow penetration, longitudinal, and sloping unconfined aquifer effects. Other models (Birkhead and James, 2002; Hantush et al., 2002; and Hantush 2005) have been developed from channel storage functions, such as the Muskingum equation, and use changes in water flow instead of changes in river stage. Understanding of GW/SW interaction has also been furthered with analytical solutions developed by work done with pumping wells near streams (Hantush, 1965; Hunt, 1999; Fox et al., 2002; and Yeh et al., 2008). While analytical solutions often provide reasonable predictions of flow patterns, the boundary conditions and geometry typically imposed to solve the equations are simple and idealized (Walton, 1979 and Tang and Alshawabkeh, 2006). Three solutions that highlight the different methods and incorporate a robust realization of boundary conditions are: Moench and Barlow (2000), Zlotnik and Huang (1999), and Hantush (2005).

A common practice in applying analytical solutions is to use head in an observation well to calibrate the solution (Hall and Moench, 1972; Zlotnik and Huang, 1999; Singh et al., 2002; and Serrano et al., 2007). In the calibration process aquifer hydraulic conductivity, specific yield and streambed conductance factor are changed to determine the best fit (Christensen, 2000 and Fox et al., 2011). The uniqueness of the aquifer parameters determined from the calibration can differ by as much as two orders of magnitude as shown by Fox (2006).

Numerical methods have increased in complexity as computational resources have improved and become more widely available. Numerical models often use a diffusion type equation with many sources and sinks to obtain the influence of the interactions between GW/SW (Wang and Anderson, 1982; McDonlad and Harbaugh, 1988; and Anderson and Woessner, 2002). The MODular three-dimensional finite-difference groundwater FLOW (MODFLOW) model, developed by the U.S. Geological Survey (McDonald and Harbaugh 1988; Harbaugh et al., 2000), is one of the most popular numerical groundwater tools available that explicitly considers GW/SW interaction using a streambed conductance factor, fixed streambed width, and water surface head differences to account for interaction complexities. The streambed conductance factor relies on a linear relationship between river stage and aquifer water levels and has been heralded as too simplistic (Osman and Bruen, 2002; Jackson, 2005; Rushton, 2007; Desilets et al., 2008; Brunner et al., 2009). Increased competition for finite water supplies in watersheds experiencing GW/SW exchanges requires water resources managers to better comprehend the transfer mechanisms. Since many of the approaches taken in describing the interaction between groundwater and surface water are not robust, an in-depth look at the current methodologies used to describe the interaction is warranted.

MODFLOW makes the assumption that all flow is saturated, thus to complete a rigorous study, a method employing variable saturated conditions is needed. Often across a riverbed the assumption of fully saturated conditions is not valid and the use of a variable saturated model allows the streambed conductance factor used in MODFLOW to describe GW/SW interaction to be explored. Also since the geometry of the river is lumped into a single value in MODFLOW a method that enables a detailed look at aquifer/river cross-sections as near as possible to a real river situation is also needed. HYDRUS 2D/3D (Šimůnek and Šenja, 2007) provides such capability and has a graphical user interface which allows control and access to input data.

One problem facing many applications of analytical and numerical solutions is inadequate temporal and spatial data for head and stage measurements in the aquifer and stream (Walton, 1979; Lal, 2000; and Singh et al., 2002). Large increase in streamflow increases errors in streamflow measurements, which makes exact comparisons to analytical and numerical solutions challenging. Also flood events are typically encompassed by large amounts of precipitation or melting leading to difficultly in determine pre-flood baselines. However, the large changes caused by the passing floodwater provide the head and seepage rate differences needed for determining hydraulic parameters like hydraulic conductivity. Small changes can provide local values of hydraulic conductivity but GW/SW interaction on reach scales require larger gradients. Hydropower generation can allow for water levels to fluctuate quickly providing isolated hydraulic events (Arntzen et al., 2006) but do not allow for an equalizing period for a solid starting point. What would be desirable is a reservoir that is held at a constant level and released when climatic changes to aquifer inputs are minimal. In the Spokane Valley Rathdrum Prairie (SVRP) aquifer such a condition exists. At the headwater of the Spokane River, Lake Coeur'd Alene, ID is held relatively constant throughout the summer by Avista's Post Falls Dam and released in early to mid September. In 2005 a unique data set was developed for six-minute time intervals for roughly thirty days before and after the release and at over thirty locations along the Spokane River. The thirty locations in the aquifer are spread over the length of the Spokane River between Post Falls Dam and Nine Mile Dam, which is about 43 river miles. The gages collect the water surface elevation both in the aquifer and the river, which change as the flood wave passes.

1.2 Objective Statement

The overall goal of this thesis is to quantify how the methodology used to conceptualize GW/SW exchanges across streambeds impacts the resulting hydraulic conductivities of the streambed materials, specific yield of the aquifer, and the use of head alone to calibrate analytical models. The bi-state SVRP MODFLOW model developed by Hsieh et al. (2007) will be used to define the initial values for hydraulic conductivity and specific yield and to delineate the aquifer extent and water table where data is not available from the 6-Minute study or the USGS data sets. For most applications, this study will examine stream flux through the river bed materials and the resulting head in the aquifer for an artificial flood wave. The data used will included both a 6-Minute data set developed by the Washington State Department of Ecology (Ecology) from August 16, 2005 to October 21, 2005 (Covert et al., 2005) for the SVRP aquifer, which gives both stream stage changes in the Spokane River and water table elevation changes in the SVRP aquifer and 15-minute data available from the United State Geological Survey (USGS) river gages in the Spokane River, which includes both streamflow and stream stage. Each data set captures an artificial flood wave caused by the opening of the Post Fall Dam in Northern Idaho and subsequent changes in head in the SVRP aquifer. These data sets will be used to compare head values in the aquifer for the different methods and to determine the calibrated hydraulic conductivity of the riverbed materials in the Spokane River.

To quantify how the method used to conceptualize flow across the streambed impacts the resulting hydraulic conductivity of the streambed material and the specific yield of the aquifer; the specific objectives of this study are to:

1- Apply the analytical solutions described by Moench and Barlow (2000), Zlotnik and Haung (1999), and Hantush (2005) to determine a best fit to the data collected in the 6-Minute data set for multiple wells at four locations in the SVRP. The hydraulic properties and seepage rates, for the different conceptualizations of the method of leakage through streambed will also be determined and compared.

2- Evaluate how the conceptualization of an unsaturated zone beneath the Spokane River affects the value of the hydraulic conductivity of the riverbed materials using HYDRUS 2D/3D.

3- Determine accuracy of the regionally approved numerical MODFLOW model of the SVRP aquifer, to capture the artificial flood wave in the Spokane River and evaluate the conceptualization of leakage relationship defined in MODFLOW.

The overall result will provide multiple estimates of hydraulic properties, seepage rates and water volumes, and a detailed look at how different methods of conceptualizing leakage through the streambed affect the resulting flow and head conditions in both aquifer and stream environments.

Using five different conceptualizations of flow across the streambed to determine the hydraulic conductivity of the Spokane River is a unique approach, as most studies use only one conceptualization and vary natural parameters. While many studies could be sited that compared multiple analytical and/or numerical solutions (Spalding and Khaleel, 1991; Sophocleous et al., 1995; and Barlow and Moench, 1998), most of those used an idealized data set and removed the element of natural system complexity. The temporal density of the 6-Minute and 15-minute data available in the SVRP area allows the comparison of the conceptualization with measured hydrographs instead of asymmetric curves. Also unique about the SVRP setting is the high range of hydraulic conductivity (0.01-22,100 ft/day) that aids the assumptions of the five solutions. Natural settings with hydraulic conductivities on the order tens of thousands of feet per day are uncommon and are usually only seen in limestone aquifers that form characteristics similar to pipe flow. Thus the high hydraulic conductivities allows for the limits of the conceptualizations to be tested.

1.3 Outline of Thesis

Chapter 2 contains a literature review of the necessary components of stream-aquifer interaction and the equations used to describe and predict the flux through the riverbed materials. It also includes a complete description of the study area, information regarding the selection of the models, the assumptions each model makes, and a brief look at the descriptive statistic used to identify the best fit line. The methodology used to complete the previously identified objectives is in Chapter 3. Results, analysis, and discussion of the modeled data are presented in Chapter 4. Lastly, Chapter 5 states the conclusions of the research and recommendations for future work.

Chapter 2: Background

2.1 Description and Literature Review of Aquifer/Stream Interaction

Ground and surface water systems have historically been treated as completely separate sources of water in the United States. For example, the State of Washington had water laws governing surface water as early as 1891 (Publication # WR 98-152), while groundwater laws were not enacted until 1945 (Publication # WR 98-152). Interactions between the two systems have played an important role in water rights (Sophocleous et al., 1995) and water quality (Chen and Chen, 2003) in the past and as water resources continue to be stretched quantification of the amount, timing, and pollution will be needed. Awareness of the importance of understanding the interaction between ground and surface water started with the classic works of Glover (1952) and Hantush (1959).

Aquifer-stream interaction happens at multiple scales both in time and space (Toth, 1970 and Schaller and Fan, 2009). Figure 1 shows the three spatial scales with the numbers 1-3 corresponding to the direction of water movement as described by Toth (1963) as away from the water table (1), toward the water table (2), and parallel to the water table surface (3). Interactions between groundwater and surface water are principally controlled by the topography and geology of the field conditions (Woessner, 2000). For this thesis the local flow component is the scale of interest when considering the interactions between aquifers and streams. GW/SW interaction can take on many forms, with aquifers exchanging water with lakes, rivers, wetlands, or oceans. Each can have different flow patterns or different processes that are at work in the chemical and biological cycling of nutrients in their waters (Schwarzenbach and Westall, 1981;

Ward and Stanford, 1995; Sophocleous, 2002; and Alley et al., 2002). The focus of this research is the interaction between aquifers and rivers.

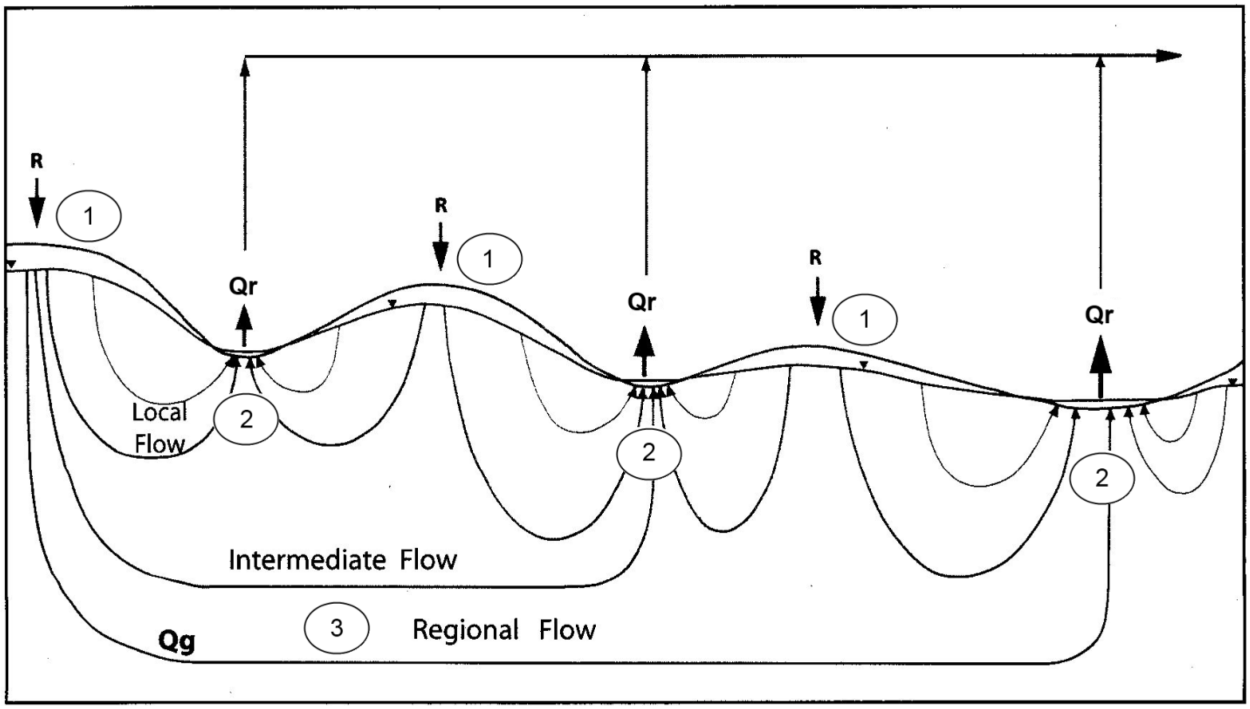


Figure 1. Multiple-scale groundwater flow showing the interaction of local, intermediate, and regional flow scales. Adapted from Schaller and Fan, 2009.

As illustrated in Figure 2, aquifer-stream interaction occurs between a surface water body and an unconfined aquifer or the vadose zone. It is governed by changes in water surface elevations (head) between the two systems and material properties along the interaction interface (Rushton and Tomlinson, 1979 and Nemeth and Solo-Gabriele, 2003). In 1856 Henry Darcy showed how the head difference between two points and a retardation factor (hydraulic conductivity) could mathematically describe flow in groundwater systems. The equation is defined as Darcy's Law:

$$q = -K\frac{\Delta h}{\Delta l} \qquad \text{Equation 1}$$

where K is the hydraulic conductivity (length/time), q is the fluid flow (flow/unit area), *h* is the head (length), and *l* is the length between points of measurement (length). Darcy's Law is valid for natural porous media where the local accelerations in the fluid are much less than the viscous forces, which is often observed at Reynolds numbers greater than 10 (Selker et al., 1999; Mays, 2005). The features associated with the interaction include: the floodplain, stream/river sediments (for the rest of this paper references to stream or river are inclusive of both), river geometry, length of the river reach, surface elevation of the river, and the elevation of the water table in the aquifer. Also included in the interaction are the spatial changes in hydraulic conductivity, both in the channel and the associated flood plain, the position of the river within the floodplain, and temporal changes in river stage and groundwater table elevation. The culmination of those features result in transient nature associated with the exchange between aquifers and rivers (Woessner, 2000).

Aquifer-river interactions can take place in three general categories: 1) water from the aquifer flowing into the stream (Gaining Stream), 2) water from the river flowing into the aquifer (Losing Stream) or, 3) some combination of both (Winter et al., 1998) (See Figure 2). Losing streams can be either connected (fully saturated zone to water table, as shown in Figure 2A) or disconnected (unsaturated zoned between stream and water table, Figure 2C).

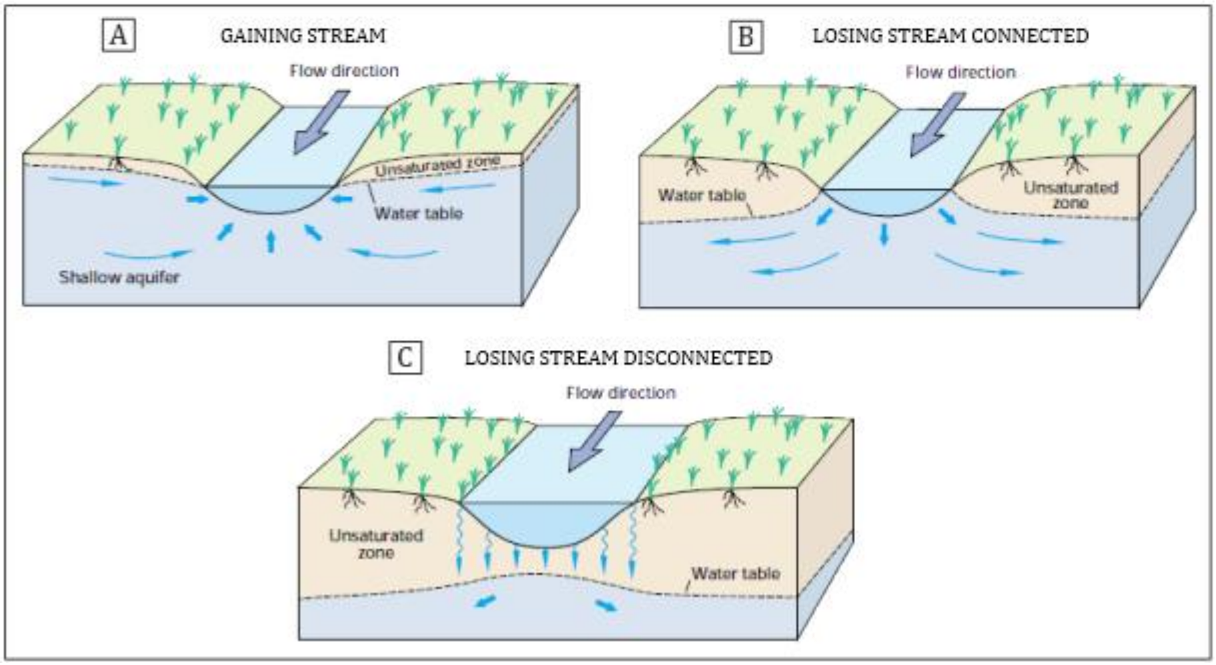


Figure 2. General Conditions for Gaining Losing Streams in an Aquifer, reproduced from Winter et al. (1998).

Aquifer-stream interaction occurs continuously and can transition between gaining and losing depending on the height of the water table. However, due to precipitation, snow melt, or the release of water from a dam, rapid increases or decreases in river stages are seen in most rivers at some point in the year. Flood waves usually have short time durations and can temporarily reverse the direction of flow depending on the groundwater system. This phenomenon is termed “bank storage” (Winter, 1999 and Chen and Chen, 2003). In a gaining reach (aquifer discharging to stream) a passing flood wave can cause water to move from the stream to the aquifer, as seen in Figure 3. This results in a general direction of flow away from the stream. The distance traveled dependents on the height of the flood wave and the duration of flooding. In a losing reach the increased stage in a river typically engages more area in seepage and increases the seepage losses overall. If the flood wave overtops the banks of the stream, water moves out onto the flood plain and vertical infiltration through the floodplain sediments allows for a large increase in seepage to the aquifer, which can take weeks, months or years to return to the river (Winter et al. 1998) depending on the K of the floodplain sediments and the regional flow path.

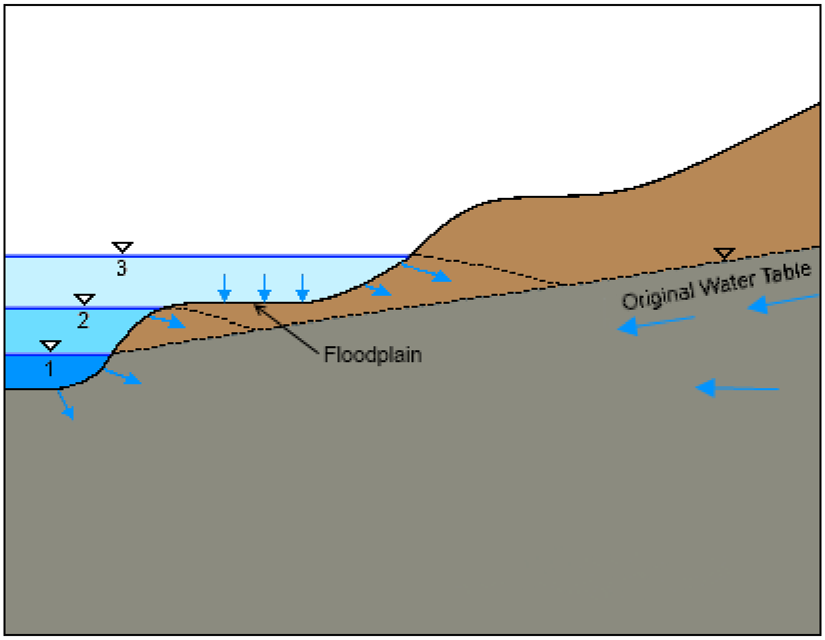


Figure 3. Bank storage from flood wave propagation

The value of K can change depending on the direction of flow through a riverbed. In a laboratory study, Rosenberry and Pitlick (2009) showed changing the direction of flow can have an impact on the value of hydraulic conductivity of the streambed. They found that for sand and sand gravel mixes, an upward seepage (gaining reach of a river) increased the value of K (K_{up}) and a downward seepage (losing reach of a river) decreased the value of K (K_{down}). Reported ratios of (K_{up}/K_{down}) were 1.38 to 1.66 as flow in the surface water increased in velocity from 0-30 cm/s. Their major finding was that the direction bias of K and the changes in ratio of (K_{up}/K_{down}) only occur if fines (that portion passing No. 200 sieve) are present and comprise a lower K riverbed sediment layer. Field studies by Doppler et al. (2007) in Zurinch Switzerland and Blaschke et al. (2003) in Vienna Austria observed similar results.

2.2 Equations Governing Aquifer-Stream Interaction

Having conceptually provided a framework for the way that rivers and aquifers interact, the equations that have been developed describing the interaction are presented. The basic groundwater flow equation will be used as the starting point and the equation describing the flow mechanism between rivers and aquifers will be incorporated into it. Derivation of the diffusion equation is common in groundwater flow literature (Domenico and Schwartz, 1990; Selker et al., 1999; Fetter, 2001; and Mays, 2005) and will not be repeated in detail here. However, a brief overview of how to arrive at the diffusion equation is given.

The diffusion equation can be determined from applying the conservation of mass equation to a control volume, inserting Darcy's Law (Equation 1), and making the assumptions of constant density and that gains or losses in fluid volume are proportional to changes in head (Domenico and Schwartz, 1990). Thus, the diffusion equation can be written as

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right)=S_s \frac{\partial h}{\partial t}-R \qquad \text{Equation 2}$$

where $K_{x,y,z}$ are the component of the hydraulic conductivity in the x, y, and z directions [length/time], h is the head in the system [length], S_s is the specific storage [1/length], t is time, and R is a general sink/source term and defines the volume of inflow to the system per unit volume of aquifer per unit of time.

From this point many different assumptions can be used to rearrange Equation 2 to take the form of the Laplace equation, Boussinesq equation, or a one, two or three dimensional equation for the following conditions: heterogeneous or homogeneous, anisotropic or isotropic, confined or unconfined aquifer, or for steady or transient conditions (Zlotnik and Huang, 1999;

Moench and Barlow, 2000; Anderson and Woessner, 2002; and Hantush, 2005). The Richards equation can also be found using Equation 2 with the modification that the K in Darcy's Law (Equation 2) becomes a function of soil moisture and the soil diffusivity function is used to convert the Richards equation to a diffusion equation (Selker et al., 1999). The specific assumptions used to arrive at the equations in this thesis will be described in the next section.

For the interaction between groundwater and surface water the diffusion equation must incorporate the source/sink term R. This term allows water that is not presently in the aquifer to enter or leave depending on the boundary conditions. Rivers are a type of boundary condition where R takes the form of Darcy's Law (Equation 1), which is commonly used to define the quantity of water that moves between a river and an aquifer (Rushton and Tomlinson, 1979). Generally two scales are used in river-aquifer interactions: local and regional. The local scale details head and flow close to the river as water is transferred in either direction between the aquifer and river. The regional scale looks at the overall value of flux to or from the river and head values in the adjacent flood plain. Additionally there are two approaches to attaining head distribution and flux across the streambed, one from the top down (streamflow routing) and the second from the bottom up (diffusive type equations).

The use of leakage through a semi-pervious layer (Darcy's Law (Equation 1)), similar to flow through an aquitard, results in a linear relationship and can be easily graphed as seen in Figure 4, from the equation:

$$Q = K\frac{(L*W)}{M}*(HRIV - H) \qquad \text{Equation 3}$$

where k is the slope of the line is defined by the hydraulic conductivity of the riverbed sediments [L/T], L is the length of the stretch of river in a given cell [L], w is the average width of stream in stretch of river in a given cell [L], M is the thickness of the riverbed sediments [L], HRIV is head in the river [L], H is and head in the aquifer [L]. The terms K, L, W, and M are often lumped into one value called the riverbed conductance (RC), which is determined through calibration. The present form of Equation 3 describes total flow [L^3/t] into or out of a section of a river (McDonald and Harbaugh, 1988). The analytical models of Moench and Barlow (2000), Zlontik and Huang (1999) and Hantush (2005) use this type of leakage relationship.

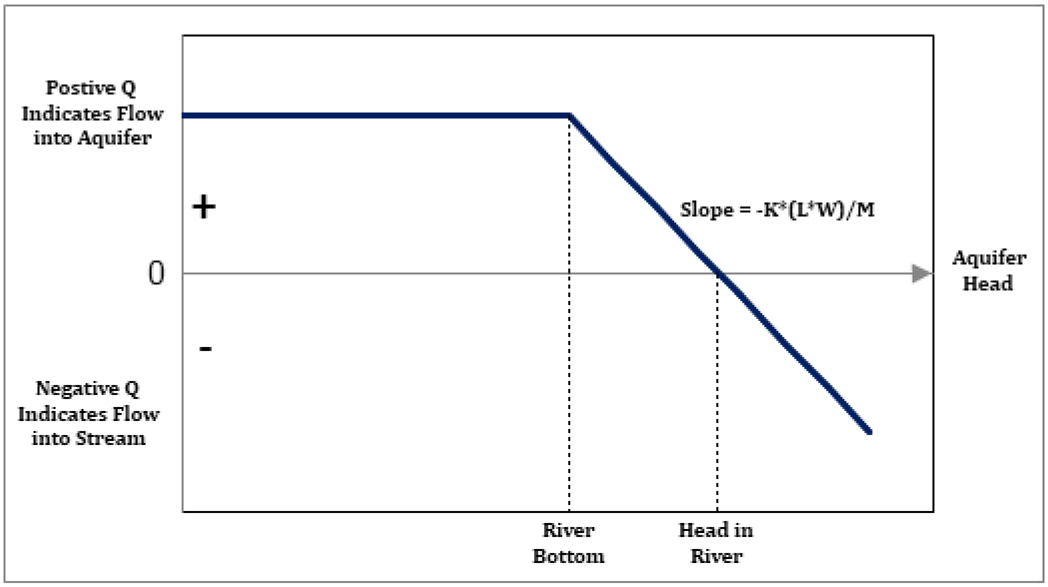


Figure 4. Graph of flow (Q), from stream into underlying aquifer as function of head in the aquifer.

The concept of using a linear relationship as defined by Equation 3 and shown in Figure 4 has made it easy to describe the changes in flux to or from a river. The ability of this relationship to represent field conditions has been explored in many studies (Nemeth and Solo-Gabriele, 2003 and Mehl and Hill, 2010) and two findings stand out: 1) the relationship is too simplistic for saturated flow conditions in the field and 2) unsaturated flow conditions under the riverbed materials cause more infiltration than predicted. A few studies are highlighted here to illustrate the difficulties in assuming a linear relationship.

Rushton and Tomlinson (1979) used a one dimensional saturated model to investigate linear relationship between q and Δh (see Equation 1 for explanation of variables). They found that periods of very high sustained recharge resulted in most of the recharge exiting the groundwater system as leakage to the river. This behavior is typical except that Rushton and Tomlinson (1979) noted no significant delay in the leakage and that build up in heads was seemingly absent. Additionally, total leakage during streamflow recession was largely independent of hydraulic conductivity. Rushton and Tomlinson (1979), therefore, provided a nonlinear relationship in the form:

$$Q = C_1 \Delta H + C_1 \left(1 - e^{-C_2 * \Delta H}\right) \qquad \text{for } \Delta H \geq 0 \qquad \text{Equation 4}$$

$$Q = C_1 \left(e^{C_2 * \Delta H} - 1\right) \qquad \text{for } \Delta H \leq 0 \qquad \text{Equation 5}$$

where C_1 and C_2 are constants that must be determined experimentally.

Rushton (2007) did an in depth study with saturated flow conditions on how changes in the clogging layer (reduce K streambed sediments), channel geometry, permeability of the aquifer bottom, head in the low permeability layer, anisotropy ratio's in the aquifer material, depth of water in river channel, varying recharge in adjacent upland areas, the specified head at a boundary, and a combination of those conditions affect the RC using a fine-grid model. The main results of the study were that a linear relationship existed between surface and groundwater interactions when there was hydraulic connection, ratio between the river channel and the aquifer K are near 1, and when no zone of higher K is present above the bottom of the channel in the aquifer. Rushton (2007) found that in the cases where linear relationship failed, it might still be possible to represent the flux from or to the river using two different RC. However, neither the transition point between them nor a method for finding the value of the RC was determined.

Jackson (2005) and Desilets et al. (2008) both used variably saturated flow models based on Richards equation to look at how the unsaturated zone affects leakage value. Both reported that flow to the aquifer was under predicted by the linear relationship. In the setup of the models a fine grid representation near the river was used, which allowed for more accurate representations of the flow fields and did not invoke the assumption of a linear relationship. Brunner et al. (2009) went further and quantified three conditions involving losing reaches of rivers. In the study he used connected, transition, and disconnected zones to explain where the full disconnection takes place and what value of flux is associated with that condition. Osman and Bruen (2002) showed how the unsaturated conditions beneath a river could be represented by three linear relationships based on water table depth, maximum suction head and the position of the bottom of the riverbed.

Taking the above difficulties into account, it would seem that the linear relationship shown in Equation 3 is best suited for simple geometry and connected streams (See Figure 2), as supported by the study conducted by Rushton (2007). It might be possible, however, to use multiple linear relationships to represent more complex geometries and account for disconnection between the stream and the aquifer when information regarding such complexities is known. This being the case, along with the temporal nature of the RC, great care must be exercised in applying the linear relationship for river-aquifer interactions.

The top down approach lies on the continuity equation in combination with the storage function, as both outflow and storage are generally unknowns. The continuity equation is:

$$\frac{dS}{dt} = I(t) - Q(t) \qquad \text{Equation 6}$$

where S is storage in the river channel [L^3] and I and Q are inflow and outflow [L^3/t] respectively. The storage function can take on many forms and depends on the system being analyzed (Chow et al., 1988). The Muskingum method is one of the common forms used, which presents a linear finite-difference scheme to the partial differentials in Equation 6. The storage function for the Muskingum method is:

$$S(t) = \eta\left[\xi I(t) + (1-\xi)O(t)\right] \qquad \text{Equation 7}$$

where S(t), I(t), and O(t) are as defined above and η and ξ are the storage time constant for the reach [t] and a weighting factor varying between 0 and 0.3 for natural river systems, respectively. Cunge (1969) provides an excellent review of the derivation and application of the Muskingum method. The goal of the approach in Equation 6 and 7 is to determine flow at a downstream gauging location and compare it with measured values. Although most commonly applied to strictly river routing, Equation 6 can be modified to include an additional outflow component, which takes on the form of leakage to or from the aquifer beneath the river. Birkhead and James (2002), Hantush et al. (2002), and Hantush (2005) outline the process. This approach is used in analytical solutions and often is focused on changes in streamflow, and not the mechanism of interaction between rivers and aquifers. It is introduced generally here to demonstrate a different approach to understanding how the interaction is conceptualized; the assumptions and benefits will be explained in Section 2.6.

2.3 Analytical Methods for GW/SW Interaction

Analytical solutions provide exact solutions to the governing differential groundwater equations for simple boundary conditions. These equations are generally used to solve specific aspects of groundwater interaction when various stresses are applied to the aquifer. The door was opened for analytical groundwater solutions in 1935 when aquifer drawdown created by a pumping well was solved by Theis (Theis, 1940). Analytical solutions now abound and can be used to solve infiltration problems (Glover, 1964; Hantush, 1967; Marino, 1975; Rao and Sarma, 1981), well drawdown near streams (Hantush, 1965; Neuman, 1972, 1974, 1981; Christensen 2000; Christensen et al. 2010), and river aquifer interaction (Cooper and Rorabaugh, 1963; Moench and Kisiel, 1970; Hall and Moench, 1972; Moench et al., 1974; Dever and Cleary, 1978; Kabala and Thorne, 1997; Guo, 1997; Zlotnik and Huang, 1999; Harada et al., 1999; Moench and Barlow, 2000; Barlow et al. 2000; Bolster et al., 2001; Singh et al., 2002; Birkhead and James, 2002; Hantush et al., 2002; Anderson, 2003; Hantush, 2005; Anderson 2005; Akylas and Koussis, 2007; Koussis et al., 2007) . These analytical solutions rely on simplifying assumptions in geometry and/or boundary conditions to allow for exact mathematical representations of the problem domain. Information about the groundwater flow system is often revealed at a single point (i.e. an observation well) by solving the analytical equation. When the assumptions are valid, analytical equations have the advantages of requiring less data and can quickly estimate solutions of complex phenomenon. Because of their relative simplicity, analytical models have been widely used to validate more complex numerical models (Sophocleous et al., 1995; Hunt et al., 1998; Bolster et al., 2001; and Chen and Chen, 2003). The assumptions made in analytical equations will be looked at in detail in Section 2.6.

2.4 Numerical Methods.

Numerical methods are split into finite difference and finite element methods. Both systems use nodes to solve for the head at a position in the aquifer based on the nodes that surround the node of interest. These nodes are positioned so that they occupy the same region as the problem domain. Finite difference models split the problem domain into a rectangular grid which is then solved either at the center of the node (block-centered nodes) or at the corner of each rectangle (mesh-centered nodes). Finite element models split the problem domain into triangle elements that have nodes at the corner of each triangle. Numerical models are better able to handle complex boundary conditions and multiple types of interactions then there analytical counterparts. The governing equation for the model is derived by looking at flow into and out of a representative elementary volume (REV) and including terms for sources and sinks (i.e., a well or river). The resulting general form of the equation is presented in Chapter 2.2, Equation 2. The use of the numerical method linearized the solution of the governing equation through multiplying by the saturated thickness of the aquifer and using a current (known) value of saturated thickness during an iteratively numerical scheme (Anderson and Woessner, 2002). Since the 1960's with the availability of high-speed computer, numerical methods have become an important method to study and solve groundwater problems (Wang and Anderson, 1982). Anderson and Woessner (2002), Wang and Anderson (1982), Fetter (2001), or Mays (2005) provide additional information on the implementation of the numerical scheme, the conceptual model and grid design, boundary conditions, sources and sinks, or model calibration.

Many numerical models are commercially available (Visual MODFLOW, MODHMS, Groundwater Vistas, Goundwater Modeling System) and some open source models such as MODFLOW are also available. MODFLOW uses multiple packages to describe various parts of the groundwater flow system, giving it flexibility as new packages become available (McDonald and Harbaugh, 1988).

2.5 Description of the Study Area.

The Spokane Valley Rathdrum Prairie aquifer (SVRP) is located in the Northwest region of the United States and is positioned across a section of the mid northern boundaries of Idaho and Washington (Figure 5). The bi-state nature of the aquifer plays heavily into political and social pressures to manage water resources, but does not affect the physical response of the aquifer to natural or anthropogenic stresses. As of 1978, the SVRP was designated as a "Sole Source Aquifer" (Federal Register, Vol 43, NO. 28 – Thursday, February 9, 1978), which signifies that at least 50 percent of the drinking water in the area is supplied by the aquifer, with no alternatives that could fill the same need. Thus, yearly declining low flow trends have sparked an interest in possible methods to ensure the sustainability of the aquifer. Many studies have been completed in the SVRP, providing much of the data used in this study.

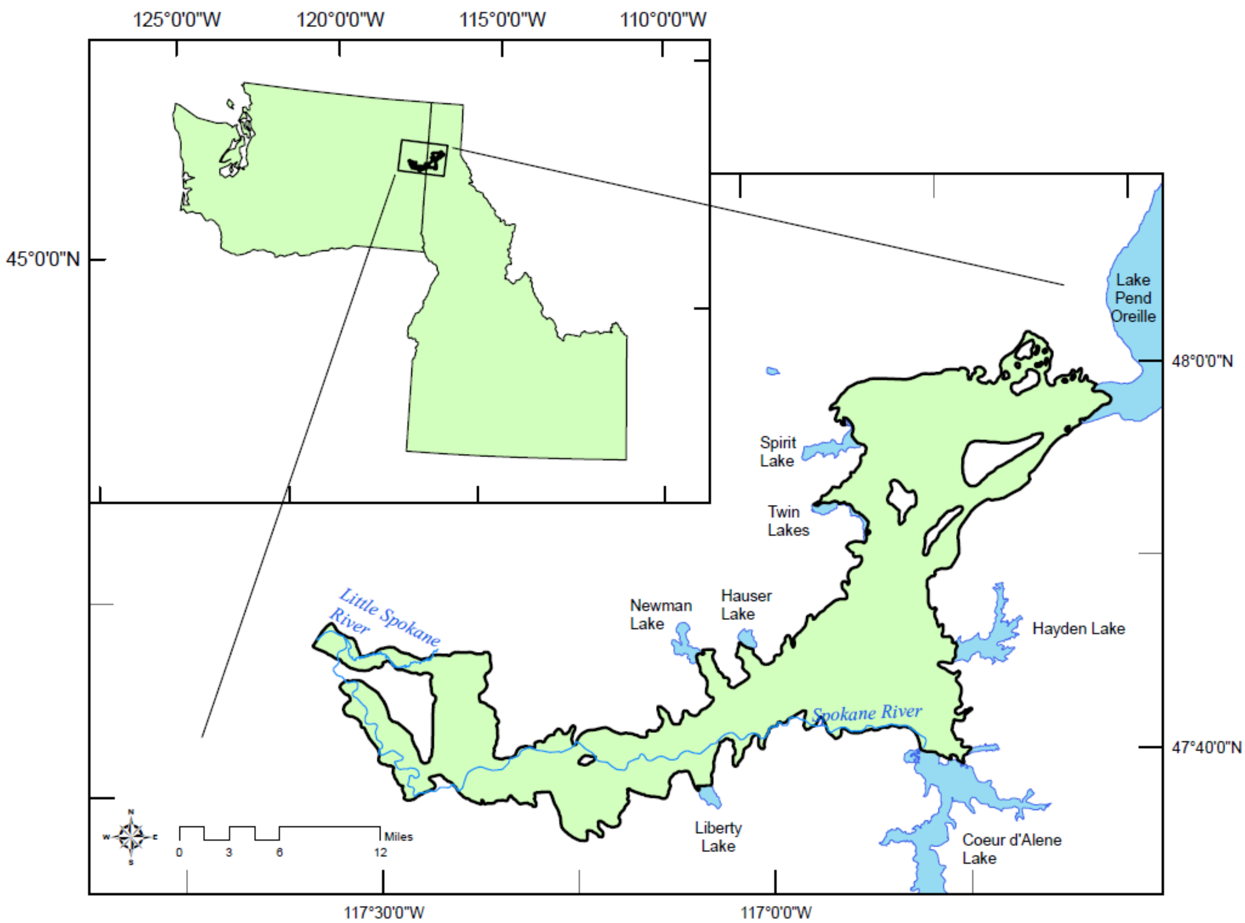


Figure 5. Location Map of the SVRP Aquifer

The SVRP aquifer encompasses a total area of 326 mi^2, with nine lakes that touch the boundary of it. The two largest lakes are Lake Pend Oreille and Lake Coeur d'Alene. Lake Coeur d'Alene's outlet is the Spokane River. The smaller lakes do not have perennial outlet streams, but during the spring runoff periods flow over the outlet structures is common. The Spokane River is the main drainage in the aquifer and is regulated by Post Falls Dam, which serves as the outlet control of Lake Coeur d'Alene. Details of the SVRP aquifer are provided by Hsieh et al. (2007).

The climate of the SVRP aquifer is semi-arid, with most of the precipitation falling between October and March with about half in the form of snow (Bartolino, 2007). Spring runoff peaks in May and recedes to low flow in September. There are four defined seasons with mean lowest to highest temperature ranges from 18°F in the winter month of January to 80°F in summer month July (Bartolino, 2007). Mean annual precipitation is about 18 inches in the western portion of the aquifer near Spokane, WA (Stormwater Management Manual for Eastern Washington, 2004) and about 30 inches in the Northeastern portion by Athol (The PRISM Group at Oregon State University, Idaho Department of Water Resources, 2006). The period used in this research was the low flow period around September.

The groundwater flow gradient of the aquifer varies significantly from the surface topography. Topography is very steep on the surface, with a change is elevation of 3000 feet in some areas. The aquifer is characterized by very flat gradients from the Northeast (Lake Pend Oreille) to the West (Spokane). The aquifer is very deep in some areas (Figure 6) with a large saturated thickness. The hinge line (location of transition from a predominately losing river to a gaining river (Larkin and Sharp (1992))) for the SVRP is located west of the Washington Idaho State Line (Figure 6). The flow mostly follows the regional flow path conceptualization done by Toth (1963), until it reaches the hinge line after which local and intermediate flows also contribute to the overall flow to and from the Spokane River.

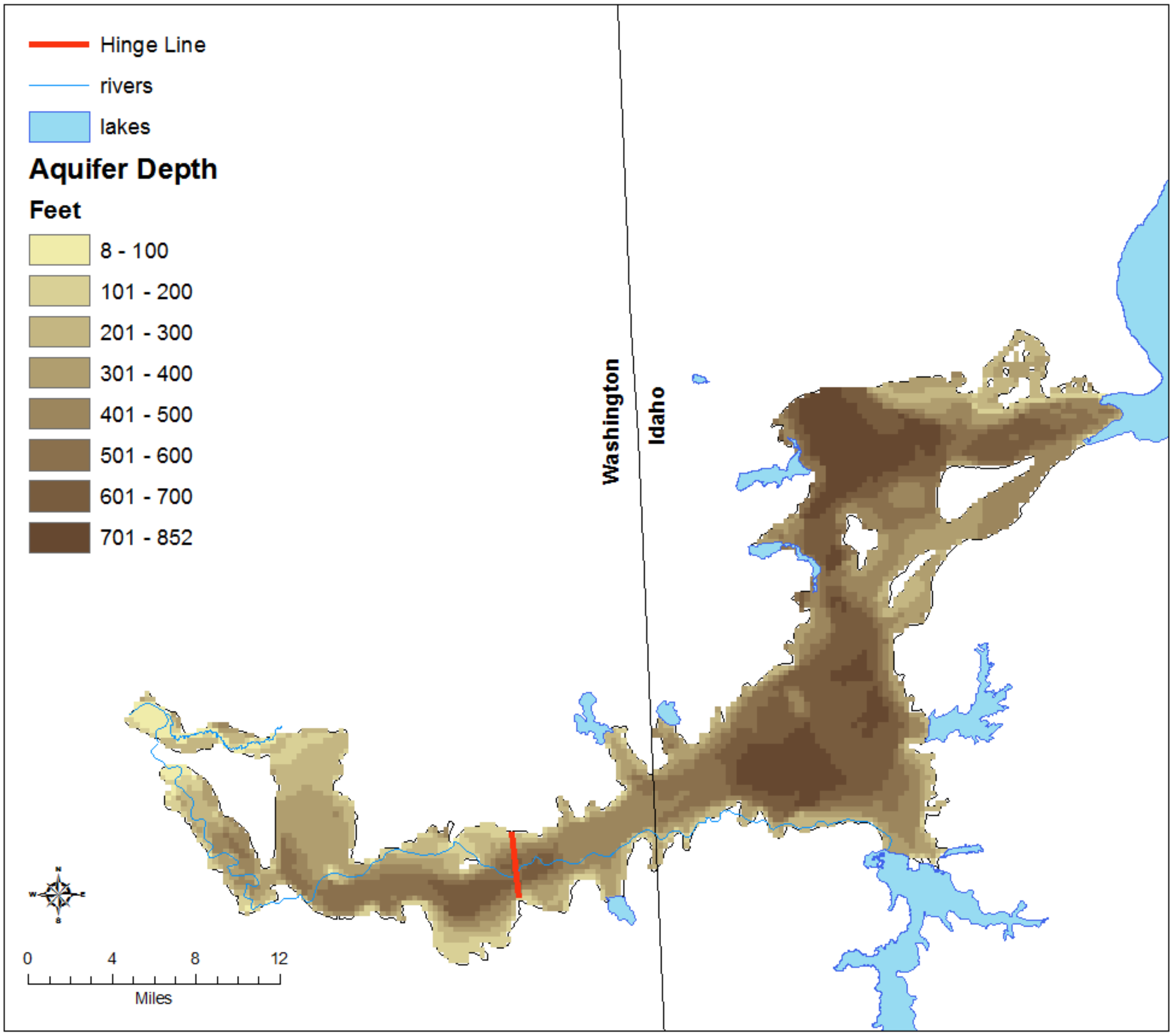


Figure 6. Depth from the Ground Surface to Bed Rock (data from Hsieh et al., 2007)

The geology of the aquifer and surrounding area are described in detail by Kahle et al. (2005). The porous media in the aquifer includes course sands, gravels, cobbles, and boulders underlain by a bed rock layer. Thus the SVRP aquifer is characterized as an unconfined aquifer. The course soils of the aquifer lead to high hydraulic conductivities, with a range in the SVRP aquifer from 5 to 22,100 feet per day (Hsieh et al., 2007). The extremely high hydraulic conductivity values through the majority of the aquifer allow for water to move through almost freely. The saturated portion of the SVRP varies from over 500 feet in depth to less than 25 feet in depth, with the majority of the aquifer over 200 feet in depth (Hsieh et al. 2007).

Major sources of inflow to the aquifer are the Spokane River, spatial distributed recharge from precipitation, the lakes surrounding the aquifer, and streams/subsurface flow coming from the nearby mountains. Outflows are the Spokane River, Little Spokane River, and various municipal wells. The Spokane River (outflow from Lake Coeur D'Alene) is the largest source and sink of water in the aquifer.

Previous studies of the SVRP are plentiful and vary in purpose and scope. Four numerical computer models of the SVRP were completed pervious to the Hsieh et al. (2007) model (Bolke and Vaccaro, 1981; CH2M Hill, 1998; Buchanan, 2000; and Golder Associates Inc., 2004). Of the four only Buchanan (2000) modeled the entire aquifer but all studies found values of high hydraulic conductivity and Buchanan (2000) found values of up to 50,000 ft/day in parts of the SVRP.

The composition of the porous media that makes up the SVRP aquifer provides a location where many of the assumptions used in groundwater modeling equation are valid. Therefore, the boulders, cobbles, gravels and course sands that make of the aquifer which lead to high hydraulic conductivities, up to 22,100 ft/day (Hsieh et al., 2007) provide both a unique setting and lay within the assumptions of many groundwater equations. Hydraulic conductivities in natural settings are typically much lower than those in the SVRP aquifer and Fetter (2005) gives a typical range of hydraulic conductivity for well-sorted gravel as 28 – 2800 ft/day.

2.6 Groundwater Models and Assumptions.

The rationale for model selection used to meet the study objectives is presented in this section. The discussion includes both the reasons for choosing the analytical models from Moench and Barlow (2000) “STWT1”, Zlotnik and Huang (1999) “A2”, Hantush (2005) “A3”, Hydrus 2D/3D “HYDRUS”, and MODFLOW-2000 “MODFLOW” and the major assumptions defined in each model. The models presented in this section are representative of a wide range of approaches to solving the flux between aquifers and rivers, but are not all inclusive. The models below are well documented and predict observational data very well for other data sets. First the analytical models are presented followed by the two numerical models.

Over fifteen different analytical models were used in the selection pool from which three models were selected. The selection criteria focused on representing the SVRP properties as close as possible and selecting different approaches for the depiction of river-aquifer interaction. The most important properties in the SVRP aquifer for river-aquifer interaction are: 1) a layer of reduced K streambed sediments separating the river and the aquifer, 2) ability to represent a gaining or losing stream condition, 3) ability to include the affects of partial penetration of the river in the aquifer, and 4) unconfined multi-dimensional flow. Of these properties, 1 and 3 were considered essential and 2 and 4 were used to represent the regional nature of the aquifer. The uniqueness of the analytical model’s approach was based on how the conceptual model of river-aquifer interaction was setup and the limiting assumptions that were used. The analytical solutions that were used in the selection process are shown in Table 1. The general assumptions used in each solution are also listed, allowing for comparison of the models. Also presented in Table 1 is the approach of each model to describe the river-aquifer interaction. The top-down approach uses streamflow to evaluate the model (this can be modified to predict head in the aquifer in some cases). The bottom-up approach uses the aquifer head to evaluate the model and assumes that the head in the stream is the same for the whole reach. Using these two categories the values in Table 1 are separated with the gray shaded area representing the top-down approach and the non-shaded area showing the bottom-up approach.

Seven assumptions that are commonly applied to groundwater surface water interactions are outlined by Sharp (1977) and are present in Table 1 as:

1) The alluvial aquifer is homogeneous, isotropic, and infinite in extent.

2) The alluvium's bottom boundary is horizontal and impermeable.

3) The Dupuit-Forchheimer conditions are valid. Three conditions are inherent: a) in any vertical section the groundwater flow is horizontal; b) the velocity is uniform over the depth of flow; and c) the slope (α) of the free surface (unconfined flow) is small enough that $\tan(\alpha) \approx \sin(\alpha)$.

4) All flow is saturated.

5) Water from storage is discharged instantaneously upon reduction in head.

6) Fully penetrating streams (i.e., bottom of the streambed reaches the bottom of the aquifer)

7) A solely unconfined or solely confined groundwater system. Small fluctuations in the water table are often implied by solely unconfined condition, leading to constant transmissivities.

From these general assumptions two comparisons are to be made; the first is with the SVRP conditions and the second is the conditions in the SVRP applied to the different analytical assumptions in Table 1.

Table 1. Analytical model selection pool

Author	Year Published	1D / 2D	Linear / Non-Linear	Convo-lution Used	Stream-bed Layer Present	Homog-eneous	Iso-tro-pic	Fininte / Infinite	Full/P-artial Penet-eration	Horizontal Impermeable Lower Boundary	Dupuit Condi-tions Assumed	Confi-ned/ Unco-nfined
Moench et al.	1974	1	L	X	X	X	X	I	F†	X	X	U
Harada et al.	1999	1	L	X	X	X	X	I	P	X	X	U
Hantush et al.	2002	1	L	X	X	X	X	I	P	X	X	U
Birkhead and James	2002	1	L	X		X	X	I	F	X	X	U
Hantush	2005	1	L	X	X	X	X	F	P	X	X	U
Cooper and Rorabaugh	1963	1	L			X	X	B	F	X	X	U
Moench and Kisiel	1970	1	L	X		X	X	I	N/A	X	X	U
Hall and Moench	1972	1	L	X	X	X	X	B	F†	X	X	U
Dever and Cleary	1979	2	L	X	X	X*	X*	I	F			B
Neuman	1981	1	L			X		I	F	X		U
Kabala and Thorne	1997	B	L	X	X	X		I	P	X		U
Guo	1997	1	NL			X	X	I	F	X	X	U
Zlotnik and Huang	1999	1	L	X	X	X	X	I	P	X	X	B
Moench and Barlow	2000	B	L	X	X	X*		B	F†	X		B
Singh et al.	2002	1	L	X	X	X	X	I	F†	X	X	U
Akylas and Koussis	2007	1	L	X	X	X	X	I	F		X	U

† Stream fully penetrates the aquifer, but a resistance factor is used to represent effects of partial penetration
* Model is homogeneous or isotropic in each layer, but is allowed to have multiple layers
B is used to represent both features
Shading is used to distinguish top-down approach (shaded) from bottom-up approach (non-shaded)

The assumptions outlined by Sharp (1977) oversimplify the SVRP aquifer. Reconciliation of the assumptions, in most cases, is achieved by breaking the SVRP aquifer into multiple areas and analyzing each. The assumption of fully penetrating the aquifer is not valid in the SVRP and not all flow was saturated in the losing reaches of the Spokane River (see Figure 2 for example of disconnected losing reach). The other common assumptions were considered consistent with conditions in the SVRP aquifer. The SVRP was known from previous studies (Hsieh et al., 2007) to have very high values of hydraulic conductivity, which allows for quick responses of the water table. This unique feature of the SVRP, along with the saturated thickness of the aquifer, often over 200 feet (Hsieh et al., 2007), gives many of the common assumptions validity. Changes in water table elevation can be masked by the large thickness, allowing for relativity constant transmissivities, and groundwater mounds are dispersed quickly, so vertical flow components play less of a role, especially at distances of two or three river widths away.

As can be seen in Table 1, all models were able to simulate unconfined flow and homogenous conditions, resulting in those criteria not able to eliminate any models. Using the streambed reduced K layer and partial penetration the pool of models was reduced. Four models remained for top-down approach and five for the bottom-up approach. Using the finite/infinite boundary condition the final top-down approach was selected as Hantush (2005) and two bottom-up approaches remained, Barlow and Moench (1998) and Hall and Moench (1972). From these Barlow and Moench (1998) had a more complete documentation and a FORTRAN-77 computer code to solve the convolution integral, so it was chosen. In addition, since the partial penetration of the stream in the aquifer was considered very important a second bottom-up approach was chosen. Of the two analytical solutions directly dealing with partial penetration, Kabala and Thorne (1997) solution was designed to be used for pumping tests and not changes in river-stage. The Zlotnik and Huang (1999) solution was designed for analyzing changes in river-stage and provided a linear leakage mechanism as described in Chapter 2.2.

To be explicit the assumptions of each of the three analytical solutions chosen are presented so that the strengths of each model are clear. For the Barlow and Moench (1998) model the assumptions made are as follows:

1) Each aquifer is homogenous and of uniform thickness.

2) The lower boundary of each aquifer is horizontal and impermeable.

3) Hydraulic properties of the aquifers do not change with time.

4) The porous medium and fluid are slightly compressible.

5) Observation wells or piezometers are infinitesimal in diameter and respond instantly to pressure changes in the aquifer.

6) The water level in the stream is initially at the same elevation as the water level everywhere in the aquifer and aquitard (if included).

7) The semipervious streambank material, if present, is homogeneous and isotropic and has negligible capacity to store water.

8) The stream forms a vertical boundary in the aquifer and fully penetrates the aquifer.

9) The stream flows in a straight line (that is, without sinuosity).

10) Each aquifer can be anisotropic, provided that the principal directions of the hydraulic conductivity tensor are parallel to the x, z coordinate axes.

11) Water is released (or taken up) instantaneously in a vertical direction from (or into) the zone above the water table in response to a decline (or rise) in the elevation of the water table.

12) The change in saturated thickness of the aquifer due to stream-stage fluctuations or recharge is small compared with the initial saturated thickness.

13) Seepage and ground-water head at the stream-aquifer boundary are independent of depth.

Of the three models the assumptions in Barlow and Moench (1998) are the most detailed. The remaining two models have fewer assumptions due to the two-dimensional nature of Barlow and Moench (1998) solution. The assumptions for Zlotnik and Haung (1999) are as follows:

1) The stream is infinitely long in the horizontal plane and has low sinuosity.

2) The aquifer is homogeneous, isotropic, and semi-infinite in later extent.

3) The stream and the aquifer are initially at hydraulic equilibrium (at rest), and the water table is initially horizontal at some level h_O.

4) The streambed partially penetrates the aquifer with a hydraulic conductivity much less than the aquifer (shallow stream).

5) Drawdown in the aquifer is small compared with the saturated aquifer thickness so that the Dupuit-Forcheimer approximation is applicable.

6) Leakage across the streambed (to or from the stream) is vertical and occurs only through the bottom sediments of the stream.

7) The groundwater flow under the stream is confined while groundwater flow not under the stream is unconfined.

The assumptions for Hantush (2005) are as follows:

1) Stream channel is rectangular and hydraulically connected to the aquifer.

2) The aquifer is homogenous, isotropic, and on a horizontal plane with an impermeable base.

3) A no flow or prescribed head boundary is present normal to the channel axis at some distance.

4) Groundwater is one-dimensional and normal to the channel axis.

5) Alluvial sediments are separated from the stream by a distinct, semipervious layer.

6) The linearized form of the Boussinesq equation is applicable (Dupuit-Forcheimer conditions).

7) Storage below the channel is negligible and leakage from the channel is driven by the hydraulic gradient.

Since most numerical solutions are proprietary, a review by Kumar (2006) of fifty of the most common groundwater flow models was used to choose an appropriate model. Kumar (2006) gives a brief description of each groundwater flow model and provides what he considers to be the greatest strengths and weakness of each. MODFLOW (McDonald and Harbaugh, 1988) was recognized as the most widely used and since it was also used by Hsieh et al. (2007) for the SVRP aquifer and the data from the Hsieh et al. (2007) model was available, it will be used.

A second numerical solution HYDRUS 2D/3D (Šimůnek and Šenja, 2007) will also be used to determine the impacts of unsaturated flow on the GW/SW interaction in the SVRP aquifer. HYDRUS 2D/3D, referred to as HYDRUS from this point forward, will be reviewed first, followed by the review of MODFLOW.

HYDRUS is a finite element model that can simulate two- or three-dimensional water movement in variably saturated porous media. In HYDRUS the Richards equation is solved numerically for saturated and unsaturated flow. The solution uses either a Galerkin-type linear finite element scheme with the resulting matrix equations solved by either Gaussian elimination or conjugate gradient method depending on the size of the problem. The model assumes that water flow is uniform and in a variably saturated rigid porous medium with negligible air phase influence. The media can have anisotropy in all three coordinate directions and at angles to the coordinate directions. Many material layers can be included for heterogeneous conditions. Boundary conditions present in HYDRUS are specified pressure head (Dirichlet type), specified flux (Neumann type) and specified gradient. Additional boundary conditions which cannot be defined *a priori* are also available. Leakage from the river to the aquifer is based on head in the river and head in the underlying cells. No conceptualization of the interaction is included but values of head can be specified for each grid cell connected to the river. Fine grid approximation of the river allows for accurate representations of flux into the aquifer. In HYDRUS, pre-processing and post-processing packages are available and allow a user-friendly environment to construct the model and analyze the results. The details referenced in this paragraph are from the manual developed by Šimůnek and Šenja (2007).

MODFLOW, a groundwater model developed by the United States Geological Survey (USGS), solves the three-dimensional groundwater flow equation for a porous media by using a finite difference method (McDonald and Harbaugh, 1988). MODFLOW has a modular structure, which allows additional packages (solution methods to various groundwater flow conditions) to be added as they became available. MODFLOW solves the three-dimensional diffusion equation with a sink source term. Groundwater flow can be in a heterogeneous and anisotropic porous media, provided that the principle axes of hydraulic conductivity are aligned with the coordinate directions. Boundary condition packages used for this thesis include the River Package, Recharge Package, General Head Boundary Package, and Well Package. Many other packages are available and can be found at the USGS website: http://www.usgs.gov/. Of the four packages listed the Recharge, General Head Boundary, and Well Packages remain unchanged from the original Hsieh et al. (2007) form and are discussed there. The conceptualization of the river-aquifer interaction done in River Package is presented here to elucidate the assumptions used.

MODFLOW is designed to simulate aquifer systems in which 1) saturated flow conditions exist, 2) Darcy's Law applies, 3) the density of ground water is constant, and 4) the principal directions of horizontal hydraulic conductivity or transmissivity do not vary within the system (McDonald and Harbaugh, 1988). However, due to the practical mesh spacing at the regional scale, numerical models are larger than the dimensions of the river and the geometry and properties of the river channel cannot be represented in detail (Rushton 2007). Rushton (2007) also noted that even if very fine mesh spacing is used it is still impractical to represent the detail of the river channel.

MODFLOW's River Package models the interaction between the river and the aquifer by a seepage layer that separates the surface water body from the groundwater system. The surface water body is allowed to act as either a recharge zone for the aquifer or a discharge zone from the aquifer. Figure 7 describes the interaction.

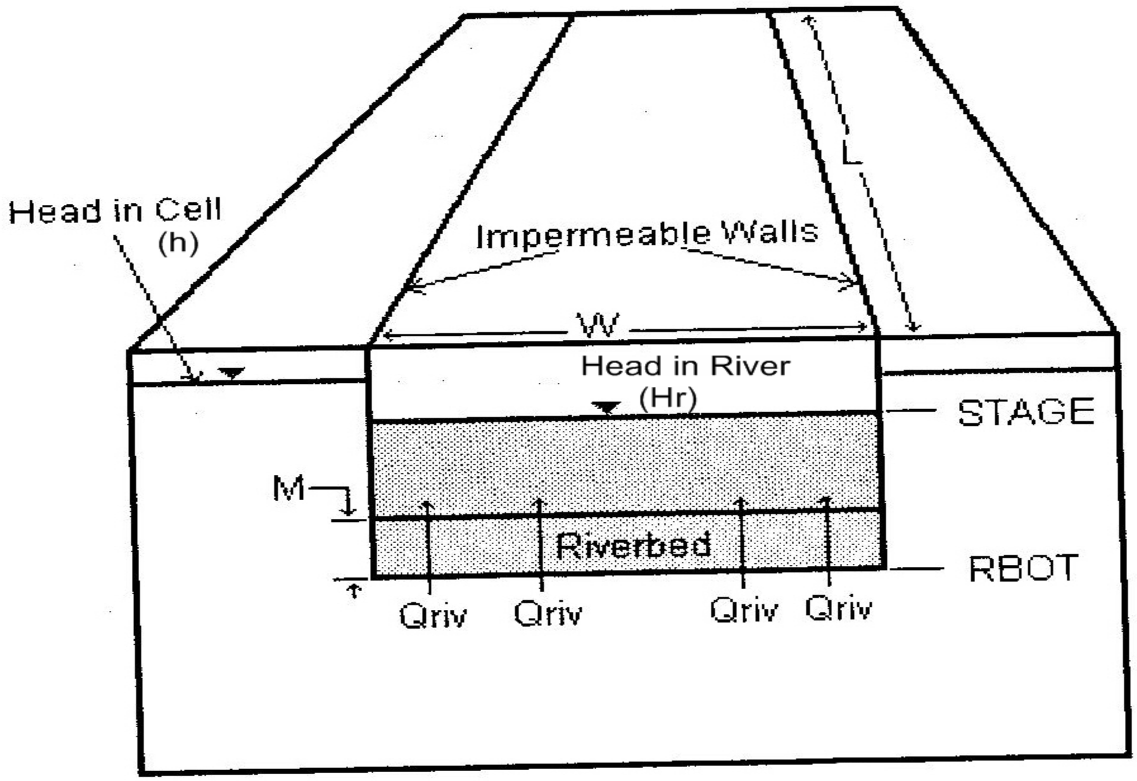


Figure 7. Schematic of River Boundary (Schlumberger Water Services 2009) with physical definition of variables

Inputs required by the MODFLOW River Package for the riverbed conductance factor are: river stage, the free water surface elevation of the water table (which may change with time), the Riverbed bottom (which is the elevation of the bottom of the seepage layer (bedding material) of the surface water body), and the conductance (a numerical parameter representing the resistance to flow between the surface water body and the groundwater caused by the seepage layer (riverbed)). MODFLOW uses Equation 8 to compute the conductance factor.

$$C = \frac{(KLW)}{M} \qquad \text{Equation 8}$$

where C is the riverbed conductance factor (L^2/time), K is the hydraulic conductivity (L/time), L is the reach length through a cell (L), W is the width of the river in the cell (L), and M is the thickness of the riverbed (L).

This approach relies on the assumption that modeling the banks of the river as impermeable is adequate and that bank storage and subsequent aquifer recharge can be measured as differences in the head between the aquifer and the river. It also assumes that the limitation to the linear relationship between the difference in river and aquifer head and flux across the riverbed found by Rushton and Tomlison (1979) are not significant for regional scale models. MODFLOW also assumes the head loss between the stream and the aquifer is limited to the cross-sectional width of the channel, the thickness of the riverbed material, and the point representing the underlying model node (Rushton 2007). MODFLOW then uses Equations 9 and 10 to determine head elevation and flow into or out of the aquifer.

$$Q = C(Hr - RBOT) \qquad \text{when } h \leq RBOT \qquad \text{Equation 9}$$

$$Q = C(Hr - h) \qquad \text{when } h > RBOT \qquad \text{Equation 10}$$

where Q is the flow in the river (L^3/time), C is the riverbed conductance factor (L^2/time), Hr is the river head in the cell (L), RBOT is the river bottom elevation (L), and h is the head in the cell (L).

McDonald and Harbaugh (1988) recognize that RBOT is not elevation where flux across the riverbed becomes independent of head (as shown in Figure 4) and state that the elevation at which further reduction of h no longer affects the amount of induced flux should be used as RBOT. No method to determine the elevation of the limiting aquifer head elevation is given.

The River Package does not have the flexibility of other MODFLOW packages which model the interaction between groundwater and surface water and does not include a method to allow for complex river geometries. Consideration of the flood plain is not a part of the package itself. The characterization of the river, as underflow or baseflow, is determined by the values computed for head in the cell and the value of the river stage. The overall characterization is therefore determined by the simulated head values. The position of the river in the alluvial flood plain is determined by the coordinates of the river cells and is very easily considered in modeling of the river. River reaches are allowed different soil property values of hydraulic conductivity and are incorporated into the model with zones. Zones allow for increasing heterogeneity and the accurate portrayal of the groundwater/surface water system.

2.7 Descriptive Statistics

To determine model performance, the Nash-Sutcliffe Efficiency Index is used. The Nash-Sutcliffe Efficiency Index (NSE) is a goodness-of-fit index (efficiency index) representing a slight modification of the coefficient of determination used for a simple linear regression model. The coefficient of determination is a least squares method, which uses the error sum of squares (SSE) and total sum of the squares (SST) to interpret the proportion of the observed variation in y that can be explained by the simple linear regression model (Devore, 2008). The equation for the coefficient of determination is:

$$r^2 = 1 - \frac{SSE}{SST} \qquad \text{Equation 11}$$

The modification to get to the NSE is to replace the linear regression line in the SSE for the line produced from a model. The resulting equation for the NSE is:

$$NSE = 1 - \frac{\sum_{i=1}^{n}\left(\hat{Y}_i - Y_i\right)^2}{\sum_{i=1}^{n}\left(Y_i - \bar{Y}\right)^2} \qquad \text{Equation 12}$$

where $\hat{Y}_i$ and Y_i are the predicted and measured values of the quantity of interest (dependant variable), respectively. $\bar{Y}$ is the average value of the measured quantity, often head in this study. The NSE has been used for a wide variety of model types, which indicates its flexibility as a goodness-of-fit statistic (McCuen et al., 2006). However, sample size and strong seasonal trends (Schaefli and Gupta, 2007) can lead to high NSE values that can be misleading. McCuen et al. (2006) presented a hypothesis test, confidence interval, and bias magnitude and ratio estimators, which helped to place the NSE in context. The bias magnitude and ratio of the bias to the mean of the measured values are given as:

$$\bar{e} = \frac{1}{n}\sum_{i=1}^{n}\left(\hat{Y}_i - Y_i\right) \qquad \text{Equation 13}$$

$$R_b = \frac{\bar{e}}{\bar{Y}} \qquad \text{Equation 14}$$

McCuen et al., (2006) concluded that NSE is a useful index, but it should always be reported with the model bias, where the bias can be differences in magnitude, time offset, or both.

In this application the NSE will be used to judge the ability of the models to match the data from the SVRP aquifer and Spokane River. Values of NSE close to one are a good match and values less than one show an increasing departure from the field data.

Chapter 3: Methodology

3.1 Approach and Models Used

The approach used to quantify how the conceptualization of flow across the streambed impacts the calibrated hydraulic conductivities of the aquifer and the streambed is presented here. The three analytical solutions (STWT1, A2, and A3), HYDRUS, and MODFLOW models selected from the literature review will be used for a number of scenarios to determine the similarity or differences each incorporates into the data. The quality of the data will be reviewed and the gaps will be identified. The variables that were used in the sensitivity analysis will also be presented as well as an explanation of why they are important.

3.1.1 Field Data Sets

The time scale for which data was collected for a large portion of the SVRP aquifer is in six minute intervals. The Six-Minute data study (Covert et al., 2005) provides the elevation of the water table head at 35 locations (see Figure 8) on the western portion of the SVRP aquifer. This allows for a general response in the aquifer to be established in conjunction with the artificial flood wave for the Post Falls Dam, ID as it travels through the aquifer.

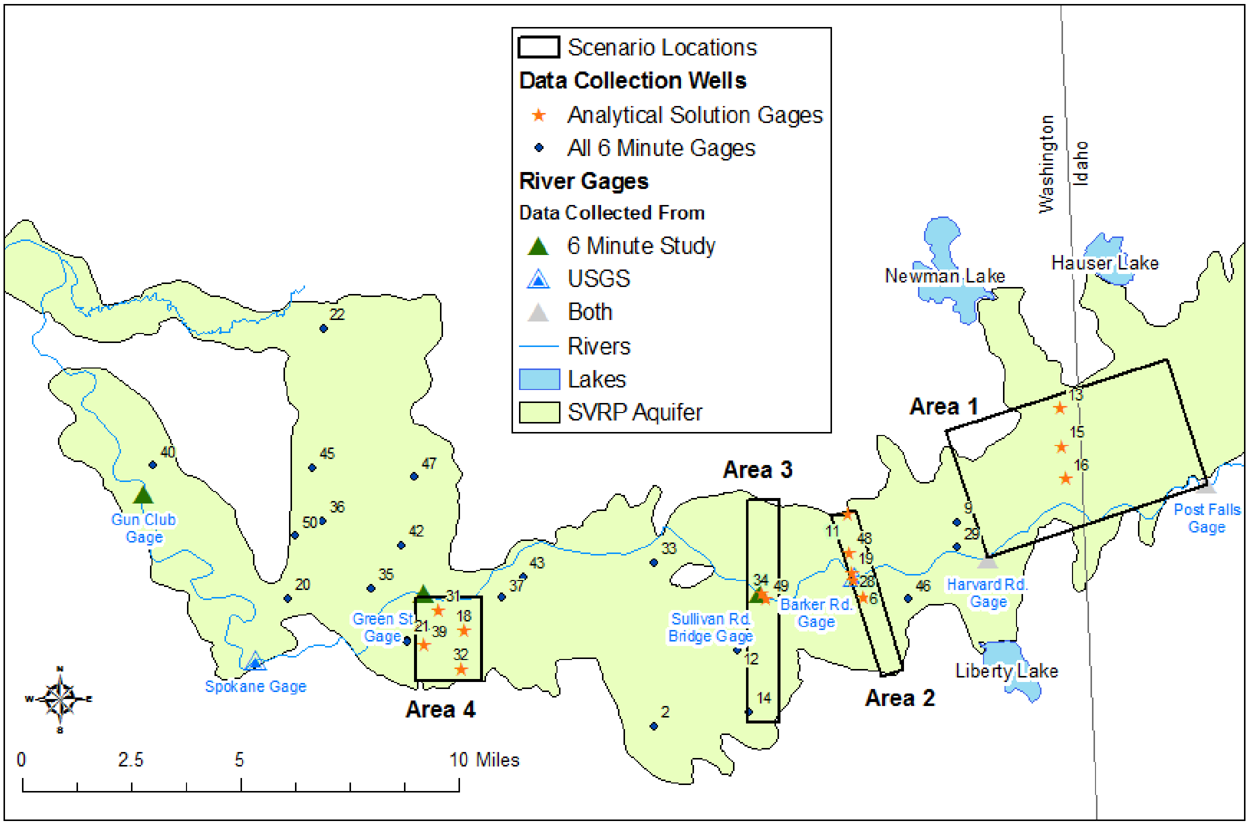


Figure 8. Observation well and aquifer test areas

Along with the 6-Minute data set the USGS has established flow measurement sites (Figure 8) along the Spokane River with fifteen-minute data points for both stage and flow. The two data sets allow for detailed temporal analysis of the aquifer around the artificial flood wave. In addition to the knowledge of the timing of the flood wave, September, a time when rain is minimal and irrigation pumping is fairly steady, is used to isolate the changes in the aquifer as other inputs to the aquifer are constant for most years (Hsieh et al., 2007).

FiguresFigure **9** and Figure **10** show examples of the data from the 6-Minute data set (Covert et al., 2005) and the USGS gage at Post Falls Dam, respectively. The measurement error for the Six-Minute Data set is 0.07 feet, which comes from a 0.1% error of the full scale which was 66 feet (Solinst Model 3001 Levelogger Junior, 2001). The artificial flood wave was generated during the low flow period in the Spokane River. This allows the flow measurements to be between five and ten percent error (USGS 12419000 SPOKANE RIVER NR POST FALLS ID, 2010).

The main advantage of these data sets is the temporal density of the data points, which provide a nearly continuous hydrograph for both the SVRP aquifer and the Spokane River. This allows robust comparisons of the STWT1, A1, A2, HYDRUS, and MODFLOW solutions at many data points during the flood wave. The overall impacts of each model prediction can thus be determined in addition to the range of hydraulic conductivities that can produce the actual response.

In addition to the water levels and flow values for the SVRP aquifer and the Spokane River, factors such as depth of the aquifer, river cross-sections, width and length of river reaches, and distance to the various observation wells are available in the Hsieh et al. (2007) report. Thus a full analysis can be completed for each of the five solution methods chosen.

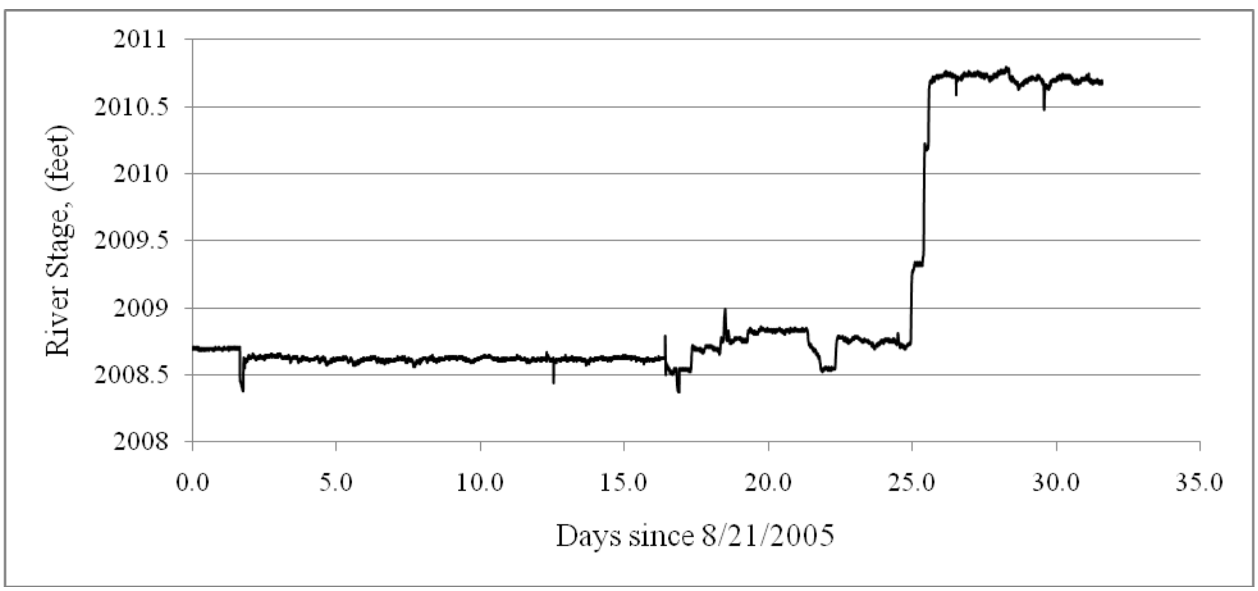


Figure 9. 6-Mintue example data from Post Falls Data Logger

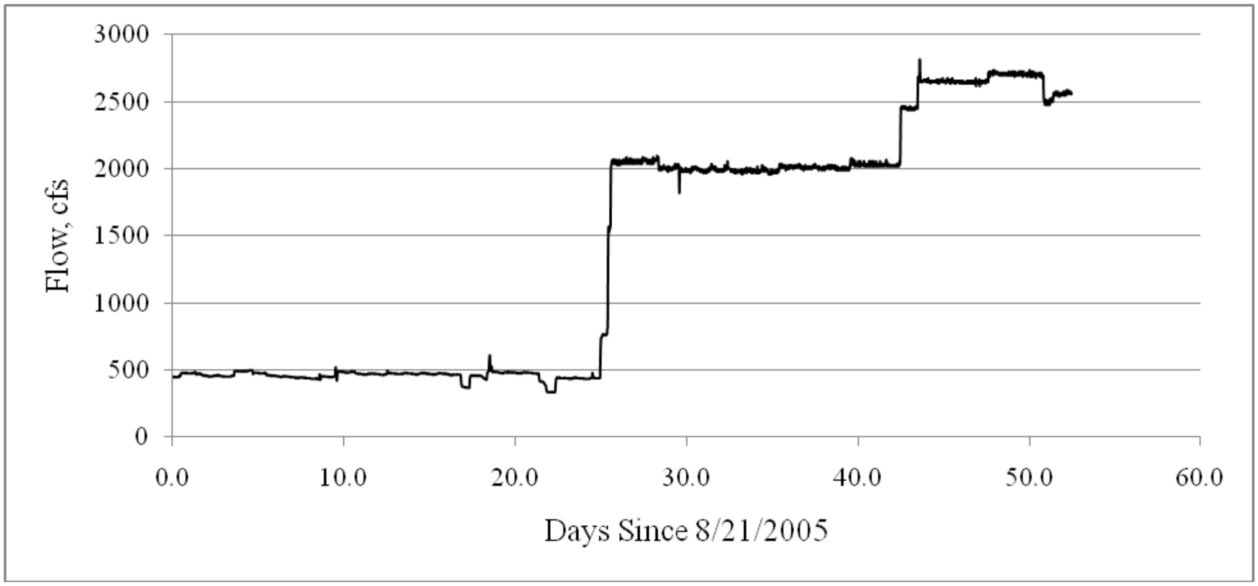


Figure 10. 15-Mintue example data from Post Falls USGS gaging station

3.2 Analytical Experiment

Analytical solutions solve the groundwater flow equation for one location normal to the stream channel. Out of 30 wells monitored in the Washington State Department of Ecology 6-Minute data set (Covert et al., 2005), a subset of 14 adjacent to the Spokane River stream gage locations were selected for comparison to the analytical models. This allowed direct correlations between changes in the water surface elevation in the Spokane River and water table changes in the SVRP aquifer. Table 2 shows the distances from the Spokane River gages to the 14 wells. Figure 8 shows the location of all the wells used in the analytical solutions, the locations on the Spokane River where elevations or flow measurements in the river were collected, and the areas selected for the analytical solutions.

Table 2. Distance from the Spokane River to the observation wells used in analytical analyses

Area	Well Name	Distance (ft)
Area 1	Well 13	14262

	Well 15	9809
	Well 16	5995
Area 2	Well 11	7312
	Well 48	2768
	Well 19	353
	Well 28	478
	Well 6	2822
Area 3	Well 34	85
	Well 49	237
Area 4	Well 31	1478
	Well 18	3938
	Well 39	5748
	Well 32	8665

The four areas selected in Figure 8 can be grouped by the data that is available. For Areas 1 and 2 seepage rates along with heads in the observation wells are available. For Areas 3 and 4 only heads in the observation wells are available. The wells for Area 1 are located between the Post Falls and Harvard Road Gages. To find the changes in the Spokane River the time-lagged average value between the two gages is used, which is about 2 hours and 30 minutes. The other three areas use their respective change in the gage as indicated in Figure 8.

One assumption in all three analytical solutions is that initially the water table elevation is equal to the elevation of the water surface in the stream. This condition is not present in Areas 1-4. However, since the artificial flood wave generated by the opening of the Post Falls Dam does not have a falling limb, water will have only one direction of travel, either entering or leaving the aquifer. If the falling limb was present then the direction of flow would be reversed during the falling limb. Areas 1, 2, and 4 are strictly losing reaches and will always have water entering the aquifer from the river. In Area 3, the initial condition of the water table is greater than the elevation of the surface water. The resulting values in the observation wells are thus expected to be greater than the measured values in the 6-Minute Study. Figure 11-Figure 14 show cross sections of Areas 1-4 looking upstream, with the wells at approximate locations in the aquifer as the solid vertical green lines (see Table 2 for actual distance), the blue color shows the saturated portion of the aquifer, the tan color the unsaturated portion and the diagonal pattern shows the location of the bedrock.

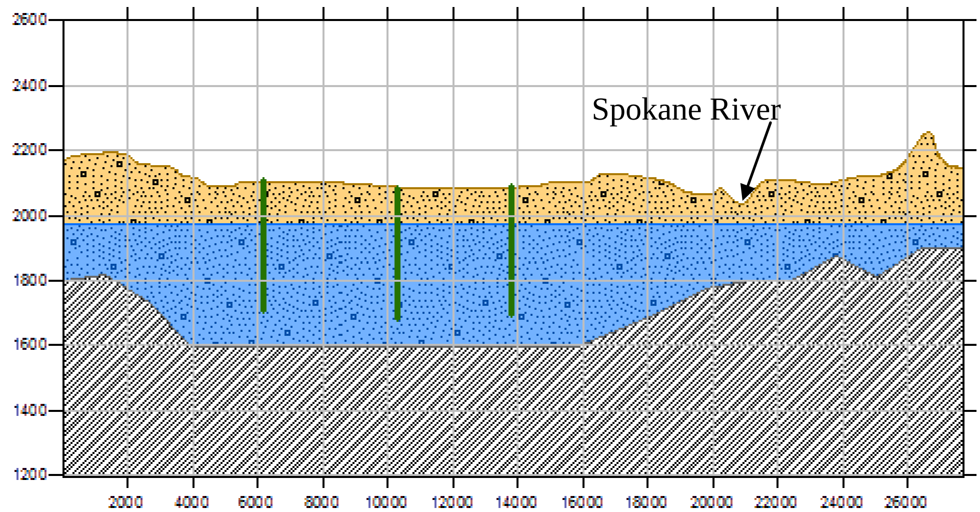


Figure 11. Area 1 center line cross section.

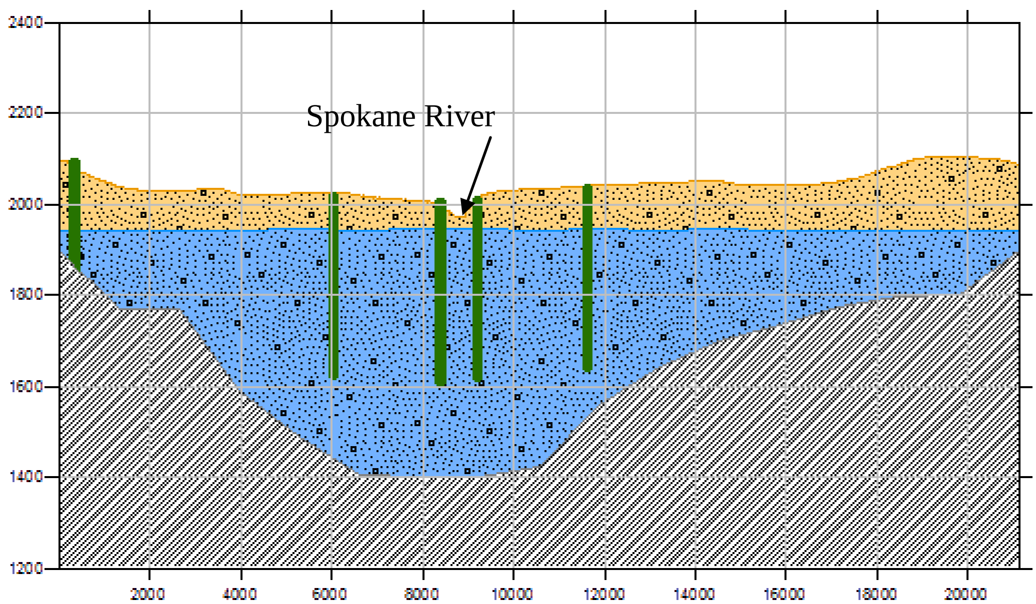


Figure 12. Area 2 center line cross section.

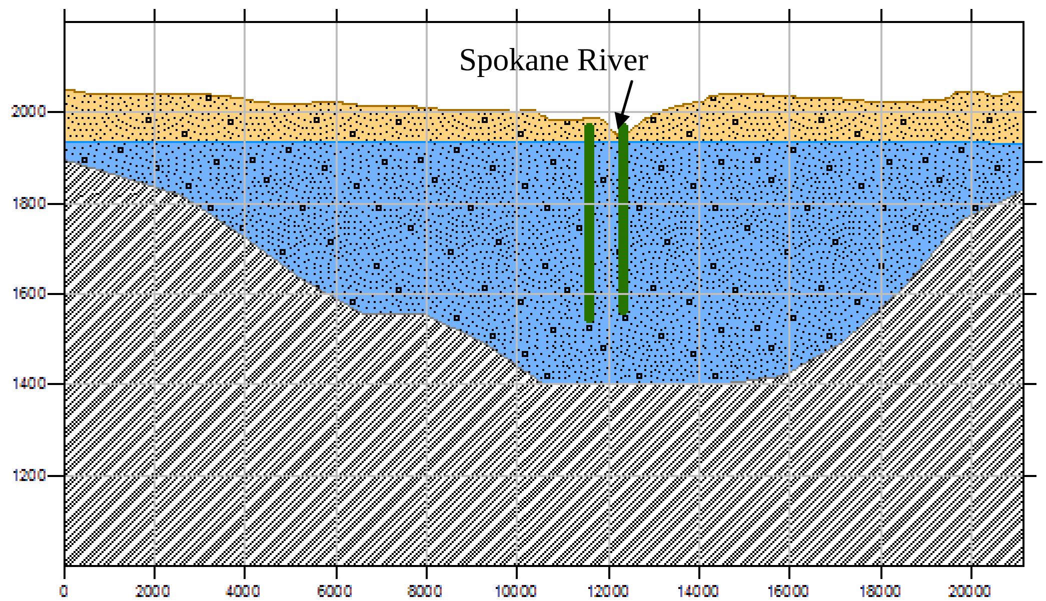


Figure 13. Area 3 center line cross section.

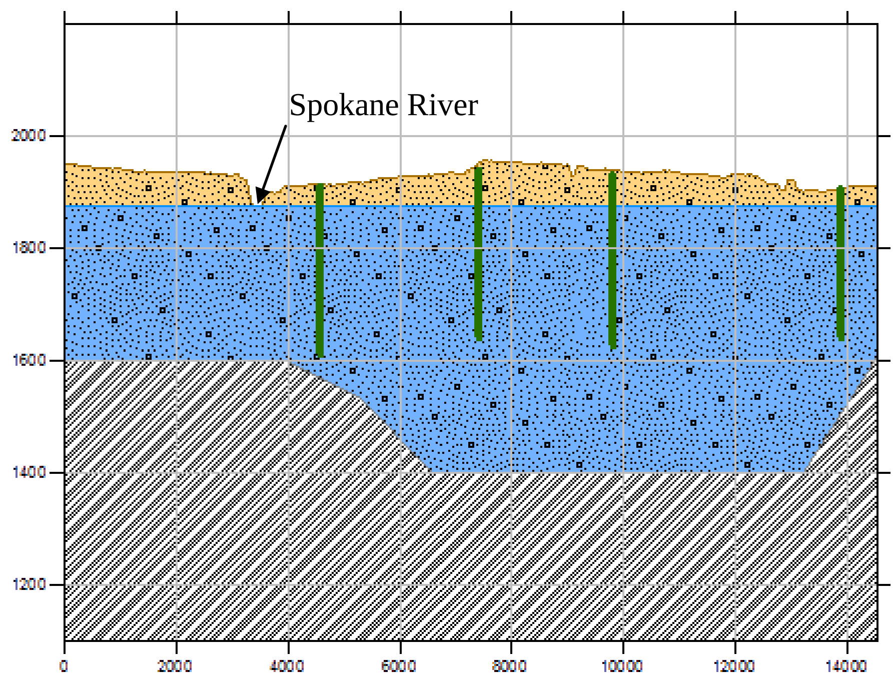


Figure 14. Area 4 center line cross section.

Also variable for Areas 1-4 were cross sections of the Spokane River that were surveyed by the USGS (see Figures Figure **15** Figure 18). The Spokane River cross sections allow for good estimates of widths and wetted perimeters for the starting values in the analytical and numerical solutions.

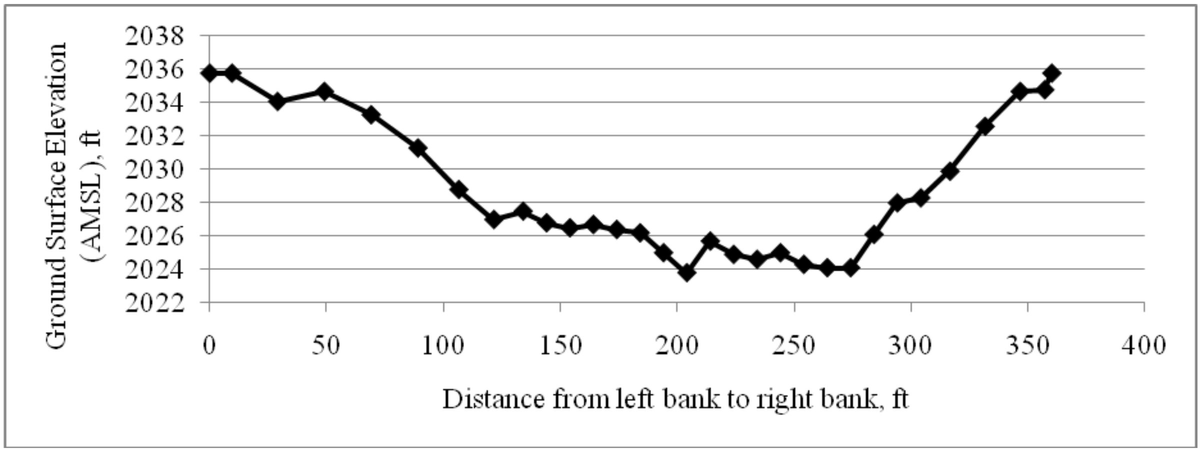


Figure 15. Area 1 cross section

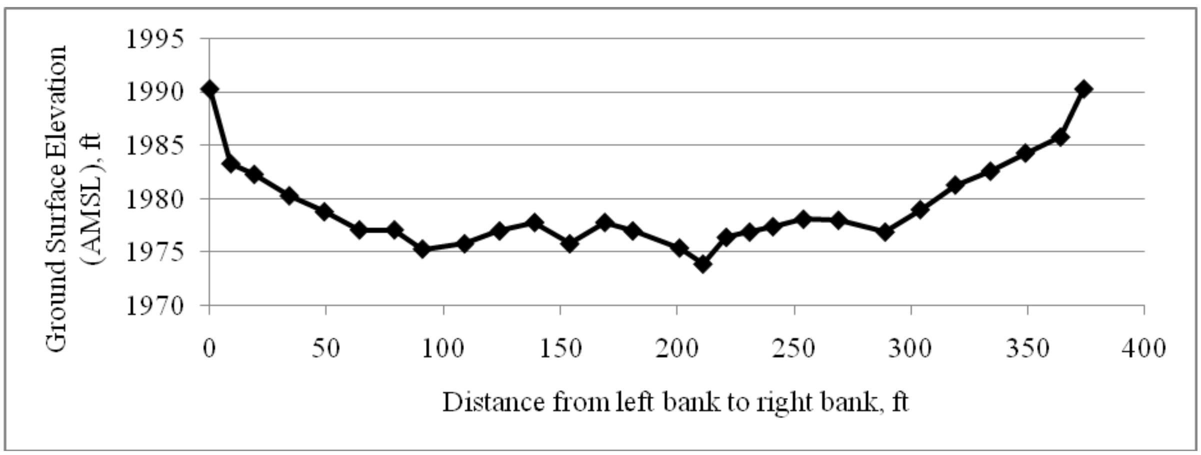


Figure 16. Area 2 cross section

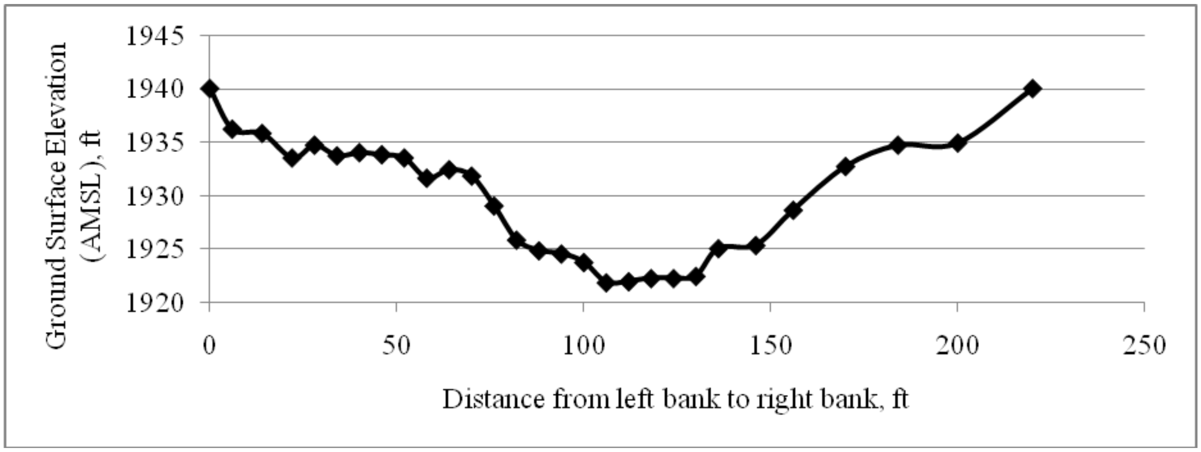


Figure 17. Area 3 cross section

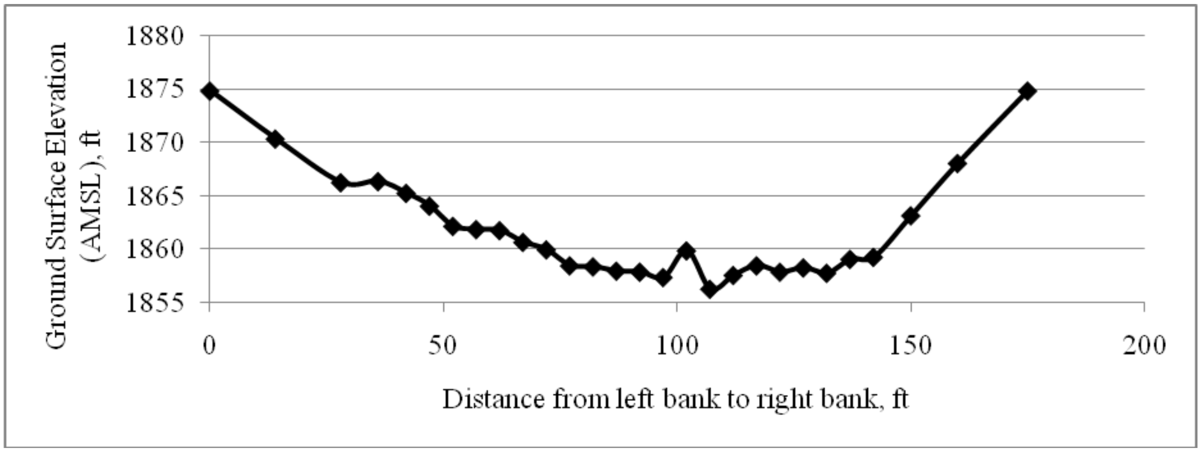


Figure 18. Area 4 cross section

3.2.1 Barlow and Moench (1998) Analytical Solution (STWT1)

A two-dimensional vertical general section of an aquifer, which depicts groundwater flow in the vicinity of stream, is given in Figure 19 to describe the variables used in the Barlow and Moench (1998) analytical solution (STWT1). In the figure, d is the thickness of the semipervious streambed material, c is the step change in water table height, h_i = b is the initial saturated thickness of the water table, h_0 is the water level after the instantaneous step change, and initially the water table and the elevation of the stream are equal. A well screen interval is used at the observation well to provide an average head value for the screened portion, as flow is two-dimensional the x and z directions. The piezometers provide water table measurements at the depth of the casing. For the additional assumptions used refer to Chapter 2.3.

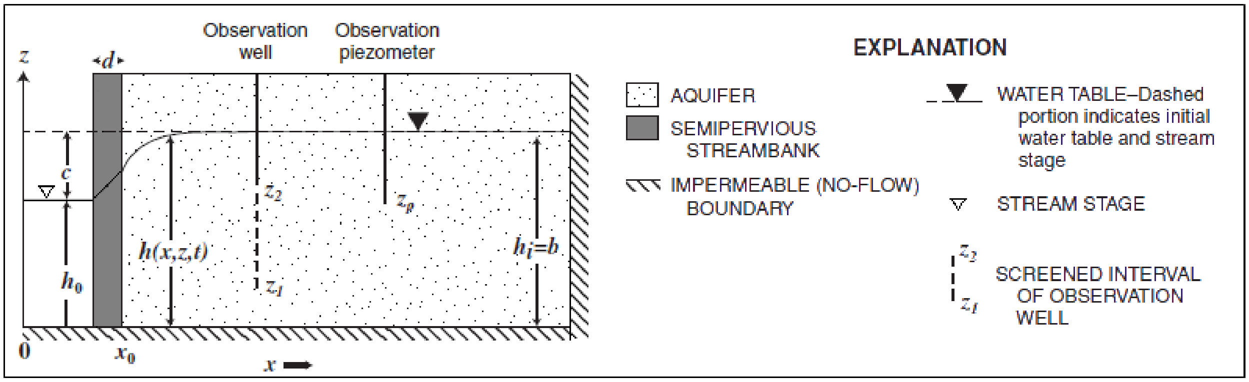


Figure 19. Barlow and Moench (1998) analytical solution depiction

The STWT1 solution is capable of determining three sets of information from the stage change in the river: 1) heads in the aquifer, 2) streambank seepage rates, and 3) bank storage. Seepage rates and bank storage were only calculated for Areas 1 and 2 (Figure 8) because of the availability of flow data in the Spokane River. The USGS gages provided both flow and stage changes, while the 6-Minute gages only provide stage changes (See the legend in Figure 8 to distinguish gage locations). Between Barker Road gage and Spokane Gage multiple dams and gaining and losing reaches are present so flow was convoluted from the river management decisions, making direct comparisons of seepage not possible. Areas 1 and 2 are also strictly losing reaches, so bank storage would not be a useful comparison since it is derived from seepage rates. Therefore, Areas 1 and 2 have two available comparisons: aquifer heads and seepage rates. For Areas 3 and 4 only aquifer heads are available. This gives a total of fourteen observation wells from the 6-Minute data study and two seepage rate comparisons that are available to compare with the STWT1 solution. Seepage rates are determined for Areas 1 and 2 by applying the time lag between gages then subtracting the values of flow to determine the amount gained or lost. The difference is divided by the length of the river between the gages to determine the seepage per foot of channel length.

The approach for each area (and each analytical solution) was to first test if the values given from the calibrated MODFLOW 2000 model by Hsieh et al., (2007), shown in Table 3, were able to reproduce the head changes in the various observation wells. Each well had to be tested independently because the analytical solution only solves the groundwater systems at one point in the aquifer (i.e. the observation well). The Nash-Sutcliffe Efficiency Index (NSE) was then used to determine the fit of the calculated data with the observed data from the 6-Minute Study.

Table 3. MODFLOW 2000 Calibrated Parameters in Areas 1-4 (Hsieh et al., 2007)

Area	K aquifer [ft/day]	K streambed [ft/day]	S_y []
1	21893	0.25	0.19
2	19090	0.22	0.19
3	7466	9.4	0.19
4	9503	10	0.19

Other parameters used in each area are shown in Table 4. Of all the parameters in both Table 3 and Table 4 only the hydraulic conductivity of the streambed (K') and S_y where used to refine the solution after the original MODFLOW 2000 calibrated parameters where used.

Table 4. STWT1 solution parameters for Areas 1-4

Parameter	Symbol	Units	Area1	Area2	Area3	Area4
Δ t	DELT	days	0.004167	0.010417	0.004167	0.004167
Aquifer Extent	IXL	[]	1	1	1	1
Aquifer Type	IAQ	[]	1	1	1	1
Stream Half Width	XZERO	ft	92	100	50	100
Aquifer Width	XLL	ft	16000	7570	15000	10000

Streambank Leakance	XAA	ft	87572	87569	794.26	950.3
Stream Length	XSTREAM	ft	26902	3113	1485	1485
K aquifer	AKK	ft/day	21893	19090	7466	9503
Anisotropy	XKD	[]	1	1	1	1
Aquifer Specific Storage	AS	[1/ft]	0.00001	0.00001	0.00001	0.00001
Aquifer Specific Yield	ASY	[]	0.19	0.19	0.19	0.19
Saturated Thickness	AB	ft	323	315	342	410
Distance to Obs. Well	X	ft	>5000	>350	>80	>1450
Obs. Well Type	IOWS	[]	1	1	1	1
Vertical Distance from bedrock	Z1	ft	0	0	0	0
Vertical Distance to top screened interval	Z2	ft	323	315	342	410
Obs. Well Initial head	HINIT	ft	>1	>3	>5	>1
Start Time	TINIT	days	0	0	0	0
# of Stehfest terms	NS	[]	8	8	8	8
Relative Error	RERRNR	[]	1.00E-10	1.00E-10	1.00E-10	1.00E-10
# finite sum terms factor	XTRMS	[]	20	20	20	20
# of time steps	NT	[]	6593	3448	1595	8600

In Table 4 the observation well initial head was set to low values to increase the precision of the calculations. The observation screened interval is assumed to be the full thickness of the aquifer and distance to the observation well given is to the closest well in each area in Table 2. Values in Table 4 are from Hsieh et al., (2007) and from recommendations in Barlow and Moench (1998).

3.2.2 Zlotnik and Huang (1999) Analytical Solution (A2)

A two-dimensional vertical general section of an aquifer, which depicts groundwater flow in the vicinity of stream, is shown in Figure 20 to describe the variables used in the Zlotnik and Huang (1999) analytical solution (A2). The Roman numerals represent two zones: one under the river (confined zone) and the other containing the remaining aquifer (unconfined zone). As stated by Zlotnik and Huang (1999), neglecting Zone I can be a proper idealization of the general case and produces results with a relative error of better than 0.5%. Any arbitrary unit step function $S(t) = h_0 - H(t)$ can be used to find S_I and S_{II}, which is the drawdown in Zone I and Zone II respectively, where $S_i = h_i(x,t) - h_0$. As shown, the half width of the stream is equal to w, m' is the thickness of the streambed sediments, and x is the distance to the observation well. The boundary conditions at the stream axis (x = 0) is assumed to be no-flow and symmetric. Additional assumptions are described in Chapter 2.3.

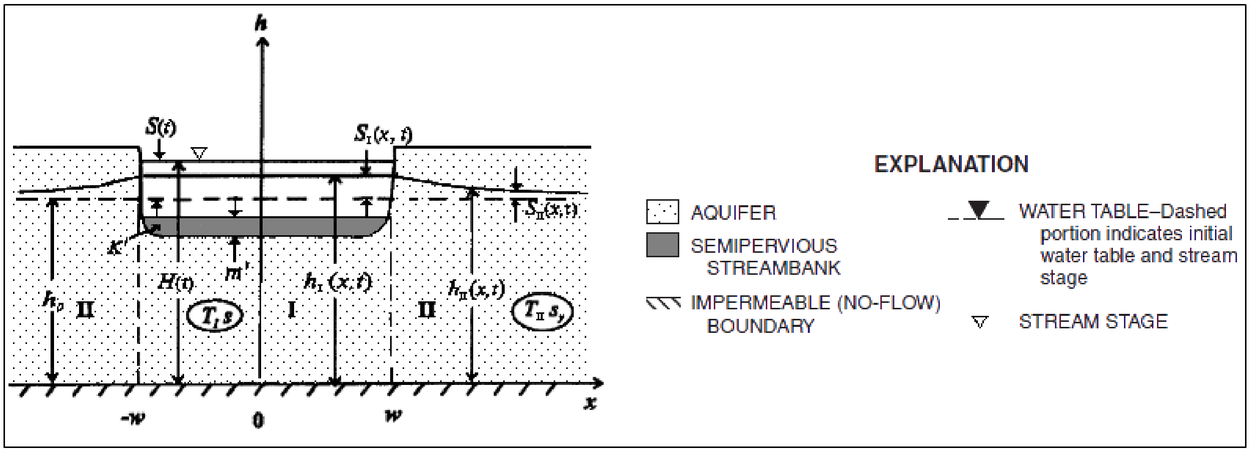


Figure 20. Zlotnik and Huang (1999) Analytical Solution (A2)

The result of the A2 solution is the value of heads in the aquifer, which can be compared to the fourteen observation wells in Areas 1-4. The parameters used to describe the characteristics of the area are given in Table 5. The size and number of time steps for each area are given in Table 4.

Table 5. Zlotnik and Huang (1999) Parameter Values for Areas 1-4

Parameter	Symbol	Units	Area1	Area2	Area3	Area4
Hydraulic Conductivity of aquifer	K	ft/day	22150	19090	7466	9503
Saturated Thickness under the river	b_I	ft	323	315	342	410
Saturated Thickness not under river	b_{II}	ft	323	315	342	410
Transmissivity under river	T_I	ft^2/day	7154450	6013350	2553372	3896230
Transmissivity not under the river	T_{II}	ft^2/day	7154450	6013350	2553372	3896230
half width of the river	w	ft	92.1	100	50	100
Hydraulic Conductivity of Streambed	K'	ft/day	0.25	0.22	9.4	10

Thickness of Streambed	m'	ft	1	1	1	1
specific yield	S_y	[]	0.19	0.19	0.19	0.19
Distance to well	x	ft	>5000	>350	>80	>1450

The approach in the A2 solution is to use a convolution integral to step through the rise of the artificial flood from the Post Falls dam to solve for the heads in the aquifer at the fourteen observation wells indicated in Figure 8. The A2 solution provides an explicit definition of the partial penetration of the streambed and uses the same variables as in the linear leakage relationship outlined in Chapter 2.1. Since MODFLOW 2000 uses the same variables in the relationship used to describe leakage a direct comparison between the two can be made.

3.2.3 Hantush (2005) Analytical Solution (A3)

The final analytical solution to be used is the model proposed by Hantush (2005). A two-dimensional vertical general section of an aquifer, which depicts groundwater flow in the vicinity of stream, is given in Figure 21 to describe the variables used in Hantush (2005) analytical solution (A3). For the A3 solution P is the wetted perimeter of half of the river, Q(t) is the deviation of seepage from its initial value at t = 0, H(t) is the channel stage deviation from its initial value, N(t) is an application of recharge (not used), l is the distance from the edge of the river to the impermeable boundary, W is the full width of the river channel, and K and S_y are as defined above. K' and b (not shown) are the hydraulic conductivity (ft/day) and thickness of the streambed sediments, respectively. The change from the initial water table level h_0 is given by h(x,t).

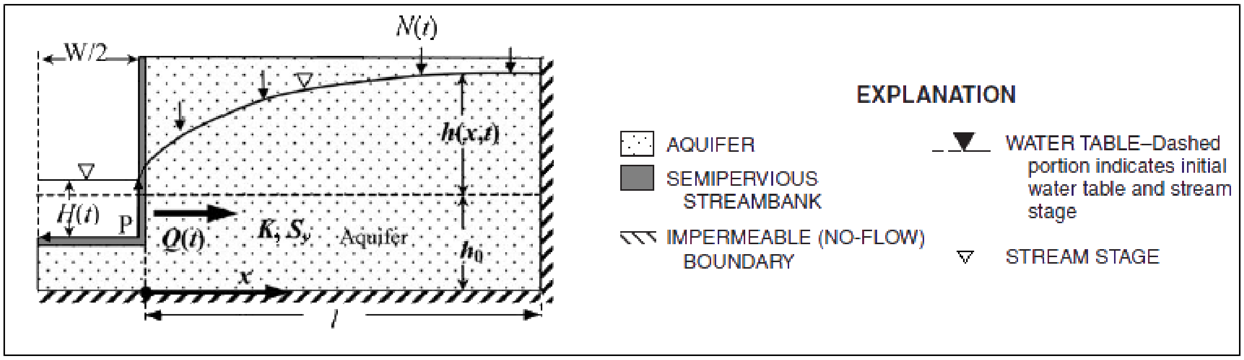


Figure 21. Hantush (2005) Analytical Solution (A3)

In the A3 solution, the change in stream flow is used to determine a new outflow from the reach given an inflow. Since only Areas 1 and 2 had flow data this solution could only be used for these areas. This solution gives the flow at the end of the reach and the seepage out of the reach. Heads in the aquifer could be used if an alternative form of the equation is used; however, this is not presented by Hantush (2005). The A3 solution also relies on the linear Muskingum channel storage model. Using the artificial flood wave from the Post Falls Dam the storage time constant η for the reach and the weighting factor ξ are determined. Then using fifteen minute flow data from the USGS, the solution is calibrated using outflow and seepage from the channel. Since outflow and seepage are solved independently from one another both can be used to evaluate the calculated values with the NSE. Four parameters were allowed to change (K', S_y, η, and ξ). Both η and ξ were allowed to change because the artificial flood wave from the Post Falls Dam only provided a rising limb and not a falling limb. This did not allow a good match to be found for either parameter, so calibration was used to determine values for both. Also, both Areas 1 and 2 are strong losing reaches and the linear Muskingum method has been shown by Birkhead and James (2002) to over-predict values of η and ξ when bank storage is significant. The values used for Area 1 and 2 are shown in Table 6.

Table 6. Hantush (2005) analytical solution (A3) parameters

Parameter	Symbol	Units	Area1	Area2
Hydraulic conductivity of aquifer	K	ft/day	22150	19090
Hydraulic conductivity of streambed	K'	ft/day	0.25	0.27
Saturated Thickness of aquifer	b	ft	323	315
Specific yield of aquifer	S_y	[]	0.19	0.19
Width of the aquifer	l	ft	16000	7570
Full width of the River	W	ft	184	200
Half width of the Wetted Perimeter	P	ft	84	93
Thickness riverbed sediments	d	ft	1	1
Muskingum Time Storage Constant	η	days	0.104	0.076
Muskingum Weighting Factor	ξ	[]	0.14	0.3
Number of Values	-	-	1322	1823

3.2.4 Reasonable Limits of Parameters

The parameter of most interest to determine for all the analytical solutions is the hydraulic conductivity of the streambed material (K'). The starting value for K' is provided by Hsieh et al. (2007). No limits were established to restrict the calibrated values. This was deemed reasonable as the goal was to determine the effect of the model on the value of K'. Specific yield (S_y) was also allowed to change for each model but was restricted to between 0.02 and 0.3 (Fetter, 2001). However, S_y is known to increase in general as diameter of the pores in the material increases. The coarse materials that comprise the aquifer (boulders, cobbles, gravels and sands) would then imply a high specific yield. Tables presented in Fetter (2001) and Domenico and Schwartz (1990) both show values of S_y greater than 0.3 with the highest value reported at 0.38, Bolster et al. (2001) report S_y up to 0.57. Theoretically, specific yield could approach a value near porosity. Thus specific yield was allowed to go above the limit to determine if a better fit could be obtained with S_y values greater than typically reported. The Muskingum parameter η was allowed to take on values that were close to the lags found between reaches (2.5 hours for Area 1 and 1.5 hours for Area 2). ξ was restricted to the theoretical range 0 to 0.5 (Cunge, 1969), with the understanding that values above 0.3 are not typical. River and aquifer width, riverbed sediment thickness, distance to the wells, saturated thickness of the aquifer, and screened intervals of the wells were not allowed to change.

3.3 HYDRUS Scenario

The HYDRUS Scenario was also conducted to determine values of K'. Three scenarios where set up: 1) Two-dimensional, 2) Three-dimensional, and 3) Two-dimensional with seepage rates exported to Visual MODFLOW.

3.3.1 Two-Dimensional Scenario

The two-dimensional scenario was used to determine how the unsaturated zone affected the value of K' by comparing seepages rates. Areas 1 and 2 were used and a cross-section of the river and the aquifer was used to model the interaction. The width of the cross-section was limited to the width of the river plus 65.6 ft (20 meters) on each side. The depth of the cross-section was limited to the depth of the water table, which is 52 feet for Area 1 and 25 feet for Area 2 at the lowest point in the river (Figure 22).

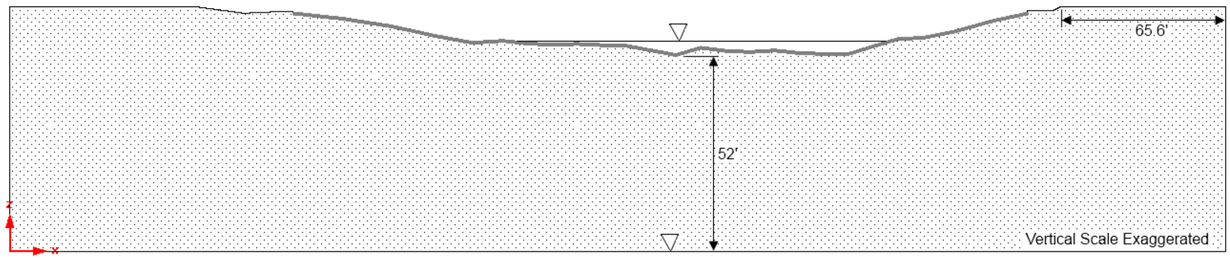


Figure 22. Two-Dimensional HYDRUS Set Up

The bold line in Figure 22 shows the one foot thickness of the riverbed with the associated K' value. The water level in the stream is representative of the initial depth. The finite element mesh has a spacing of 0.82 feet (0.25 m) except for the upper left and right corners. The riverbed sediment and the aquifer have the same K and K' values as previous Area 1 and Area 2 descriptions. Boundary conditions are constant pressure head of 0 along the bottom (water table), no flow along both sides, variable head in the river channel, and very small constant head boundary along the top to keep from creating unrealistic negative heads in the unsaturated zone. The van Genuchten parameters for Area 1 and Area 2 are given in Table 7.

Table 7. Soil parameters (Garcia, 2010)

Parameter	Symbol	Units	Area1	Area2
Residual Water Content	θ_r	[]	0.078	0.078
Saturated Water Content	θ_s	[]	0.43	0.43
van Genuchten	α	1/m	5.5	5.5
	n	[]	2.08	2.08
	m	[]	0.52	0.52

3.3.2 Three-Dimensional Scenario

For the three-dimensional (3D) scenario Areas 1–4 were reshaped to follow the outline of the aquifer as shown in Figure 23. Areas 1–4 are aligned with the aquifer boundary and the contour lines for September 2005 from the Hsieh et al. (2007) MODFLOW model. The values used for the soil parameters from Table 7 are also used in the 3D scenario. The finite element mesh spacing was about 25 ft in the horizontal direction near the river and expanded when moving away from the river. In the vertical direction the mesh spacing was always less than 5.4 ft under the river where seepage was occurring to obtain accurate results.

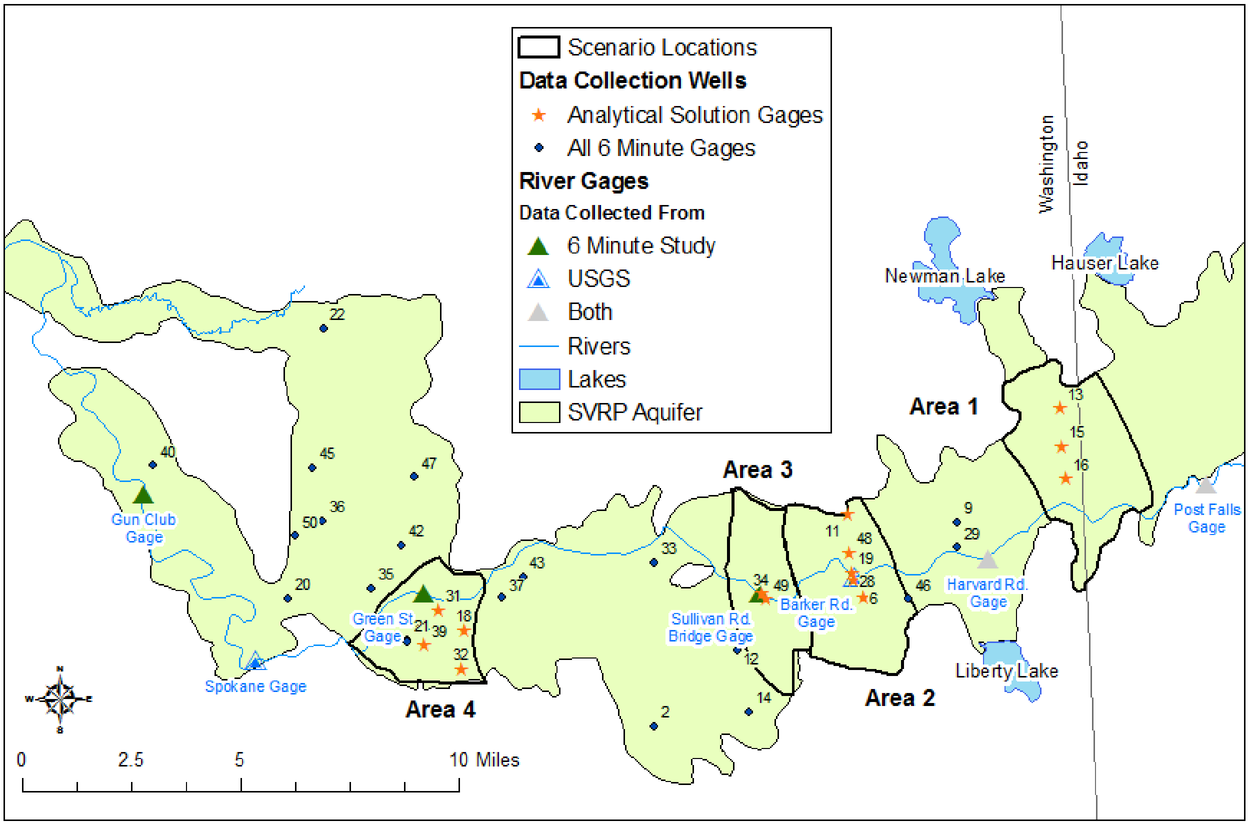


Figure 23. HYDRUS 3D area delineations

3.3.3 Two-Dimensional Scenario with MODFLOW

In this scenario the areas defined in Figure 23 are used in HYDRUS and in MODFLOW. The goal of this scenario was to use a fine grid approximation of the river aquifer interaction from HYDRUS as a boundary condition in MODFLOW. The soil parameters in Table 7 were used for HYDRUS and the calibrated aquifer properties from Hsieh et al. (2007) where used in MODFLOW. In MODFLOW the seepage rates determined from HYDRUS were input with the recharge package, which allowed the seepage to immediately reach the water table. K' from HYDRUS therefore controlled the seepage rate into the MODFLOW model. The September 2005 contours from Hsieh et al. (2007) were assumed to change by the average height of the wells in each respective area. For example, the average change in the position of the water table from Wells 13, 15, and 16 was 1.5 feet so both the east and west boundaries changed in depth 1.5 feet during the simulation period. This was necessary as the large K values in the aquifer allow for flow to move quickly in the aquifer. The inflow from Newman Lake was also modeled using the recharge package. Area boundaries coinciding with the aquifer boundary were modeled as no-flow boundaries. Visual MODFLOW was used to input the model data and collected the results at each well. Since not all areas had direct seepages rates to compare with, head in the aquifer for Areas 3 and 4 was used to determine the best approximation to the K' value that produced heads consistent with the 6-Minute Data Study.

3.4 MODFLOW Scenario

The final scenario was to use the original model from Hsieh et al. (2007) to determine how well the regional model could reproduce the artificial flood wave generated by the Post Falls Dam. To complete this, the Recharge, Well, River, and General Head Boundary Packages needed to be modified from monthly time steps to fifteen-minute time steps. The original model's final time step was September, 2005. The 6-Minute Data Study captures aquifer and river changes from August 21, 2005 to in most cases October 20, 2005. The overlap allows for all the September values currently in the MODFLOW packages to be used at each time step for the new fifteen-minute time scale. October values for the MODFLOW packages had to be derived from other data. The fifteen-minute time scale was used to allow the additional data from the USGS gages (USGS data recorded very fifteen minutes) to be used without extrapolation. The 6-Minute data could be interpolated without loss of precision to fifteen-minute intervals.

Since monthly data was the most precise data available for all but the River Package, the monthly values from Hsieh et al. (2007) were used for September in the Recharge, Well, and General Head Boundary Packages. To determine the values for October 2005, precipitation data was used to compare prior years to 2005 to establish which was most similar for the entire fifteen years of model data and which was most similar in October 2005. The report by Bartolino (2007) was used to determine the values of precipitation and recharge per month to the SVRP. Only the Spokane WSO Airport data was used. It was found that 2004 had an October precipitation that was exactly the same as 2005 and that in general and rest of the year had similar highs in the same months (Figure 24). The main differences are in precipitation happening in February and March, and July and August. However, it would seem that deficit in both March and July of 2004 are compensated by the surplus in February and August of 2004. Recharge shows a similar pattern but less pronounced, with an unexplained difference in May. No other years between 1990 and 2004 matched the October precipitation.

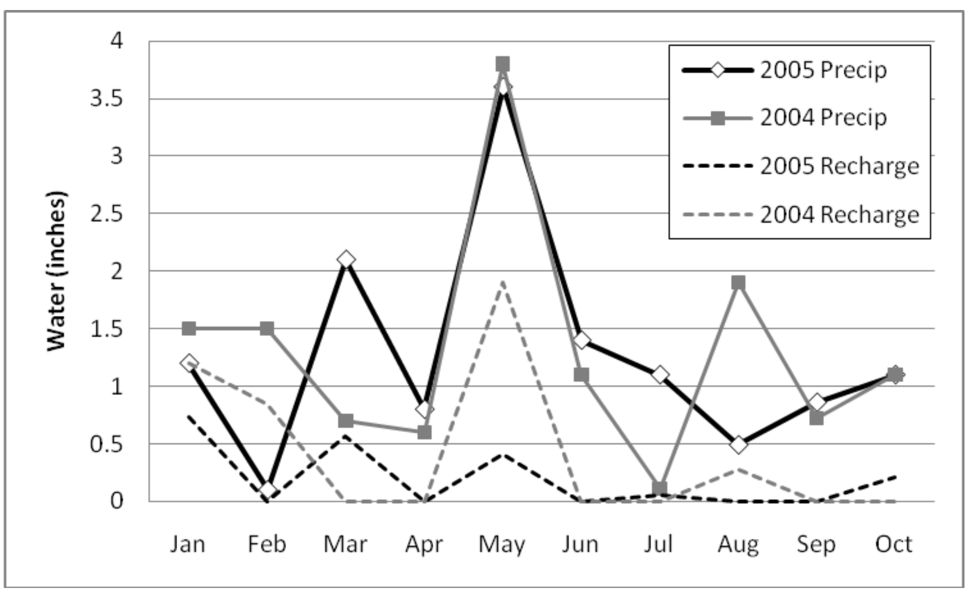


Figure 24. Precipitation and recharge comparison

Having resolved the Recharge Package data for October of 2005 the Wells Package was examined next. Since the amount of pumping and the amount of precipitation can be linearly related, it would be expected that 2004 might have similar pumping rates. Only August, September and October were used to compare the years from 1990-2004, 2004 showed the most similar pumping pattern. A year close to 2005 for pumping rates is desirable since adding and decommissioning wells in the SVRP over time could affect the location and amount of water withdrawn from the aquifer. Since development is occurring over time, the closer to the year 2005, the higher likelihood of a better match of pumping conditions. Considering both the analysis of pumping rates and the need for a close period of time, 2004 was also selected to simulate the October 2005 wells pumping rates.

The General Head Boundary Package was also analyzed for data from 1990-2005 in the original MODFLOW model. The General Head Boundary Package is only used in the bottom layer of the original model and only in four cells where the aquifer is assumed to be connected to the Spokane River downstream of Nine Mile Dam (Figure 25). From 1990-2005, all the values in August, September, and October fall within about 0.4 feet of each other, and within about 0.3 feet of each other in October. Since this boundary condition seems to be fairly insensitive to changes in head in the aquifer the average between values from October of 1991 and October 2004 were used. October 1991 was used because both August and September where exactly the same as in 2005 and October 2004 was used because of the similarity between precipitation and pumping rates for the same period.

The values for the River Package needed to be determined in both September and October due to the change in time steps from monthly to every fifteen minutes. At the monthly time steps only the average value for the month was used to determine the head in the river for a given time period. To determine the effect a flood wave that took a little over 18 hours to travel about 47 miles, monthly time steps could not be used. Figure 25 shows the locations where water surface elevation measurements were taken to establish when and how much change the artificial flood wave caused.

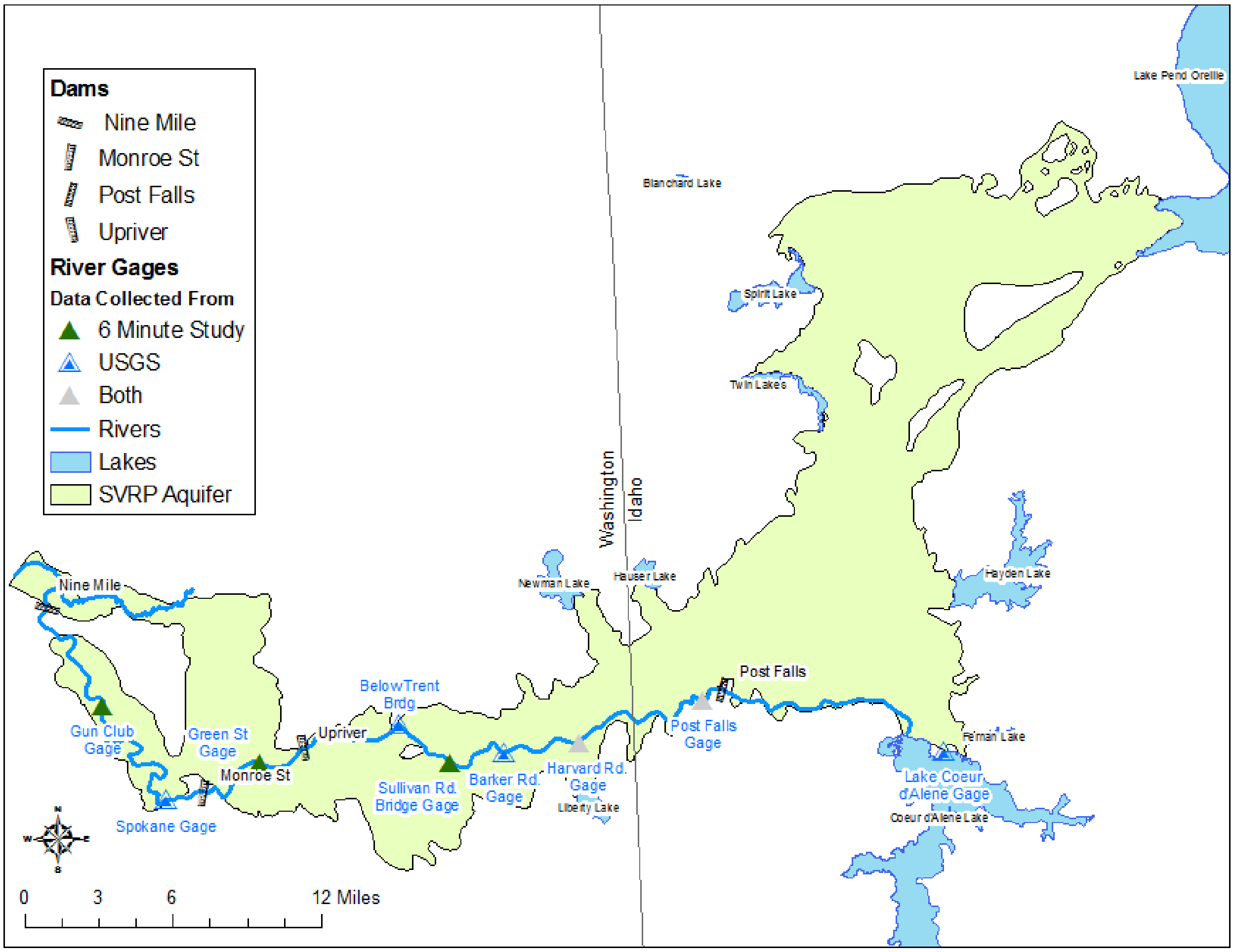


Figure 25. Location of water surface elevation measurements along the Spokane River

For the various dams shown in Figure 25 water surface elevations at the forebay were available from Avista Corporation (Patrick, 2011) at eight hour intervals. Water surface elevations are available at hourly interval for Long Lake Dam (not shown, but located downstream of Nine Mile) (Patrick, 2011). The elevations were completed with the City of Spokane Datum and were converted to NAVD 88. The forebay of the Monroe Street Dam is located between two water falls in downtown Spokane, Washington. The water surface elevation here is of little consequence because the forebay is not in the model domain and the waterfall immediately upstream, Upper Falls, with its associated check dam controls the water surface elevation in the model domain. The change in water surface elevation however, is assumed to be similar for both locations. This allows for the changes in the water surface elevation in the forebay at Monroe Street Dam to be used on the upstream Upper Falls Check Dam.

The Below Trent Bridge Gage was not operational until 2010. However, correlations with flows at the Barker Road Gage (USGS Gage 12420500) from September to October of 2010 provide a range of flow rates in the Spokane River that encompasses those for September and October of 2005. Using a lag from the Barker Road Gage determined to be about 2.5 hours and the correlation equation from the 2010 data, water surface elevations at the Below Trent Bridge Gage were estimated. The power correlation equation is shown below and produces a R^2 value of 0.999.

$$y = 0.72629605282404\mathrm{x}^{1.03726202304371} \qquad \text{Equation 15}$$

where y is the resulting water surface elevation at the Below Trent Bridge Gage and x is the water surface elevation from the Barker Road Gage. The power correlation equation has a large number of significant figures which is not meant to imply the water surface elevation are known to those values, but are needed to get the correct values to one decimal place when applying the equations.

The Sullivan Road Bridge Gage was pulled out of the Spokane River on September 21, 2005. The MODFLOW simulation run goes to 11:45 am on October 20, 2005. This caused a gap in the data that was fixed using a correlation to Post Falls Gage from Hsieh et al. (2007) and is given as:

$$y = 2.55(x - 2050.88)^{0.8} + 1924 \qquad \text{Equation 16}$$

where y is the water surface elevation at the Sullivan Road Bridge Gage and x is the water surface elevation at the Post Falls Gage.

The 6-Minute Data Study did not provide a reference elevation at the locations of the gages in the Spokane River. To overcome this, the water surface elevations given in the MODFLOW model (average of September 2005) were compared to the average September 2005 relative depth in the Six-Minute Data. Subtracting the average relative depth of the Six-Minute data from the MODFLOW model data the elevation of the water surface was estimated.

The River Package was also used for Lake Pend Oreille, Lake Coeur d'Alene, and the Little Spokane River. All three packages in the original MODFLOW model had the same water surface elevations for all September and October time steps, thus the original MODFLOW values were used. This ensured that any differences in the results would be from the flood wave in the Spokane River and not changes to other areas not affected by the flood wave.

3.5 Sensitivity Analysis

The sensitivity analysis is used to systematically test the influence of parameters in the model that are not known definitively before hand or are established through calibration. This also establishes the accuracy of individual parameters that must be known for reliable model results. For the purpose of this study the sensitivity analysis will be completed by testing influence of changing various model inputs and holding all others constant. To quantify the various changes, the NSE (Equation 12), bias (Equation 13), and relative error (Equation 14) values will be determined and the variation from the best fit model established.

Completion of this sensitivity analysis is based on establishing the model parameters for K' and S_y and then determining the influence of the other parameters once K' and S_y are estimated. Each parameter in question will be changed by 10% in both directions (Hossain, 2011) and rerun model with all other parameters held constant. The sensitivity will then be computed by determining the percent change in the NSE value from the best fit NSE values for K' and S_y. Therefore, it is possible that a change in one of the input parameters might cause a better fit to the data. The analytical solutions by Barlow and Moench (1998), Zlotnik and Huang (1999) and Hantush (2005) along with the HYDRUS models will be used in the sensitivity analysis. Since the sensitivities of the values in the MODFLOW model have already been established by Hsieh et al. (2007). In all the analytical models the values of K' and S_y are grouped into dimensionless parameters to reduce the complexity of the solution. Therefore, the direct affect of K' and S_y are not known, but the affect of the dimensional parameter which includes those values can be determined. In Table 8 the dimensionless parameters of the analytical solutions that were used in the sensitivity of the solution are shown.

Table 8. Sensitivity analysis altered parameters list

			Area 1		Area 2		Area 3		Area 4	
Solution	Parameter	Units	Increase	Decrease	Increase	Decrease	Increase	Decrease	Increase	Decrease
STWT 1	a	ft	13537	11075	19999	16363	2054.3	1680.8	1741.3	1424.7
	σ	[]	0.0118	0.0097	0.0151	0.0123	0.0125	0.0103	0.015	0.0123
	x_D	[]	71.68	58.65	3.883	3.177	1.87	1.53	16.26	13.3
A2	ξ	[]	0.009233	0.007555	0.009121	0.007463	0.01681	0.01408	0.0799	0.06537
	xbar	[]	71.6	65.09	3.88	3.174	1.869	1.529	16.26	13.3
A3	R	ft	46845	38327	47417	38796	–	–	–	–
	D	ft^2/day	26232984	21463350	22048950	18040050	–	–	–	–
	η	days	7.7	6.3	3.3	2.7	–	–	–	–
	ξ	[]	0.33	0.27	0.11	0.09	–	–	–	–

	α	1/ft	1.84	1.51	1.84	1.51	−	−	−	−
	n	[]	2.288	1.872	2.288	1.872	−	−	−	−
	Ksat	ft/day	24360	19931	21004	17185	−	−	−	−
HYDRUS	K'sat	ft/day	3.937	3.215	2.756	2.231	−	−	−	−
	Grid Size	ft	0.902	0.738	0.902	0.738	−	−	−	−
	Depth to Water Table	ft	56.96	46.59	27.43	22.44	−	−	−	−

− is an area that was not able to be run in the scenario due to lack of seepage (ft3/s) data

Both the analytical solution by Barlow and Moench (1998) and HYDRUS use numerical techniques to solve part or all of the governing equations used. Barlow and Moench (1998) use the Stehfest algorithm to numerically invert the resulting solution from the Laplace domain into the real-time domain using the Newton-Raphson iteration and summation scheme. The three variables used are the number of terms in the Stehfest algorithm, the relative error for the Newton-Raphson iteration and summation, and the factor to determine the number of terms in the finite sums for head and seepage (Barlow and Moench, 1998). Suggested values from the same paper were 8, 1E-10, and 20, respectively. These values were increased and decreased to determine if the suggested values were sufficient for this study. In HYDRUS the tolerance of the water content or head is used between two successive iterations in the unsaturated and saturated zones respectively. The solution at each node is subject to the tolerance prescribed by these variables and as such the water content and head tolerance were tested to verify the precision required to overcome numerical approximation errors.

Chapter 4: Results/Discussion

4.1 Introduction

This chapter presents the results of the analytical and numerical modeling. The results investigated how each model represented seepage across the riverbed sediments and the influences of the river-aquifer interaction. The scenarios and parameters employed are outlined and discussed in the previous section. Results of each scenario, with descriptive statistics, are presented and analyzed through graphs relating the field data to the model outputs.

4.2 Barlow and Moench (1998) (STWT1) Scenario Results

Four areas were used in the Barlow and Moench (1998) (STWT1) solution. Comparisons with the 6-minute head data and 15-minute seepage rates were completed with appropriate boundary and initial conditions. The observation wells in each area are presented in turn and the seepage rates for Areas 1 and 2 are shown for the respective area. Values for K' and S_y from Hsieh et al. (2007) are compared with the calibrated values from STWT1. Figure 8 shows the names of the observation wells for each of Areas 1–4, and the river gage sites used to establish the hydrograph in each area.

Area 2 is shown in Figure 8 to be very narrow. The definition of the area was used only to determine the aquifer parameters (see Table 4). In the STWT1 solution the width of the aquifer (width of Area 2) is used to determine total seepage, however both seepage per foot of channel and total seepage are given in the result file. Multiplying the seepage per foot of channel and the actual length of the Spokane River between Barker Road Gage and Harvard Road Gage provided the total seepage. For Area 1 a similar procedure was done, because the length of the river between Post Falls Gage and Harvard Road Gage is greater than the straight line width used to define Area 1. In Areas 3 and 4 seepage was not used in the calibration process since data was not available, thus only head in the aquifer is used to calibrate the STWT1 solution.

4.2.1 Area 1 (STWT1) Scenario Results

Figure 26 - Figure 28 show the results of the calibrated K' and S_y for Area 1 along with the values from MODFLOW parameter (Hsieh et al. (2007)) (Table 3). The measured data for wells 13, 15, and 16 are also shown for comparison.

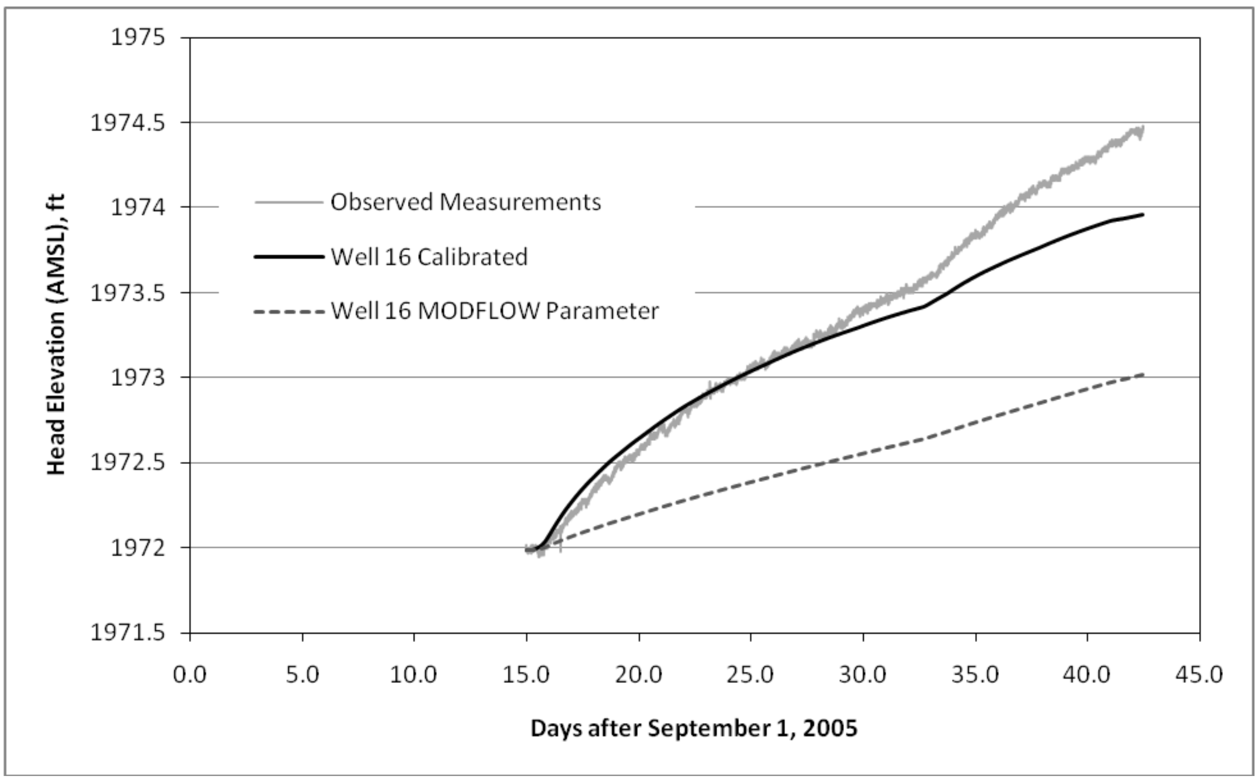


Figure 26. STWT1 results for Well 16 using calibrated and MODFLOW K' and S_y values.

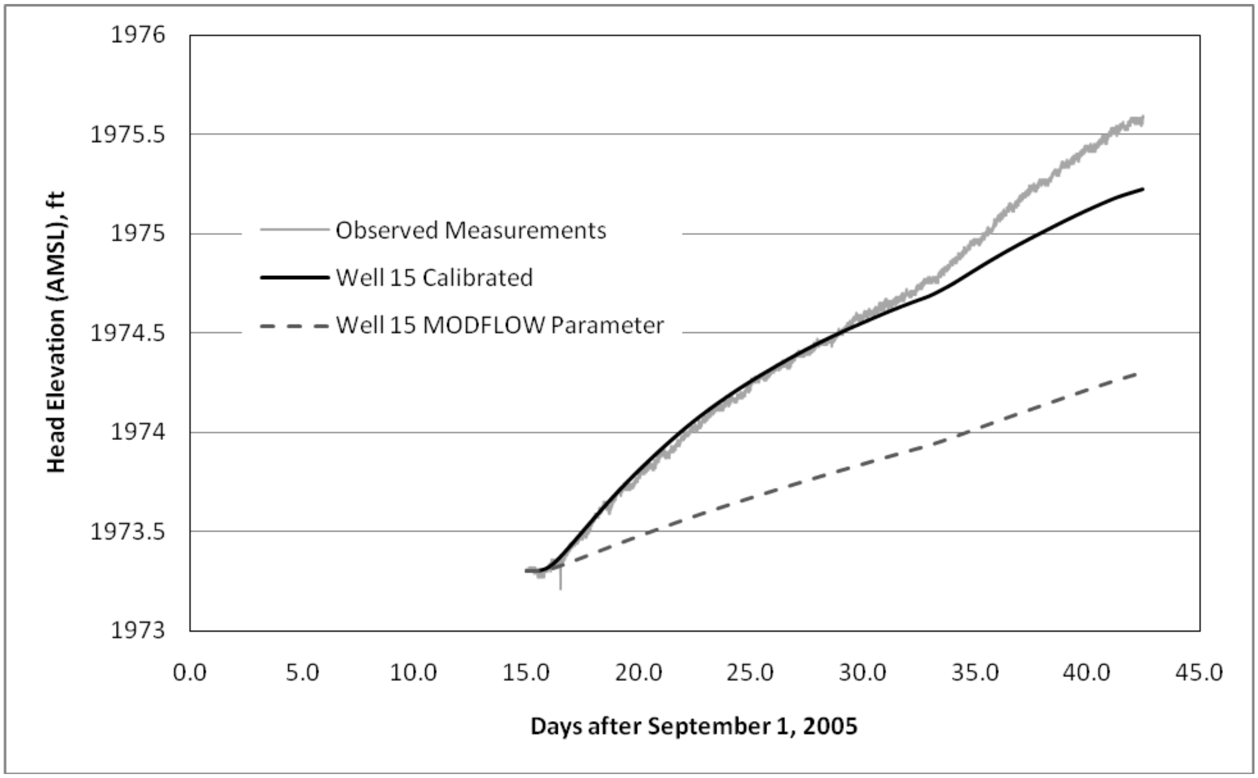


Figure 27. STWT1 results for Well 15 using calibrated and MODFLOW K' and S_y values.

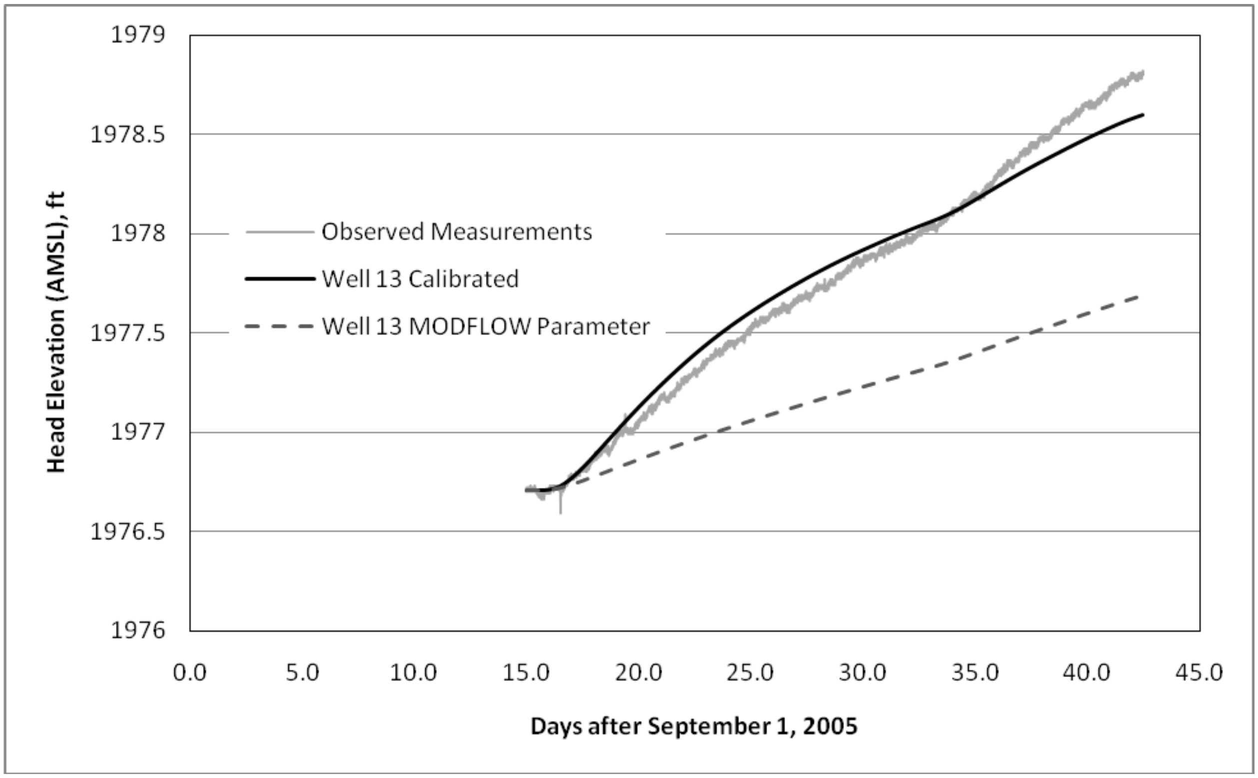


Figure 28. STWT1 results for Well 13 using calibrated and MODFLOW K' and S_y values.

The MODFLOW values for K' and S_y are not able to reproduce the overall changes in the aquifer or the distinct second jump in the observed measurements. The calibrated STWT1 values for K' and S_y reproduce a good fit to the initial increase and show the distinct jump but not the magnitude of the observed data. Table 9 reports the values of NSE and other descriptive statistic. The values above 0.9 for all three graphs confirm the overall "goodness of fit" to the observed data.

Table 9. Descriptive Statistics for Area1 Wells.

		NSE	$\bar{e}$	R_b %	min, ft	K'	K	S_y	a
Calibrated	Well 16	0.907	-0.117	-8.69%	1971.95	1.8	22150	0.3	12306
	Well 15	0.950	-0.076	-6.00%	1973.21				
	Well 13	0.978	0.007	0.57%	1976.59				
	Seepage	-10.300	-106.928	-43.54%	11				
MODFLOW	Well 16	-0.607	-0.785	-58.21%	1971.95	0.25	22150	0.19	87572
	Well 15	-0.385	-0.683	-54.00%	1973.21				

Well 13	-0.111	-0.566	-48.88%	1976.59	
Seepage	-29.067	-199.562	-81.27%	11	

ē is in the unit of the criteria variable (head = ft, seepage = cfs)

ē shows that for wells closer to the river (e.g. Wells 15 and 16) the calibrated K' and S_y from the STWT1 solution are lower than the observed data and the model results overestimate observed data for Well 13. The relative error is used here represents ē divided by the quantity of the average value of the observation data subtracted from the minimum value of the observation data. This relates the magnitude of the bias to the change in the water table height caused by the flood wave. Thus instead of using the full elevation of the water table in the well (over 100 feet in most cases), only the change water table from the flood wave is used. McCuen et al. (2006) give the recommendation that a relative bias of greater than 5% is significant. Therefore, both Wells 15 and 16 have significant bias and an explanation is required. As seen in Figures 26-28, the response of the wells further away from the river is dampened compared to Well 16 (closest to the river). In the assumptions for the STWT1 solution, it was stated that the water is moving perpendicular to the channel to the channel. This assumption is not fully satisfied in the SVRP aquifer since the main direction of flow is east to west, parallel to the river. However, Area 1 is a strictly losing reach and as such induces a significant gradient perpendicular to the river. This has two implications: 1) the possibility for flow from upstream contributing to the overall response of the observation wells, causing a larger increase that would be expected from two-dimensional flow and 2) wells far away are more influenced by the regional flow and the expected rise in the observation wells from flow perpendicular to the Spokane River over predicts the response. Both the wells near the Spokane River and those at a greater distance from it show a response to the second pulse from the artificial flood wave originating at the Post Falls Dam (Figure 8) than the STWT1 solution predicts. Therefore, the most probable explanation would appear to be the influence of upstream flow or three-dimensional flow patterns. Figure 29 shows the flood wave as it passes the flow gages along the Spokane River. All the heights change to within about two feet of the gage upstream of it, with Post Falls gage showing the true water surface elevation of the Spokane River. A larger rise was observed at the Sullivan Road Bridge Gage than any other gage, which was due to the gage being pulled out of the water by a passerby and then replaced. The gage then seemed to slide a few times as it found a new position of the river bottom and then captured an increase in river elevation about double that of the Post Falls Gage. The river does narrow in this area, but with the Sullivan Road Bridge Gage being moved and moving along the river bottom multiple times after it was replaced, to determine if the larger rise was because of the narrower region of the river or if the passing flood wave moved the gage to a deeper location is difficult. The purple line was used in some calculations but the regression equation (Equation 16) provided by Hsieh et al. (2007) was used to produce a longer time series that matched known data. The Green Street Gage is below Upriver Dam, which is the first dam after Post Falls Dam, and shows that the sharp front of the flood wave was slightly damped and the second pulse of the flood wave was more so.

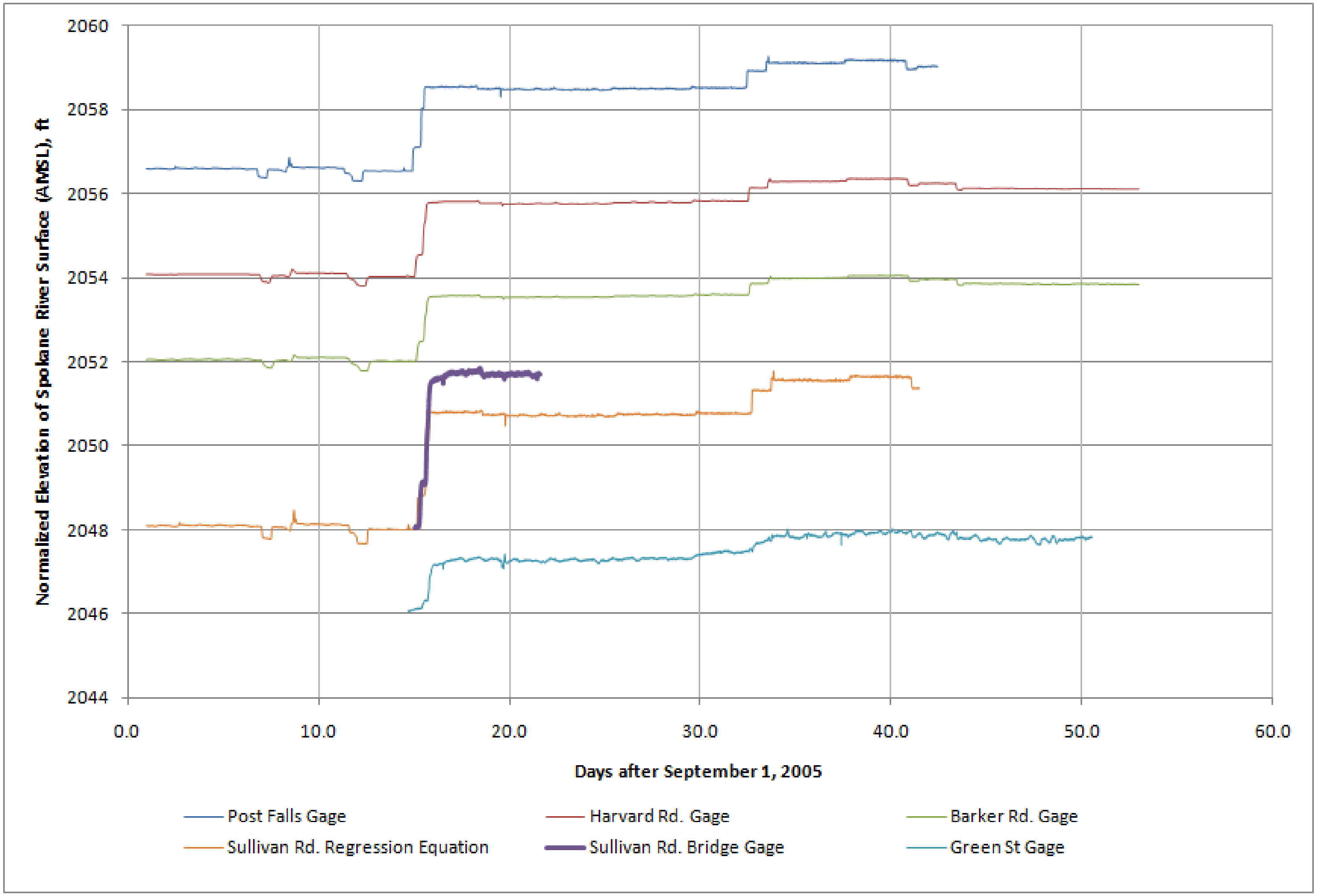


Figure 29. Flood wave at gaging stations on the Spokane River, all gages except Post Falls have modified surface elevations.

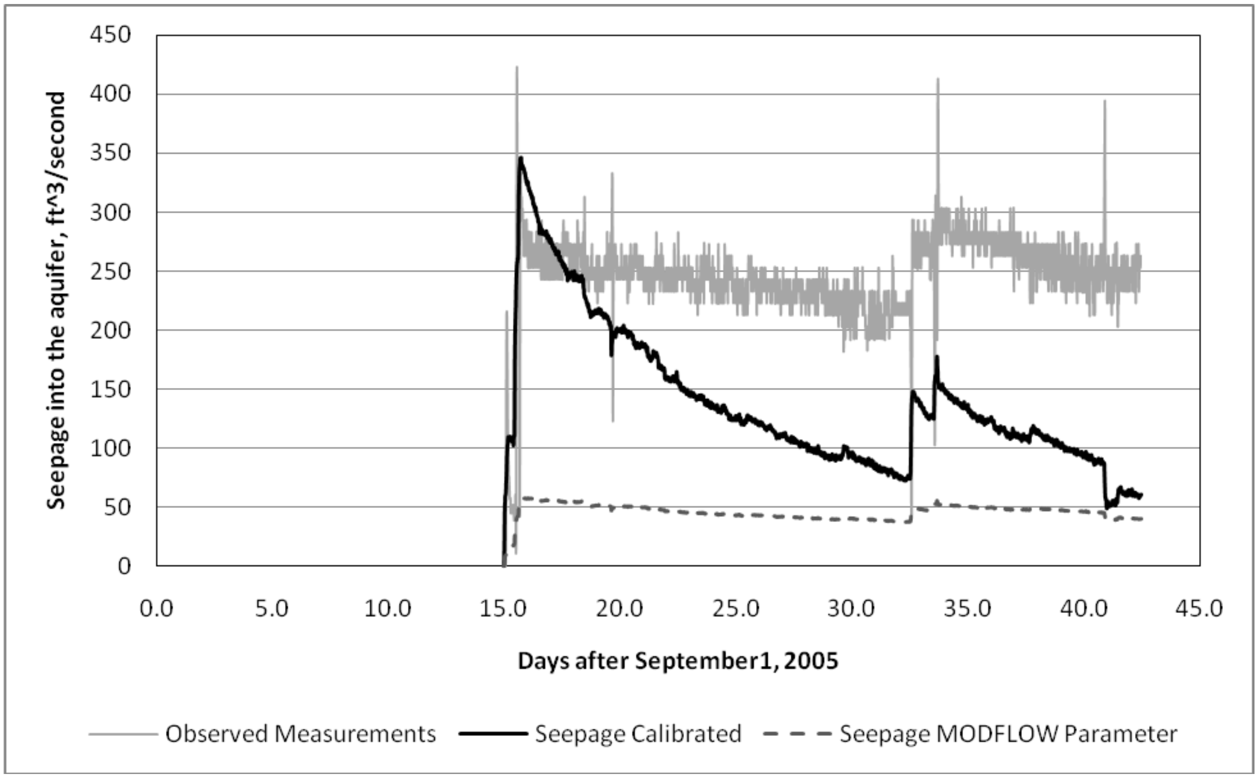


Figure 30. Seepage results from STWT1 for calibrated and MODFLOW parameter values.

Figure 30 shows the results from the STWT1 solution for seepage into the aquifer. The fit for both the MODFLOW values and calibrated values of K' and S_y are very poor. The observed measurements started at zero by removing a baseline seepage rate that was present before the flood wave passed the observation wells. The jump in the observed measurements is the passing of the flood wave. The STWT1 solution assumes the water table is initially equal to the water surface elevation. This accounts for the reduction of seepage after the leading edge of the flood wave passed; the elevation of the water table is closer to the river surface in the STWT1 model than in reality. Recognizing this, the initial increase in seepage was used as the measure of the ability to reproduce the observed values with STWT1. The calibrated values reproduced the initial increase in seepage for both flood wave pulses. The low NSE and high bias values in Table 9 for seepage are used to distinguish between the various K' and S_y values used in the calibration process versus a measure of the goodness-of-fit to the observed seepage data.

4.2.2 Area 2 (STWT1) Scenario Results

Area 2 is also a losing reach, similar to Area 1. This means that the initial assumption of the system at rest has the same influence on seepage. Also the response in the wells in Area 2 are similar to Area 1, as flow from upstream reaches cause three dimensional effects as the duration of the wave lengthens. Figures 31-36 show the results for Area 2.

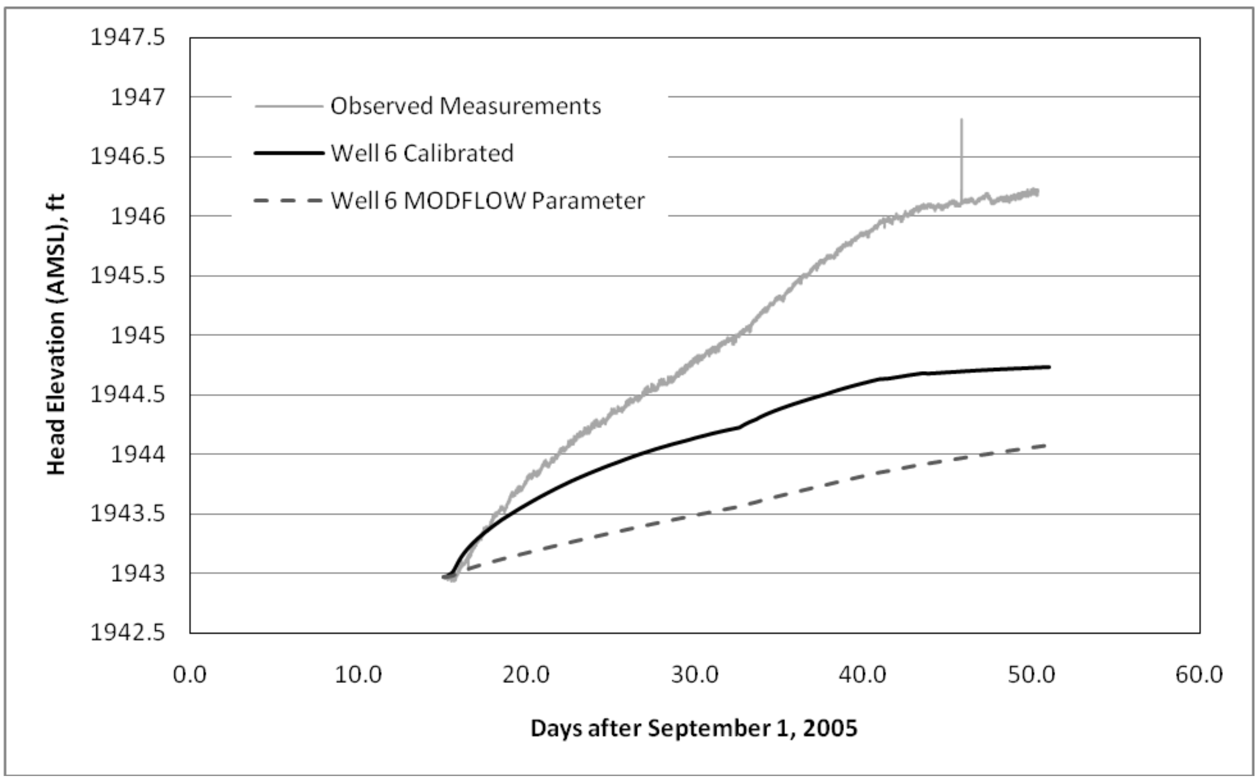


Figure 31. STWT1 results for Well 6 using calibrated and MODFLOW K' and S_y values.

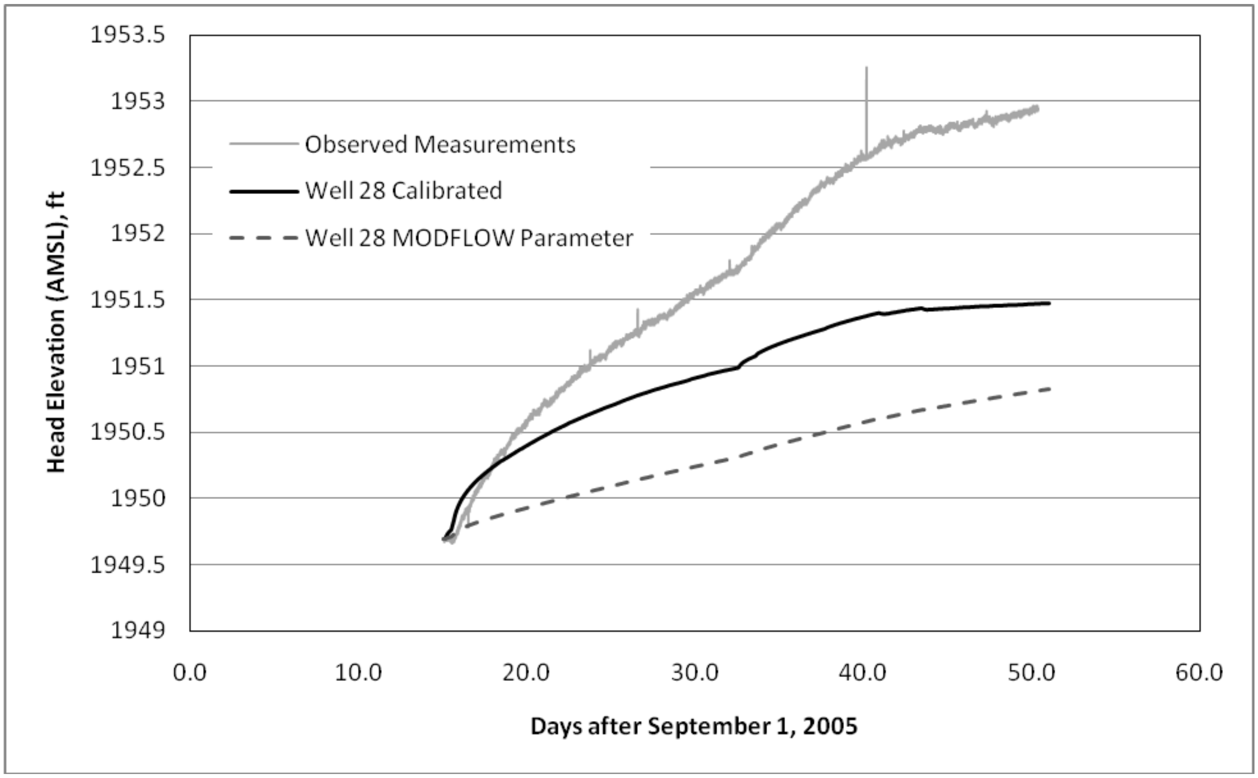


Figure 32. STWT1 results for Well 28 using calibrated and MODFLOW K' and S_y values.

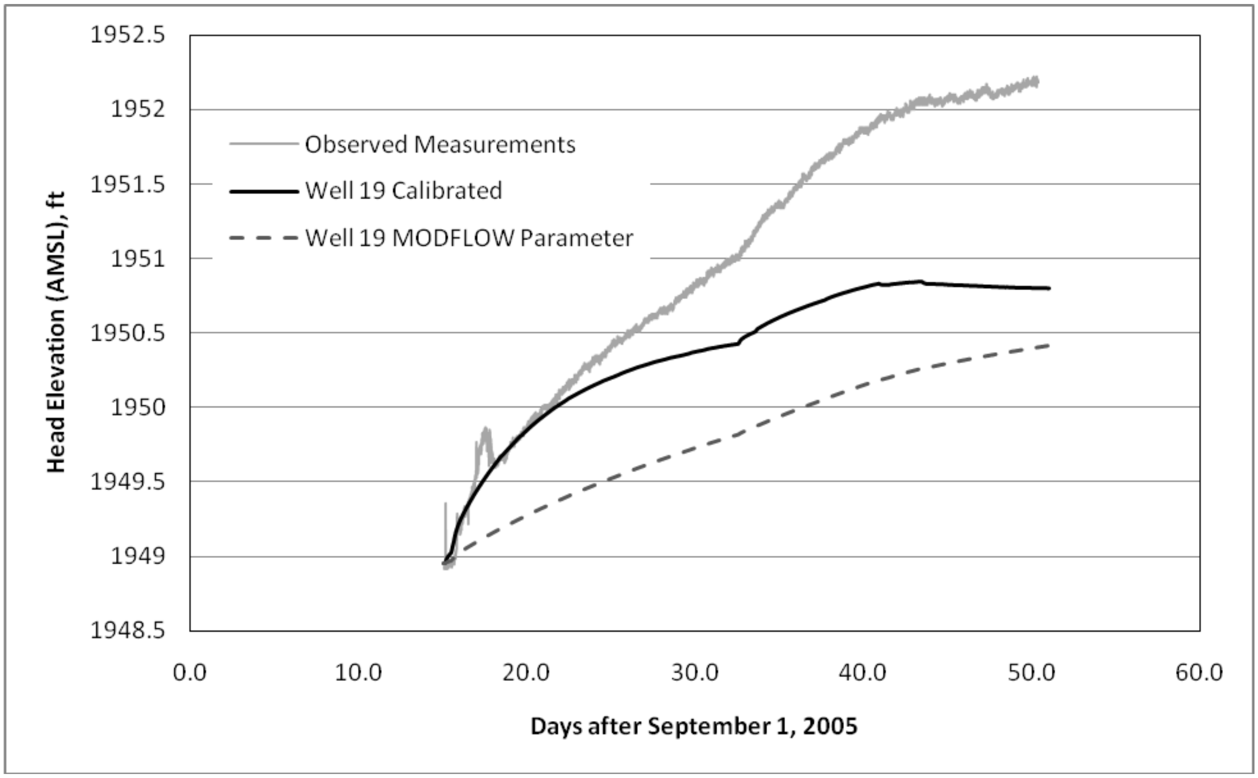


Figure 33. STWT1 results for Well 19 using calibrated and MODFLOW K' and S_y values.

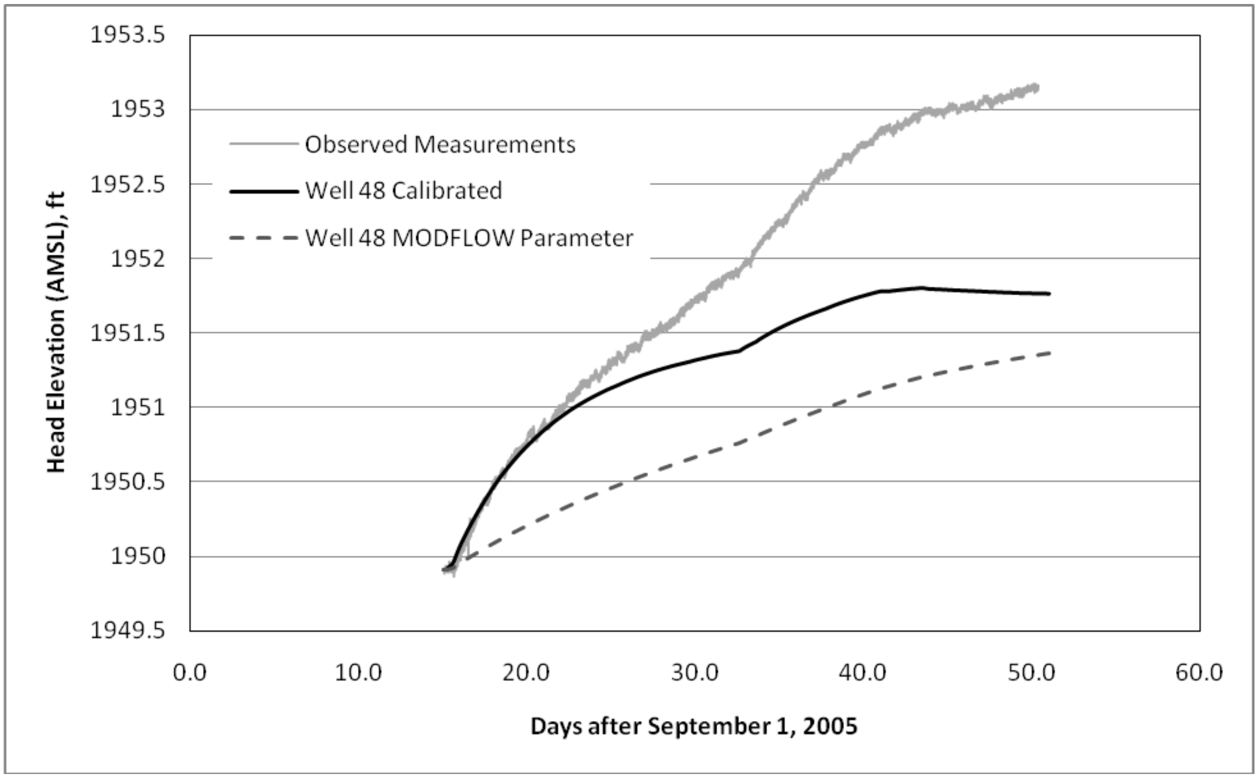


Figure 34. STWT1 results for Well 48 using calibrated and MODFLOW K' and S_y values.

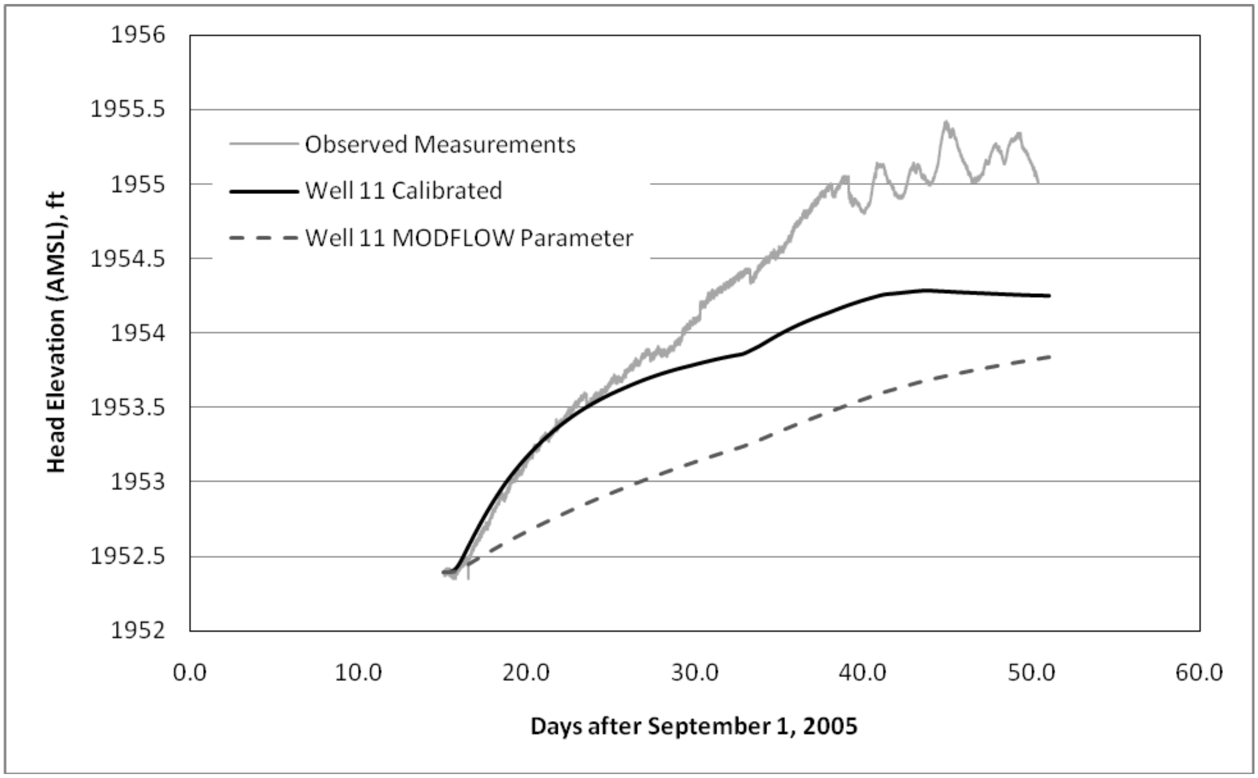


Figure 35. STWT1 results for Well 11 using calibrated and MODFLOW K' and S_y values.

Figures 31-36 show a pattern of initially estimating measured data well and underestimating data after the September 25. For Wells 6 and 28, both on the south side of the Spokane River, the model underestimates measured data after approximately five days. The model underestimates measured data for Wells on the north side of the Spokane River after about ten days. The difference between the two sides of the Spokane River may indicate that the Spokane River affects the regional flow pattern. Well 11, which is furthest away from the Spokane River, matches the predicted values from the STWT1 equation for the longest period of time. Initially, days 15-20, the model slightly over estimates the result for Well 11 and might indicate that the K of the aquifer decreases as you move towards the edge of the river. However, the over estimate is trivial when compared to the assumptions of the STWT1 equation that are different from situ conditions.

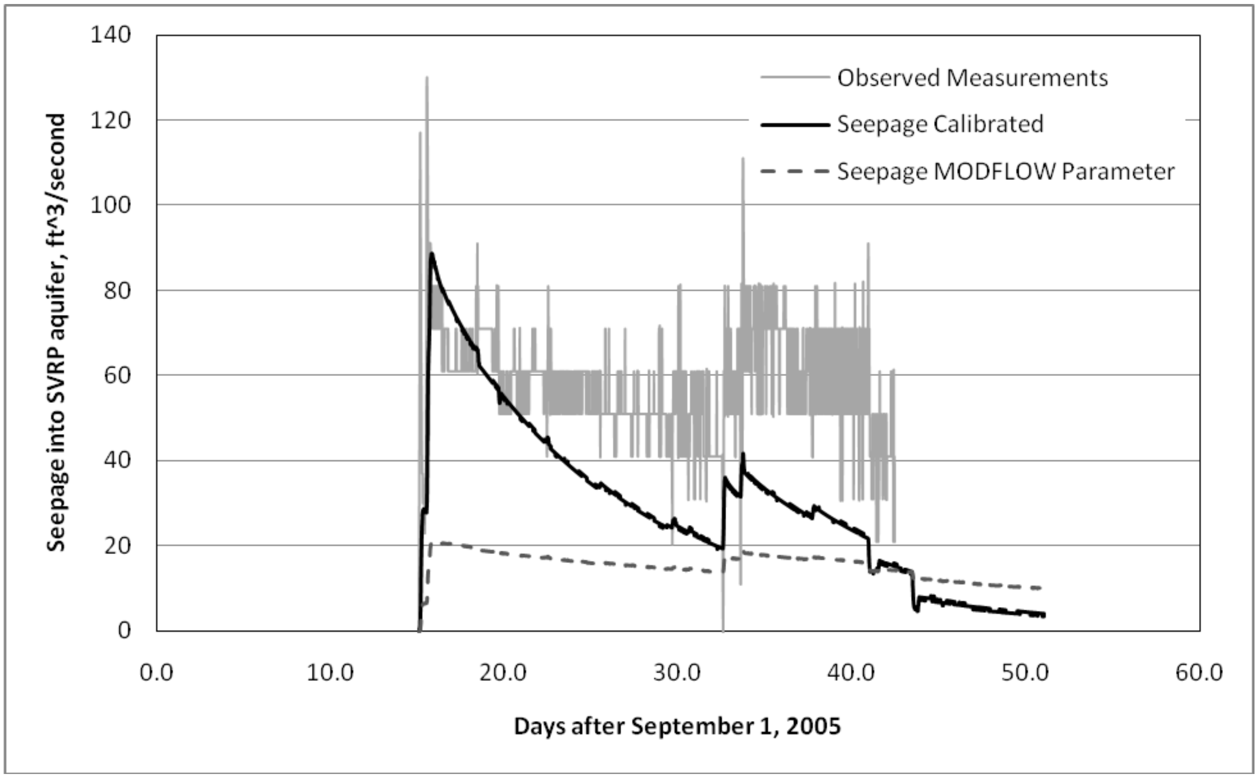


Figure 36. STWT1 results for Seepage using calibrated and MODFLOW K' and S_y values.

The fit to the seepage data from the STWT1 equation is very similar to that of Area 1 and the same method of looking only at the initial change from the flood wave is used. The initial changes match the difference between the pre-flood wave condition and immediately after the flood wave passes, but the losing condition of the stream maintains a seepage rate much greater than predicted by the STWT1 equation. One of the reasons that the seepage rates were still included despite the poor match was that head alone did not provide a unique solution. This is an important finding and will be presented in more detail in the discussion later in Chapter 4, but is pointed out here to provide reasoning for including such matches as in Figure 36.

Table 10 shows the overall fit to the Area 2 data is lower than for Area 1 and the bias is an order of magnitude higher. The distance to the wells in Area 1 is greater than the distance in Area 2 and the length of the Spokane River upstream of Area 2 is also greater than the upstream length in Area 1. Three-dimensional flow is therefore probably a larger factor in Area 2 than in Area 1.

Table 10. Area 2 Descriptive statistics from STWT1 equation

		NSE	ē	R_b %	min, ft	K'	K	S_y	a
Calibrated	Well 6	0.077	-0.794	-38.8%	1942.9	1.05	19090	0.23	18181
	Well 28	0.049	-0.779	-37.7%	1949.7				
	Well 19	0.239	-0.641	-29.9%	1948.9				
	Well 48	0.344	-0.597	-28.5%	1949.9				
	Well 11	0.547	-0.443	-23.9%	1952.4				
	Seepage	-7.591	-32.788	-56.9%	-9				
MODFLOW	Well 6	-1.536	-1.406	-68.71%	1942.9	0.218	19090	0.19	87569
	Well 28	-1.682	-1.416	-68.53%	1949.7				
	Well 19	-1.067	-1.219	-57.85%	1948.9				
	Well 48	-0.819	-1.165	-57.07%	1949.9				
	Well 11	-0.620	-1.003	-53.98%	1952.4				
	Seepage	-10.215	-41.115	-71.39%	-9				

ē is in the unit of the criteria variable (head = ft, seepage = cfs)

4.2.3 Area 3 (STWT1) Scenario Results

As Figure 29 and the proceeding discussion pointed out the change in depth of the Spokane River at Sullivan Road Bridge Gage is not well established nor the height of the water surface. Using the 6-Minute data and the MODFLOW parameter values, the curve shown in Figure 37 was produced. The curve is well above the observation well values. Another reason for the large difference between the observation data and the curve produced by the MODFLOW parameter values is the stream surface elevation is lower than the water table elevation. This can be resolved by moving the starting elevation of the STWT1 equation curve to the initial elevation of the Spokane River water surface. When the change in river stage is high enough to equal the elevation in the aquifer the STWT1 solution is also at the elevation of the water table. This could also be accomplished by starting the solution at the time the river water surface elevation and the water table elevation are equal. Removing the part of the curve where the river water surface is below the water table elevation helps to reduce the error but is not enough to overcome the large difference between the MODFLOW parameters and the observed data. The water surface elevation is about 0.3 ft below the water table elevation when the flood wave passes. In the calibrated solution the regression equation for the Sullivan Road Bridge Gage is used with a 4.46 ft correction to get the elevation at the location of the data logger. The elevation of the water table in the aquifer initial starts at about 1930 ft, the calibrated solution starts at 1929.7 ft to overcome the 0.3 foot difference in the initial conditions. Using the two corrections to the Sullivan Road Bridge Gage, the calibrated parameters match well. One piece that may be missing is the influence of flow from upstream reaches. One explanation is the errors in the two applied corrections mask the upstream influence. Another is the river channel turns northwest at this point. Starting just before Area 3 and continuing downstream for about 3 miles the Spokane River runs northwest; this allows for flow to be nearly perpendicular to the channel and possibly reduces the violation of the assumptions in the STWT1 equation. The actual causes cannot be assessed without a more accurate initial data set. For that reason Well 46 was not included in the analysis in Area 3.

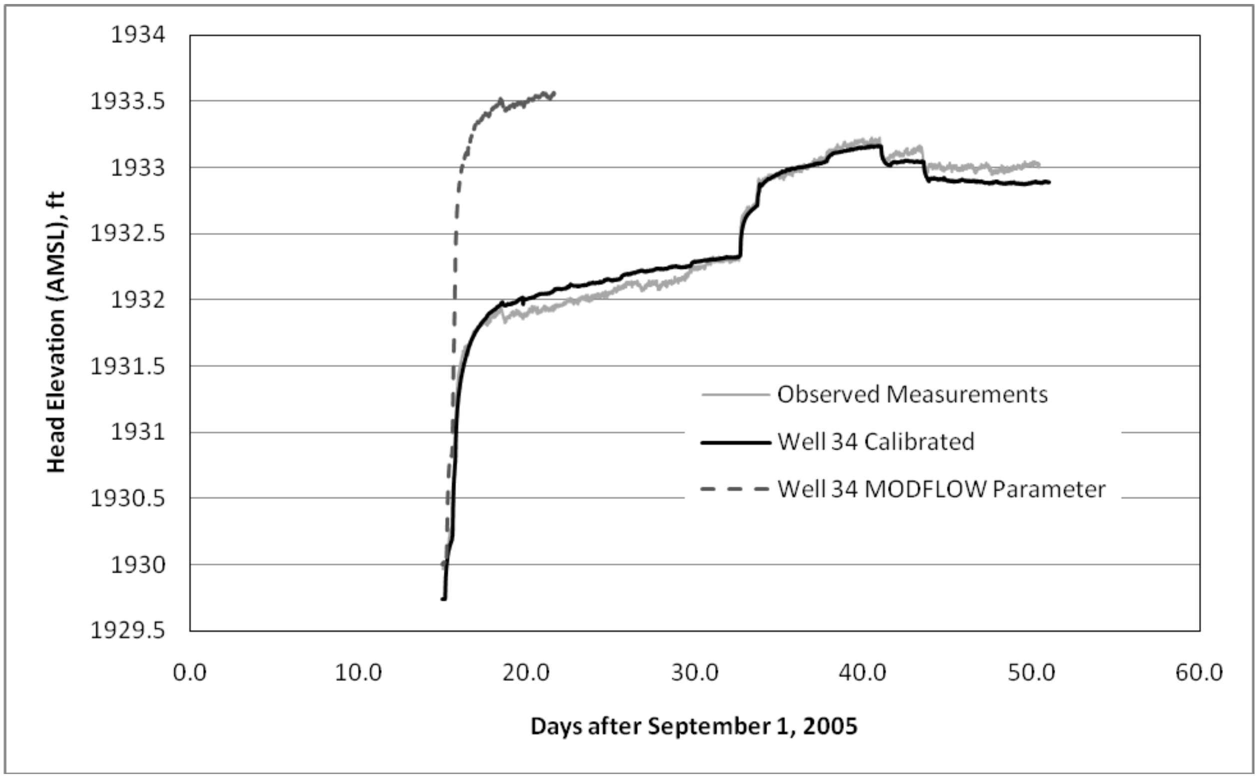


Figure 37. STWT1 results for Well 34 using calibrated and MODFLOW K' and S_y values.

Table 11 highlights that given the limited accuracy of the data for the Spokane River in Area 3, the MODFLOW values of K’ and S_y are different from the calibrated parameters. It was found that even for the regression equation, the values of MODFLOW K’ and S_y for the Sullivan Road Bridge Gage produced curves much higher than the observed measurement.

Table 11. Area 3 Descriptive statistics from STWT1 equation

		NSE	ē	R_b %	min, ft	K'	K	S_y	a
Calibrated	Well 34	0.980	-0.003	-0.11%	1929.97	4	7470	0.3	1868
	Well 49	-	-	-	-				
MODFLOW	Well 34	-6.267	1.385	82.48%	1929.97	9.4	7470	0.19	794
	Well 49	-	-	-	-				

ē is in the unit of the criteria variable (head = ft, seepage = cfs)

4.2.4 Area 4 (STWT1) Scenario Results

In Area 4, Wells 31 and 18 were closest to the Spokane River and responded similar to the wells in other areas. The STWT1 solution models the first ten days of data well, then the predicted solution underestimates the measured data due to the influence of upstream flow. Wells 39 and 32, farthest from the Spokane River, respond very differently than other areas. It appears that an additional abstraction from the aquifer near Wells 39 and 32 is present. This observation is based on the concave departure from the STWT1 solution after the first ten days. The duration of the abstraction appears to be less than ten days. The observation data shows a steeper slope than STWT1 solution near the end of September. For Well 31 (Figure 38) the match to the observed data for the first ten days is very good. Well 18 (Figure 39) also shows a good match, with a slight difference in the slope of the initial rise and a possible influence from the abstraction.

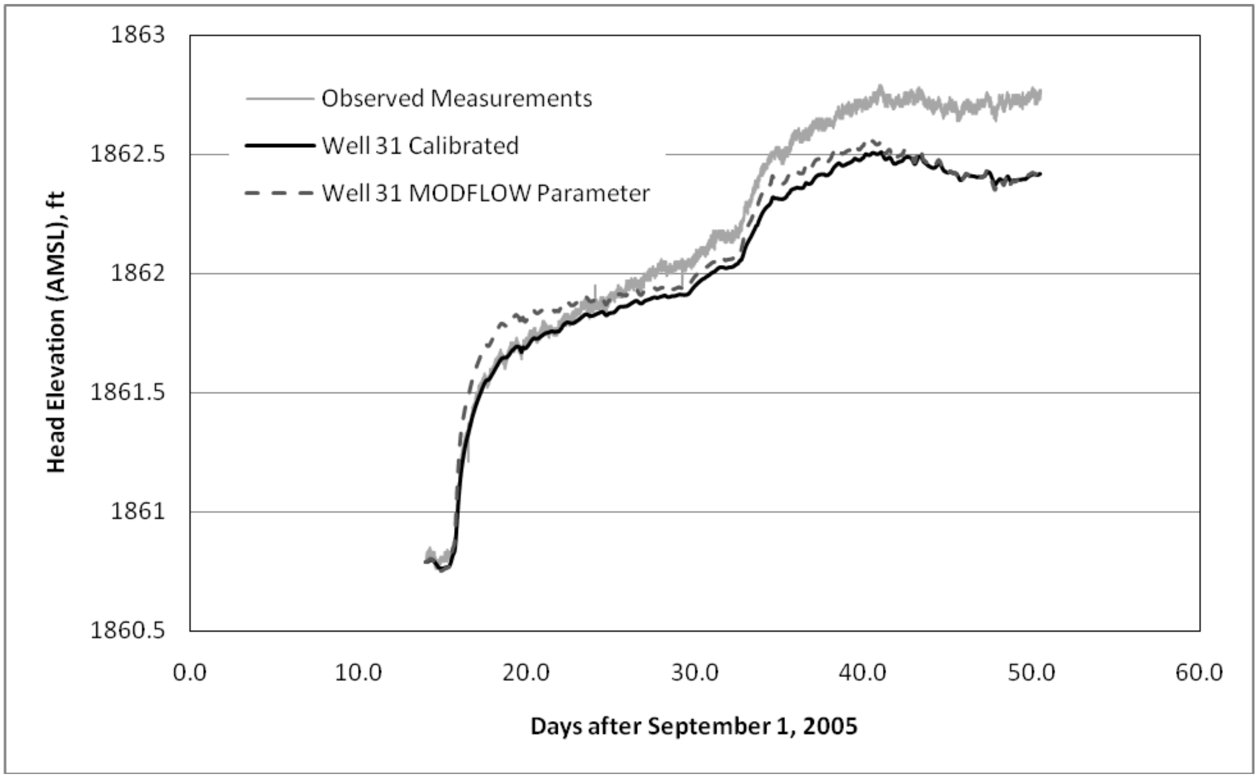


Figure 38. STWT1 results for Well 31 using calibrated and MODFLOW K' and S_y values.

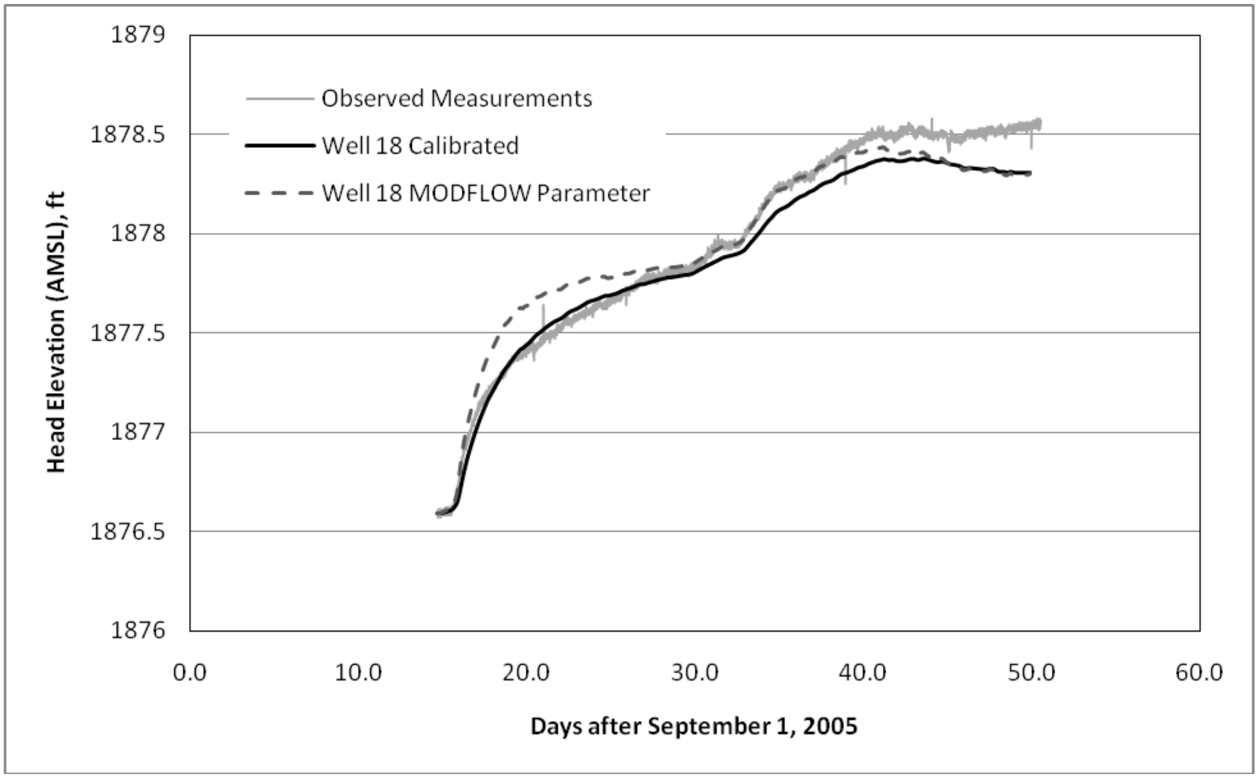


Figure 39. STWT1 results for Well 18 using calibrated and MODFLOW K' and S_y values.

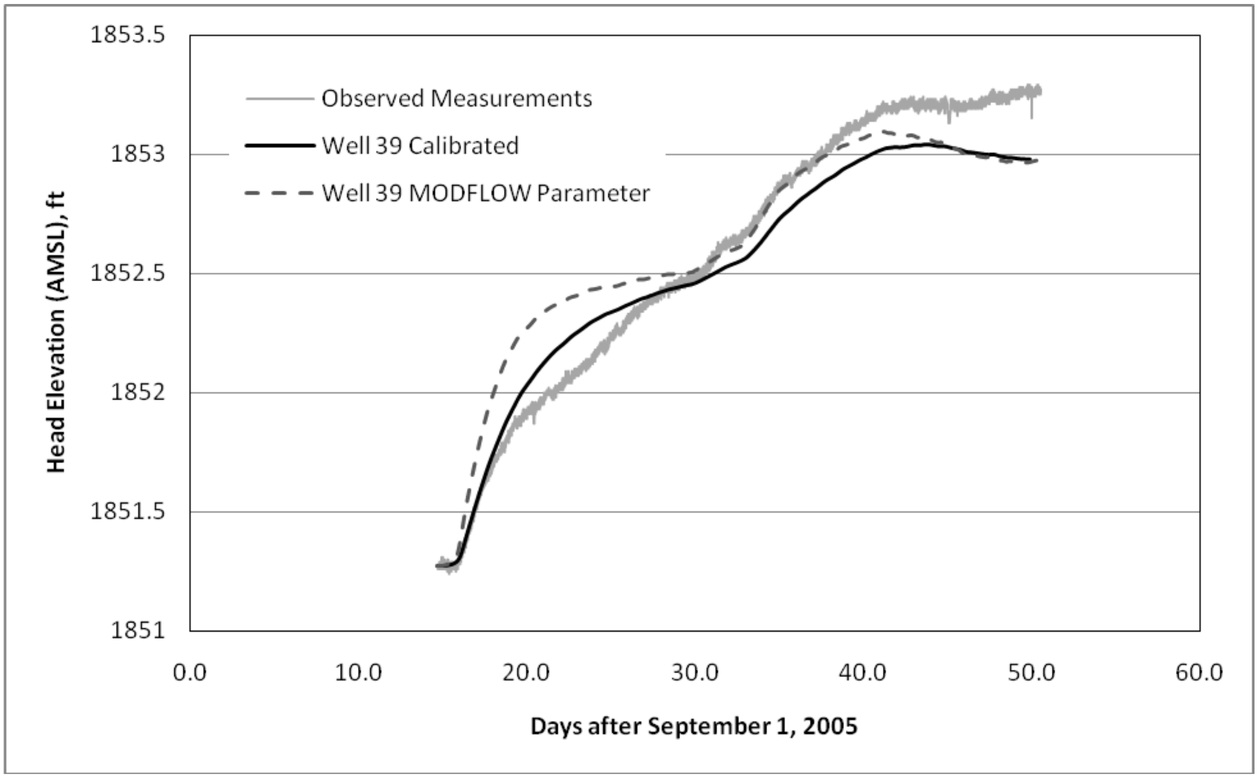


Figure 40. STWT1 results for Well 39 using calibrated and MODFLOW K' and S_y values.

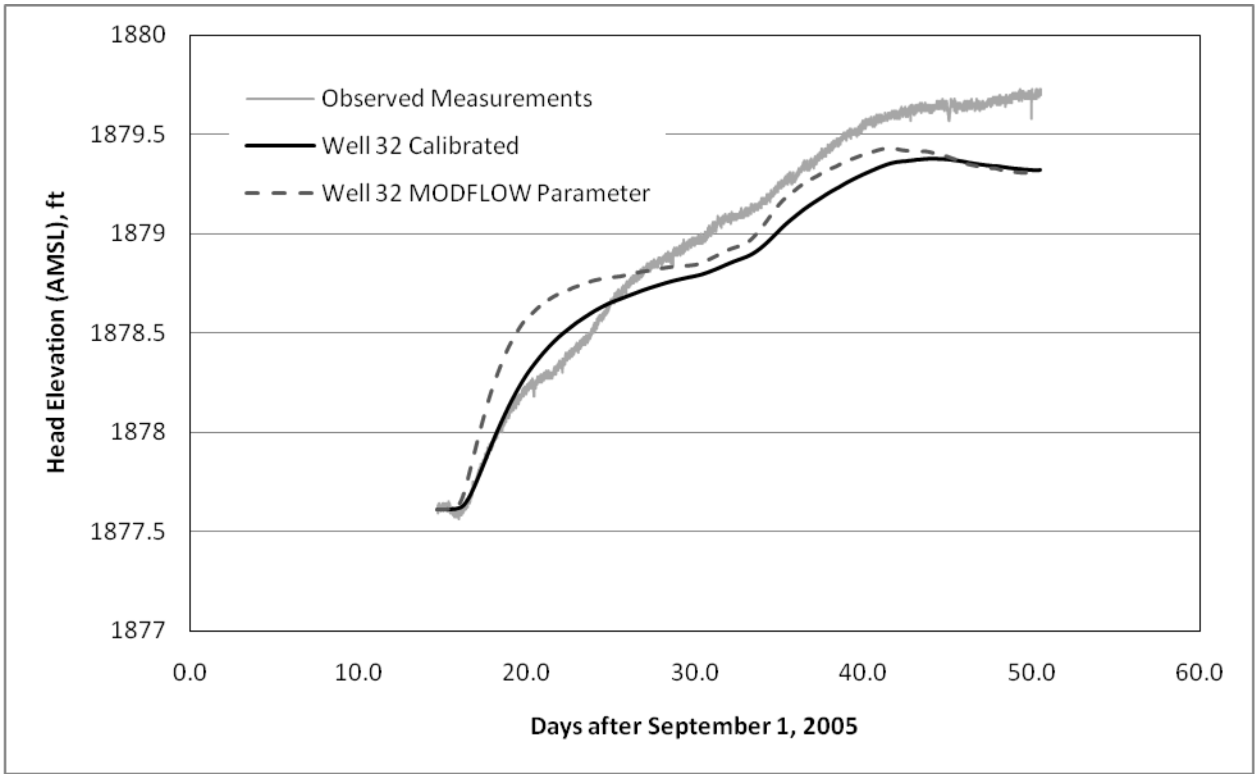


Figure 41. STWT1 results for Well 32 using calibrated and MODFLOW K' and S_y values.

Table 12 shows that the descriptive statistics for both the calibrated and MODFLOW parameters are very close. This is likely due to the over- and underestimation of the observed values. A clear depiction of this is in Figure 39 (Well 18), where the $\bar{e}$ for the calibrated values is negative and the $\bar{e}$ for the MODFLOW values is positive. The curve associated with the MODFLOW K' and S_y values starts out overestimating the observation well data and ends underestimating it. The sign of the difference between the observed and calculated values influences the value of $\bar{e}$, so the bias cancels itself out, leading to low bias and relative error values. The MODFLOW parameters are clearly over predicting the observation data initially, this works to move the STWT1 results closer to the later observational data can realistically achieve because of the influence of upstream flow.

Table 12. Area 4 Descriptive statistics from STWT1 equation

		NSE	ē	R_b %	min, ft	K'	K	S_y	a
Calibrated	Well 31	0.880	-0.151	-10.5%	1860.760	6	9500	0.3	1900
	Well 18	0.955	-0.076	-5.5%	1876.570				
	Well 39	0.941	-0.063	-4.7%	1851.240				
	Well 32	0.893	-0.147	-10.3%	1877.560				
MODFLOW	Well 31	0.894	-0.101	-7.1%	1860.760	10	9500	0.19	950
	Well 18	0.937	0.003	0.2%	1876.570				
	Well 39	0.895	0.028	2.1%	1851.240				
	Well 32	0.877	-0.045	-3.1%	1877.560				

ē is in the unit of the criteria variable (head = ft, seepage = cfs)

Barlow et al. (2000) used the Blackstone River in central Massachusetts, which penetrates 10-20% of the saturated thickness of the aquifer, has stratified layers of sand, gravel, and traces of silt which total about 20-50 ft in thickness, and vertical K of 1.4 ft/day with the STWT1 solution. They found unique values of K and S_s through calibration and independently determined estimates of transmissivity and saturated thickness at the site. The fits are visually similar to those shown in Wells 34 and 18 (Figure 37 and Figure 39, respectively). No goodness of fit statistics were reported. The STWT1 model was able to reproduce multiple fluctuations in the aquifer from stage changes in the Blackstone River very near the measured groundwater heads. Another study was done on the Cedar River in Iowa, with site characteristics of 20% partial penetration of the aquifer and composted of gravel and sand and bounded by impermeable bedrock. The best-fit, calculated values closely match the measured groundwater head for a passing flood wave. The main difference between the SVRP aquifer and the rivers given in the Barlow et al. (2000) study is the hydraulic connection of the aquifer and the river. In both cases explored by Barlow et al. (2000) the two systems were connected and produced results closely matching the measured values for the entire time series. The STWT1 model was unable to reproduce the SVRP aquifer conditions as time increased and 3D flow the near river became apparent. Thus the assumption that the initial water level in the stream and the aquifer must be equal could be relaxed if 3D flow was not a concern, since head data was reproduced accurately for the first 5-10 days.

4.3 Zlotnik and Huang (1999) (A2) Scenario Results

The assumptions used in the A2 equation are outlined in Chapter 3, but one is highlighted here to provide a rationale for the approach taken to report the results. The A2 solution assumed a semi-infinite aquifer to reduce the complexity of the equation, which is valid for many applications. In the SVRP aquifer the assumption of semi-infinite aquifer results in K' values that are up to an order of magnitude different from the other solutions and poor correlations to the observation wells. Therefore, only one graph for each area will be presented and K' will not be restricted to the same value for each well in that area. NSE values for the MODFLOW K' and S_y values will be reported as will the NSE values for the calibrated K' and S_y values. To provide meaningful comparisons, the STWT1 equation—which can be used for either finite or semi-infinite aquifers—will be run using the same K' value found from calibrated A2 solution. This will determine the effect of the semi-infinite approach used to model the interaction between the river and aquifer in the A2 solution. Additional graphs for each area are given in Appendix A.

4.3.1 Area 1 A2 Scenario Results

Figure 42 shows that the A2 solution was able to reproduce the first five days of the Well 15 data with the higher K' value. The difference between the curve from the MODFLOW parameter values and from the calibrated values is significant. The influence of upstream flow is still evident from the growing difference from start to finish of the curve. The lack of a flow boundary at some distance from the river causes the water moving in the aquifer to continue indefinitely. This does not allow a "bathtub" effect to take place, where the water is able to increase in depth as it reaches the physical boundary of the aquifer. Thus the K' of the streambed has to increase so that more flow can leave the river and enter the aquifer to try to replicate the observation data. The first five days match as the water is traveling toward the boundary but after five days it appears that the water reaches the edge of the aquifer and begins to pile up before the upstream water influences the solution. The A2 solution is not able to capture either event and under predicts the result.

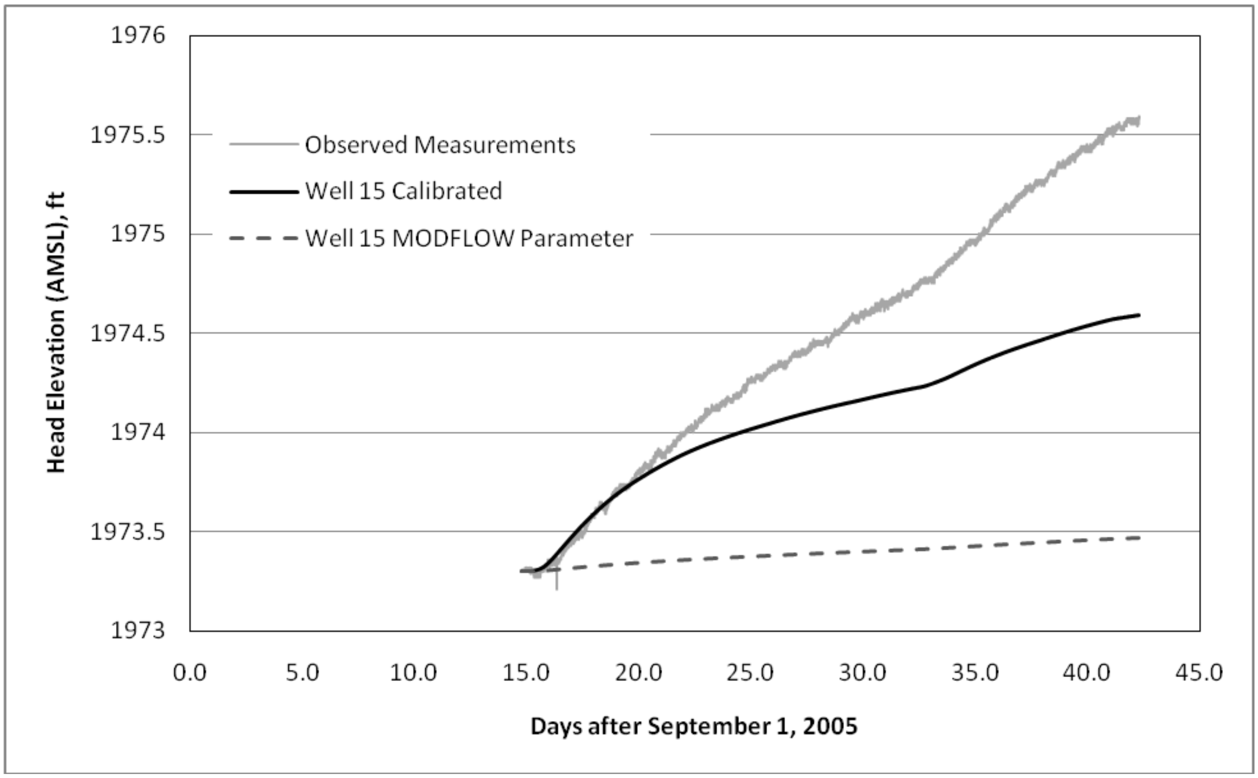


Figure 42. A2 results for Well 15 using calibrated and MODFLOW K' and S_y values.

The poor resemblances to the data are quantified in Table 13 with NSE values well below those achieved for the STWT1 solution. One observation from Table 13 is provided by the MODFLOW NSE values. Since the MODFLOW parameter curve is nearly a straight line and horizontal to the X-axis, a lower limit for the NSE value can be given provided that a curve that was negatively sloped is disregarded. Negative slopes would mean that the flood wave caused the water table to decline, which is not possible in the SVRP aquifer.

Table 13. Area 1 Descriptive statistics from A2 equation

		NSE	ē	R_b %	min, ft	K'	K	S_y
Calibrated	Well 16	0.507	-0.351	-26.0%	1971.950	7.1	22150	0.3
	Well 15	0.407	-0.395	-31.2%	1973.210			
	Well 13	0.268	-0.431	-37.3%	1976.590			
MODFLOW	Well 16	-2.823	-1.201	-89.0%	1971.950	0.25	22150	0.19
	Well 15	-2.570	-1.086	-85.8%	1973.210			
	Well 13	-2.286	-0.968	-83.6%	1976.590			

ē is in the unit of the criteria variable (head = ft, seepage = cfs)

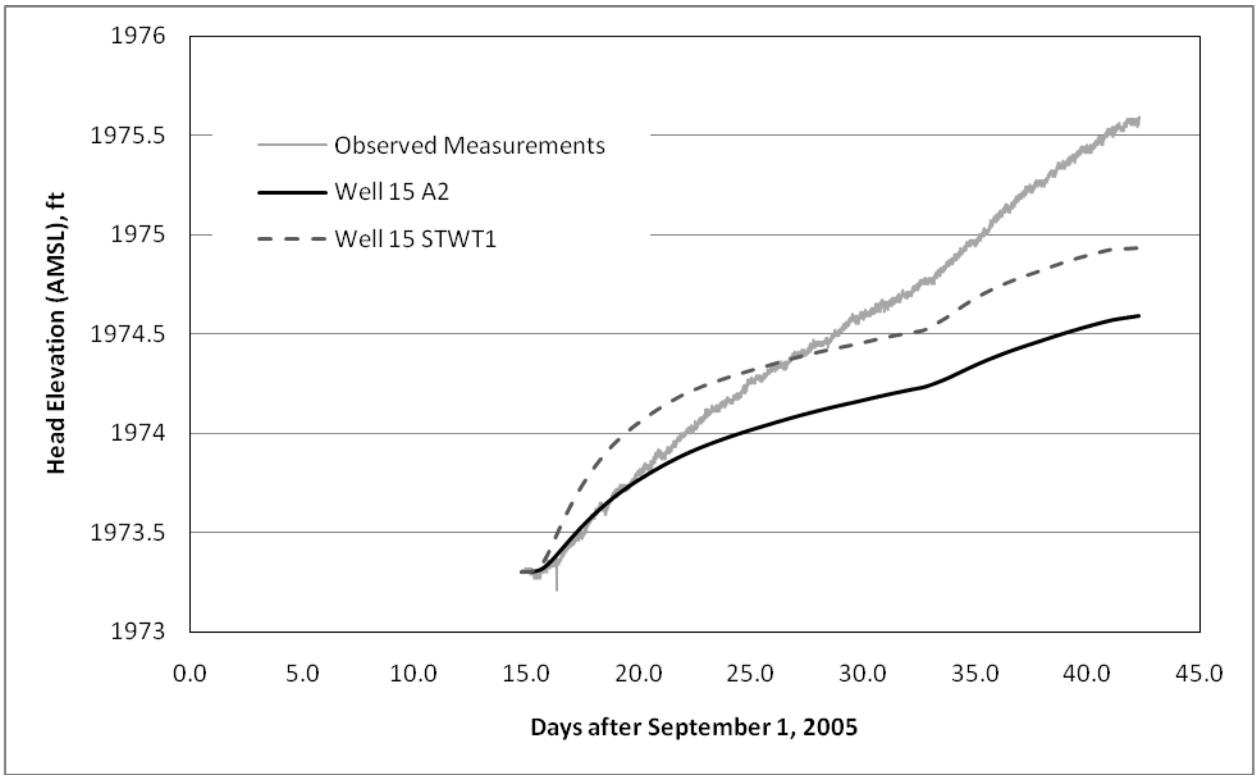


Figure 43. Comparison of semi-infinite solution from A2 and STWT1 solutions for Well 15

Figure 43 shows the comparison between the A2 solution and the STWT1 solution. Using the same aquifer characteristic for all associated parameters, it is clear that the STWT1 method predicts a higher water table. The assumption of a fully penetrating steam (STWT1) and partially penetrating stream (A2) (see Figure 19 and Figure 20) produce different results in a semi-infinite aquifer with characteristics of the SVRP aquifer. This implies that for the STWT1 solution to match the A2 solution a greater value of XAA (Table 4) is required.

$$XAA = \frac{Km}{K'} \qquad \text{Equation 17}$$

XAA is used to determine the flow across the streambed when multiplied by the difference between the water level of the stream after the instantaneous step change and water table level. How this affects a finite system is unknown.

4.3.2 Area 2 A2 Scenario Results

In Area 2 the value of K' was allowed to change for each well. This demonstrated the influence of the semi-infinite condition on aquifer response as the distance from a well to the Spokane River increased. The head curve for Well 48 has a very similar shape to Well 15 with the sharp change in slope corresponding to the change in the hydrograph for the Barker Rd Gage (Figure 29). The assumption of the system starting from rest (i.e. a water table and river water surface of equal height) clearly plays a role. With the higher K' values more water is allowed to move into the aquifer, which causes the change in slope after the initial increase in water surface elevation. The MODFLOW curve is again mostly a lower limit to the NSE value for Area 2.

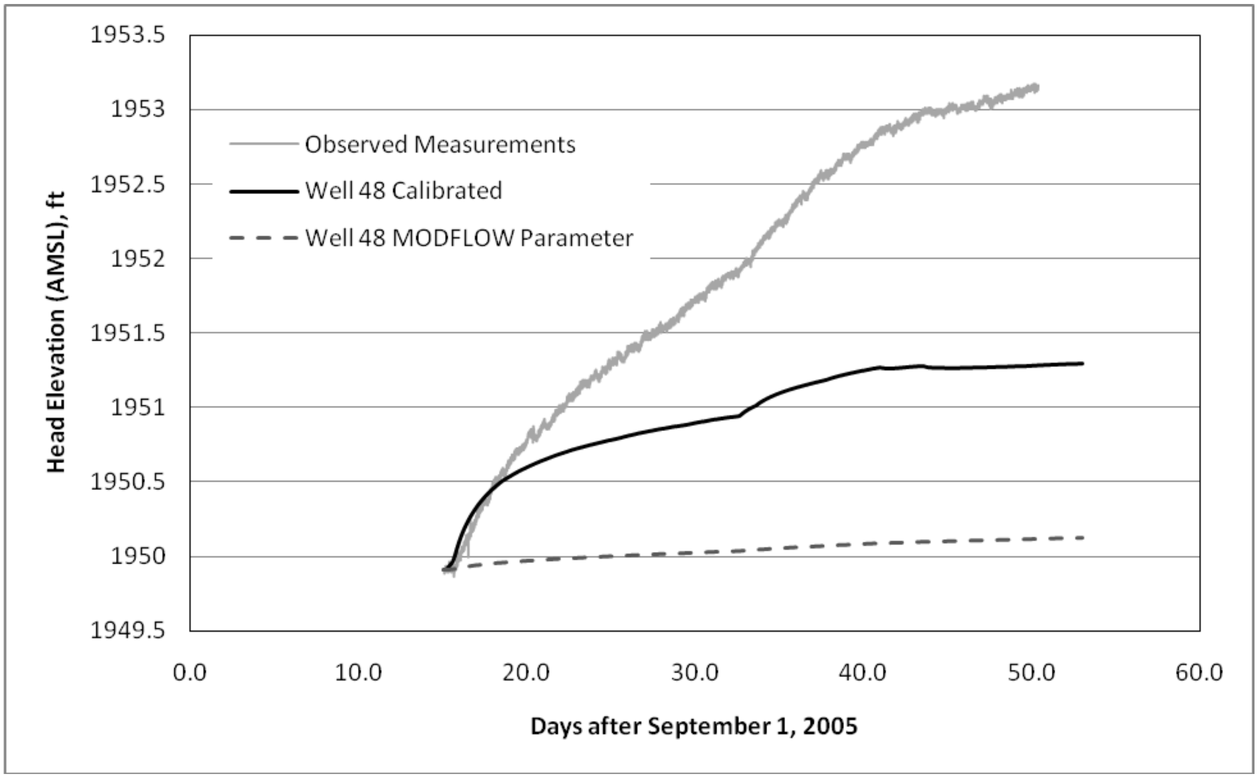


Figure 44. Well 48, A2 results for calibrated and MODFLOW K' and S_y values.

Table 14 shows an interesting trend in K’ values; wells closer to the Spokane River have lower K’ values and wells further from the Spokane River have higher K’ values. Since all five wells correspond to roughly the same section of the Spokane River, the increase is an artifact of the solution, not the actual condition. The semi-infinite assumption requires more water to infiltrate that is needed in reality to provide the same response as seen in the observation well.

Table 14. Area 2 Descriptive statistics from A2 equation.

		NSE	ē*	R_b %	min, ft	K'	K	S_y
Calibrated	Well 6	-1.150	-1.248	-61.0%	1942.9	3	22150	0.3
	Well 28	-0.744	-1.065	-51.6%	1949.7	4	22150	0.3
	Well 19	-0.616	-0.992	-47.2%	1948.9	5	22150	0.3
	Well 48	-0.501	-0.975	-47.9%	1949.9	6	22150	0.3
	Well 11	-0.257	-0.819	-44.3%	1952.4	13	22150	0.3
MODFLOW	Well 6	-3.618	-1.884	-92.1%	1942.9	0.22	22150	0.19
	Well 28	-0.744	-1.065	-51.6%	1949.7	0.22	22150	0.19
	Well 19	-4.267	-1.932	-91.9%	1948.9	0.22	22150	0.19

Well 48	-3.801	-1.876	-92.2%	1949.9	0.22	22150	0.19
Well 11	-3.876	-1.721	-93.1%	1952.4	0.22	22150	0.19

* ē is in the unit of the criteria variable (head = ft, seepage = cfs)

The difference between STWT1 and A2 solutions for the semi-infinite aquifer is very similar to that of Area 1. Figure 45 shows the differences between the STWT1 and A2 solutions for Area 2.

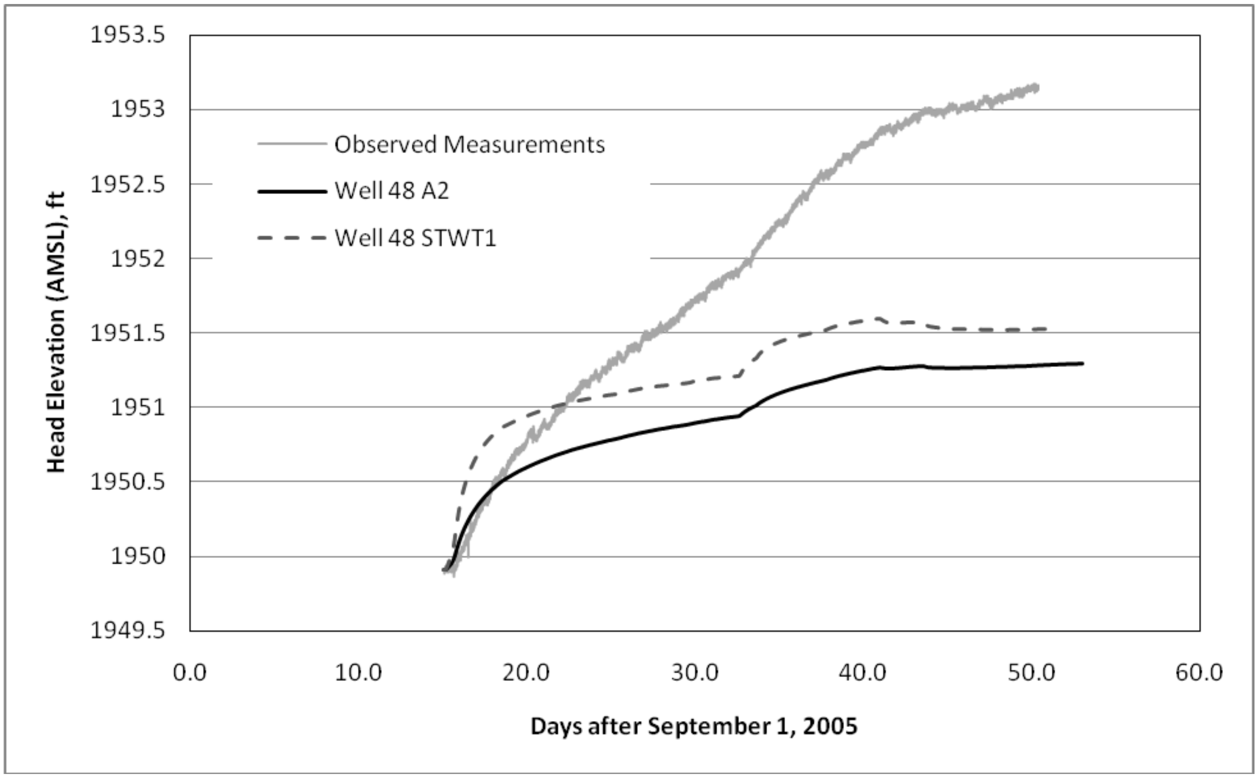


Figure 45. Comparison of semi-infinite solution from A2 and STWT1 solutions for Well 48.

4.3.3 Area 3 A2 Scenario Results

For Area 3 the same approach as in the STWT1 solution was used to overcome the difficulties in the stream stage measurements. However, for Well 49 the elevation of the stream and the water table was assumed to be equal. This was assumed because Well 49 is upstream of the Sullivan Road Bridge Gage and the Spokane River has losing conditions when moving up stream of Sullivan Road. As shown in Figure 46 the MODFLOW parameter is well above the observation well for the observed 6-Minute data. The calibrated curve shown is for the regression equation for streamflow (Figure 29) and has a lower initial change in stage from the passing flood wave than the 6-Minute data. A comparison (not shown) was also done with the 6-Minute data for Well 49, but since the calibrated K' value was so close to MODFLOW K' value (see Table 15), the calibrated curve was slightly lower but very close to the MODFLOW curve. The calibrated curve fits wells with the observation data but is biased towards overestimation.

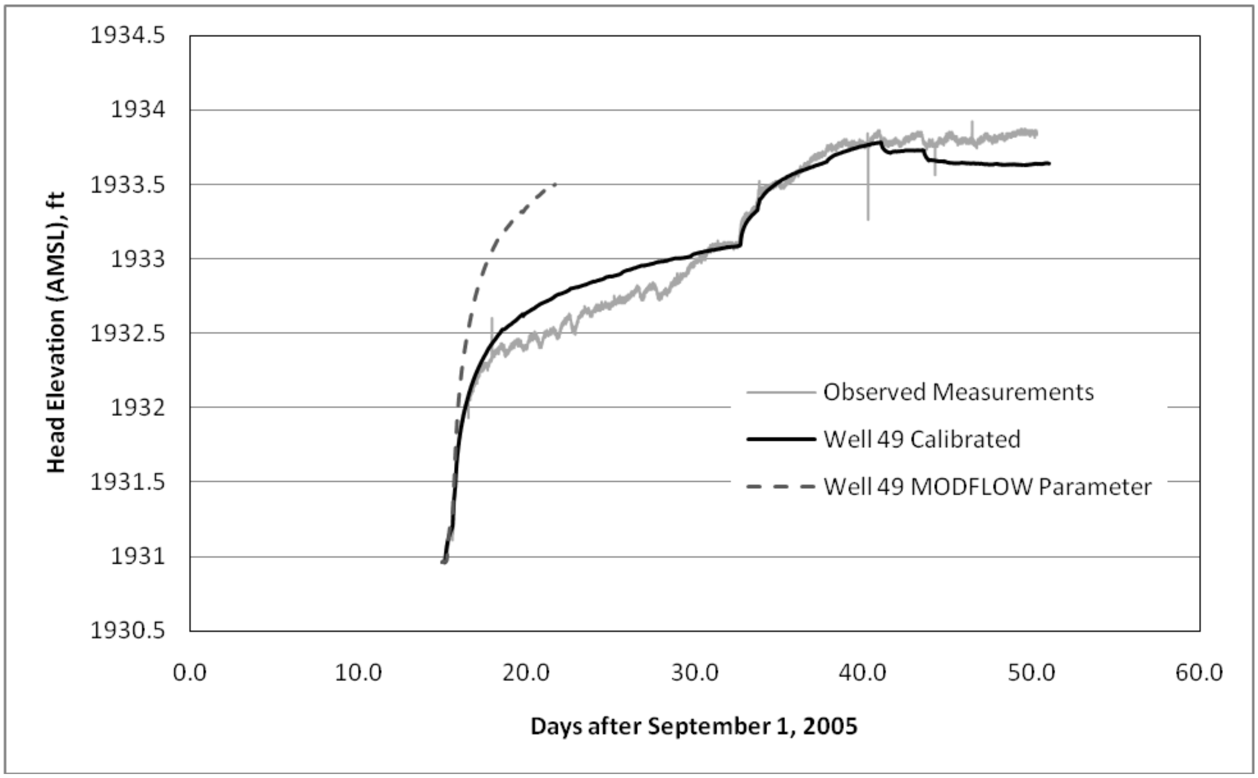


Figure 46. A2 results for Well 49 using calibrated and MODFLOW K' and S_y values.

Table 15 shows the descriptive statistics for Well 34 from the original 6-Minute Data, the MODFLOW parameters, and the calibrated curve parameters. One observation from Table 15 is the close match between the calibrated and MODFLOW parameters for Well 49. This is different than previous areas where MODFLOW values were significantly higher than the calibrated values. However, conclusions from Area 3 are difficult to determine given the assumptions made in the Spokane River to reproduce the head in the aquifer. Also the S_y that best fits the data was 0.19 not the 0.3 from the STWT1 solution.

Table 15. Area 3 Descriptive statistics from A2 equation.

		NSE	ē*	R_b %	min, ft	K'	K	S_y
Calibrated	Well 34	-0.586	0.595	35.5%	1929.97	14	7644	0.19
	Well 49	0.957	0.023	1.1%	1930.95	8	7644	0.19
MODFLOW	Well 34	0.468	0.272	16.2%	1929.97	9.4	7644	0.19
	Well 49	-1.939	0.662	54.1%	1930.95			

* ē is in the unit of the criteria variable (head = ft, seepage = cfs)

Figure 47 shows the regression stream stage used for comparison between the A2 and STWT1 solutions. The difference between the two lines is larger than seen in previous areas. The A2 solution predicts a more continuous line than the STWT1 solution which shows a steep initial slope and a nearly constant head elevation before the second pulse of flow. This indicates that the STWT1 solution simulates filling the aquifer to the level of the Spokane River more quickly than the A2 solution.

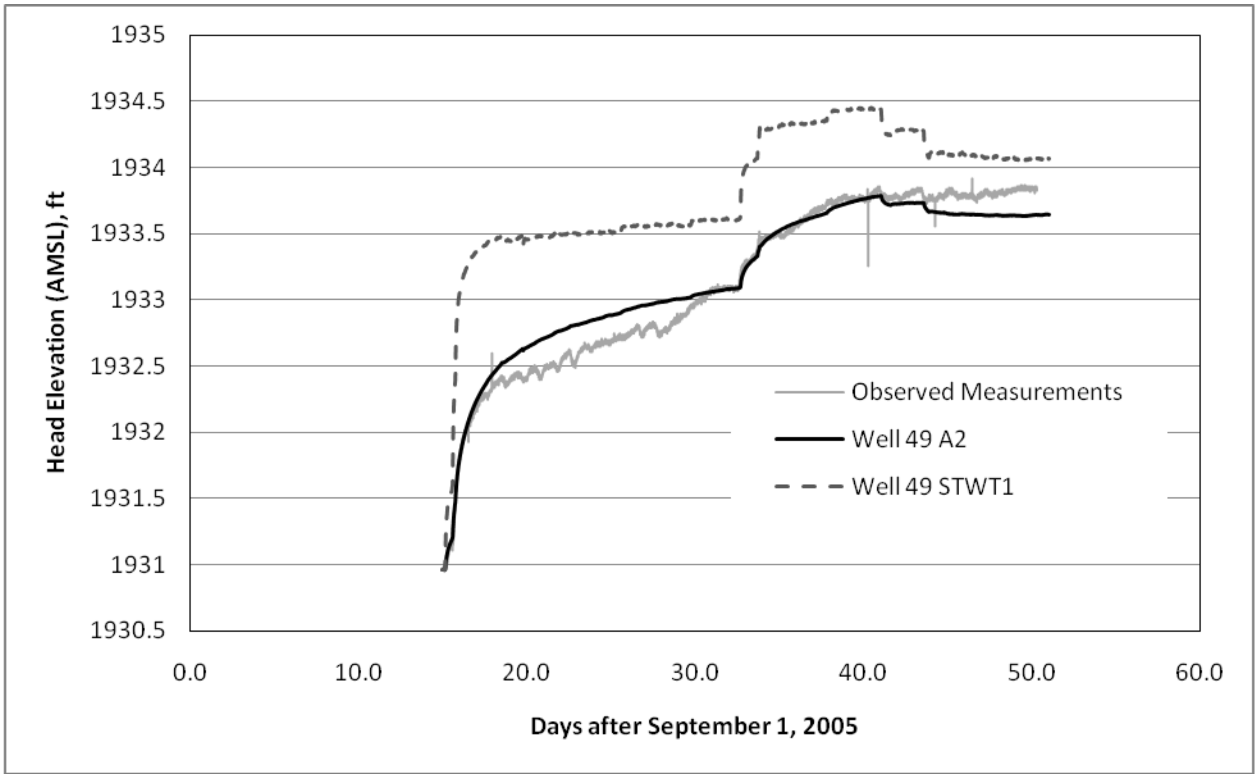


Figure 47. Comparison of semi-infinite solution from A2 and STWT1 solutions for Well 49.

4.3.4 Area 4 A2 Scenario Results

In Area 4 the MODFLOW values for K' and S_y produced curves that under predicted the change in the aquifer. Figure 48 shows the difference between the observed values and the values from the MODFLOW parameters is less than the differences for the same values in Areas 1 and 2. Area 4 was found to be far less sensitive to changes in K' and the curves for a change in K' of 9 ft/day are 0.1—0.3 ft different. In Area 1 a 6.85 ft/day change caused the aquifer water table depth to change up to 1 foot. The MODFLOW K' value for Area 4 is 10 ft/day, which is two orders of magnitude greater than the MODFLOW K' for Area 1 (0.25 ft/day). This may indicate the A2 solution is less sensitive to K' when K' is large, and more sensitive to K' when the value of K' is low. The ratio between aquifer K and K' may also play a role, in the sensitivity analysis this will be looked at further, but at this point it is uncertain if there is any real sensitivity to K' values.

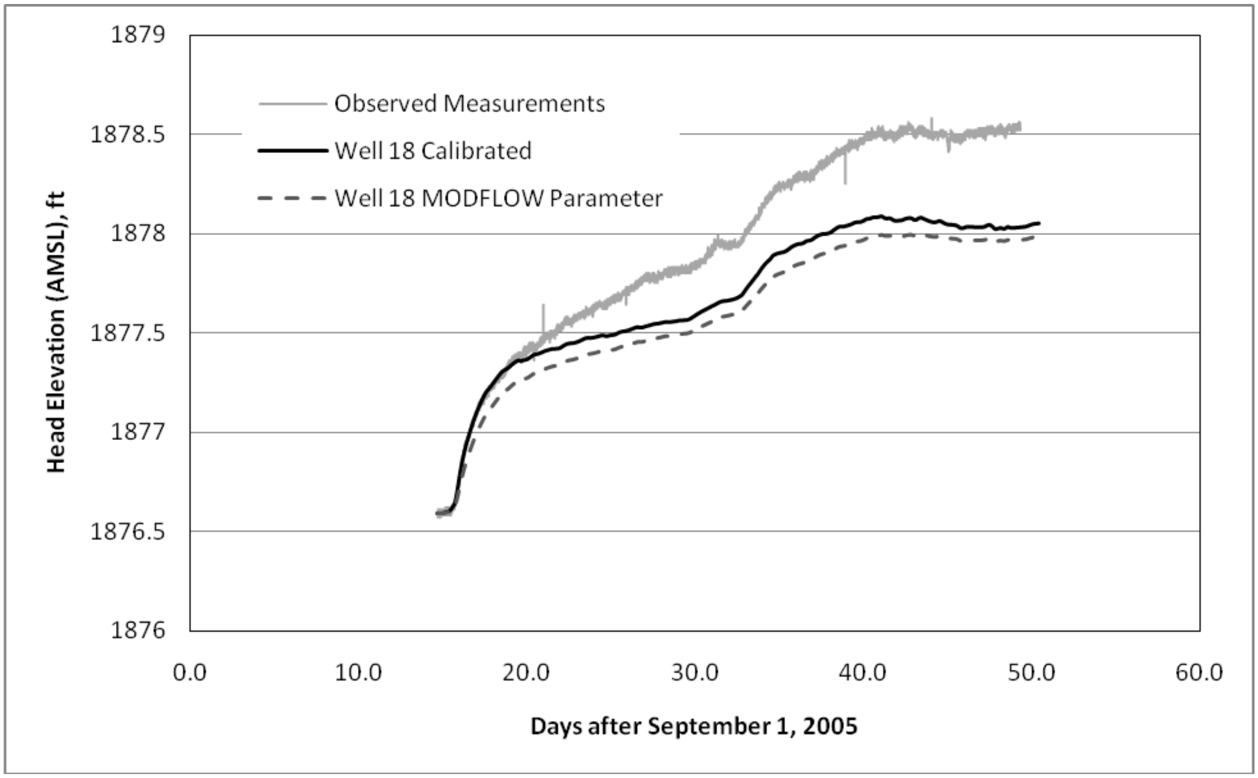


Figure 48. A2 results for Well 18 using calibrated and MODFLOW K' and S_y values.

The NSE values are low for Area 4 and could be improved if the solution was allowed to over-predict values for the first 5 days. However, the assumptions in the A2 solution are best suited for the initial few days. For that reason a second NSE value for the first five days was used to determine the K' and S_y values that were able to best simulate the early time period. This NSE is not shown but was used to determine the point when solutions began to over-predict the early time period, for the case when the MODFLOW K' and S_y values under-predicted the solutions and vice-versa. This approach aimed at capitalizing on the strengths of the A2 solution instead of where it was not valid for the SVRP aquifer. Another interesting aspect of the data presented in Table 16 is the S_y values. Both Wells 31 and 39 where able to converge to a good fit to the data with a S_y of 0.3, while Wells 18 and 32 were not. That is not to say that the highest NSE value for Well 18 and Well 32 could not have been produced by an S_y of 0.3, but only that such a fit could not be completed for a K' value of 40 or less. Since a K' greater than 40 was not reasonable the MODFLOW S_y was used and the best fit K' found.

Table 16. Area 4 Descriptive statistics from A2 equation.

		NSE	ē*	R_b %	min, ft	K'	K	S_y
Calibrated	Well 31	0.675	-0.260	-18.2%	1860.8	29	9503	0.3
	Well 18	0.643	-0.268	-19.3%	1876.6	19	9503	0.19
	Well 39	0.492	-0.355	-26.4%	1851.2	27	9503	0.3
	Well 32	0.268	-0.469	-33.1%	1877.6	23	9503	0.19
MODFLOW	Well 31	0.496	-0.343	-24.1%	1860.8	10	9503	0.19
	Well 18	0.473	-0.345	-24.8%	1876.6			
	Well 39	0.445	-0.374	-27.8%	1851.2			
	Well 32	0.042	-0.553	-39.0%	1877.6			

* ē is in the unit of the criteria variable (head = ft, seepage = cfs)

The difference between the A2 solution and the STWT1 is slight. Comparing the difference in Figure 49 with the difference in Figure 48 reveals very similar values. The STWT1 solution produces a greater aquifer response than the A2 solution but generally matches the slope of the A2 solution.

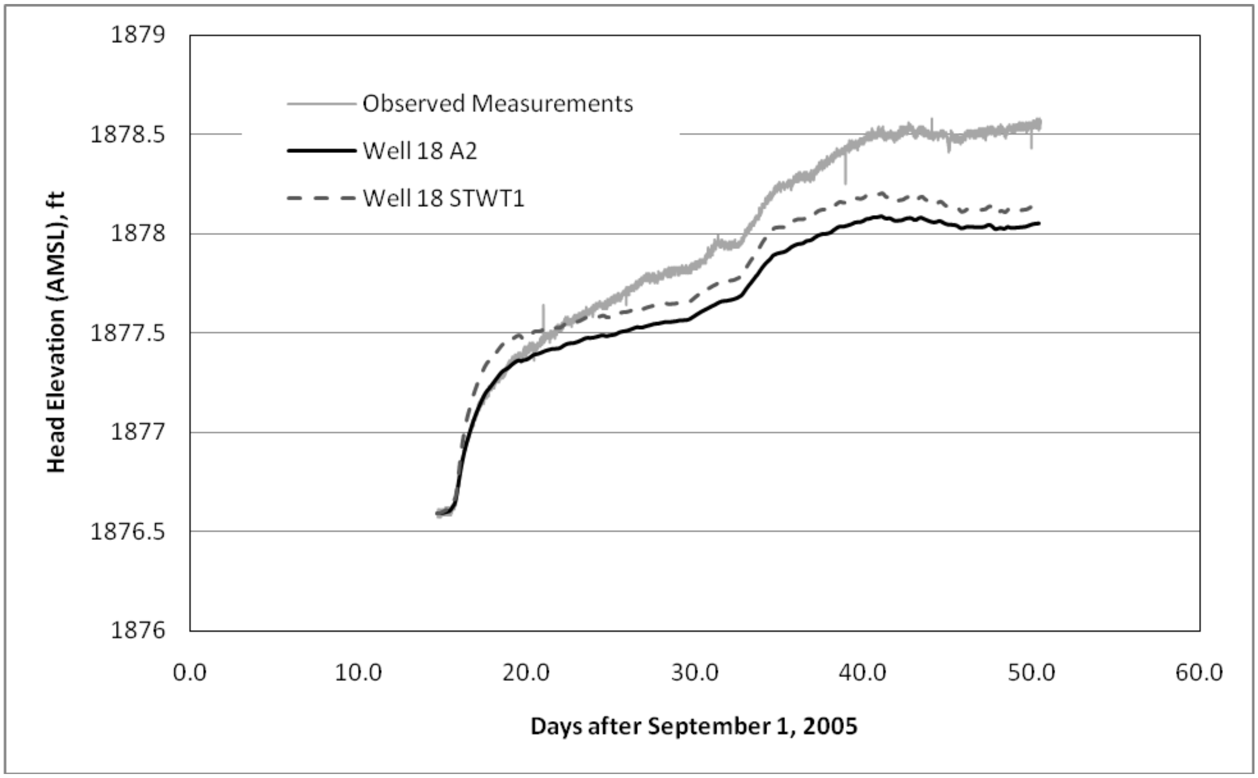


Figure 49. Comparison of semi-infinite solution from A2 and STWT1 solutions for Well 18.

Bolster et al., (2001) used the A2 solution to estimate the S_y for the Biscayne Aquifer in the Everglades National Park (ENP), southeast Florida. Bolster et al. (2001) give the following site characteristics: K exceeds 3280 ft/day, about 45 foot aquifer thickness, and two layers of limestone. Drawdown in a canal (about a foot change in water surface elevation in the canal) was used to determine S_y. The resulting values for S_y ranged from 0.05-0.57 and averaged 0.21. The fits shown for eleven piezometers were good and reproduced the magnitude and timing of the drawdown with the maximum difference between the observed and computed groundwater drawdown of less than 1.6 inches (Bolster et al., 2001). The SVRP aquifer also has high K values but in the Biscayne Aquifer the water table is usually 1.6 feet from the ground surface, which leads to connected aquifer and stream. Thus Bolster et al. (2001) achieve better fits than possible in the SVRP aquifer.

4.3.5 Comparison of A2 and STWT1 Solutions

The difference between the A2 solution and STWT1 solution is small for all areas except for Area 3. No pattern for the difference between the solutions is immediately apparent with values of K, K' or S_y. The MODFLOW K' values were used for the comparison because the calibrated K' values were different for the A2 and STWT1 solutions. The STWT1 solution was greater than the A2 solution for each area, and the average difference for STWT1 minus the A2 solution can be found in Table 17. This was the only observable pattern and is consistent. The minimum difference was not reported because each solution started as the same value.

Table 17. Comparison of A2 and STWT1 solutions differences

	Head Difference		MODFLOW		
	Average	Max	K	K'	S_y
Area1	0.29	0.36	22150	0.25	0.19
Area2	0.30	0.36	19090	0.22	0.19
Area3	0.65	1.26	7466	9.4	0.19
Area4	0.10	0.14	9503	10	0.19

The effect of the semi-finite condition for the A2 solution cannot be directly evaluated, but the STWT1 allows for this comparison. Using Well 15 in Area 1 a finite and semi-finite condition for the calibrated K' and S_y values from the A2 solution was completed. Figure 50 shows that for the first five days, the results are the same but after five days the semi-infinite solution moves away from the finite solution. This confirms the use of the early time series as the most valid period for calibration of the semi-infinite case. It also clearly shows the difference between the method of transferring seepage across the riverbed in the A2 and STWT1 solutions. The A2 solution does not follow the same curve as the STWT1 semi-infinite case early in time, as the finite case of the STWT1 solution does. This difference in the early time series for the A2 and STWT1 semi-infinite is the best representation of the dissimilarity between the two approaches taken. Thus the difference in calibrated K' values for the A2 and STWT1 solutions are largely due to the method of seepage.

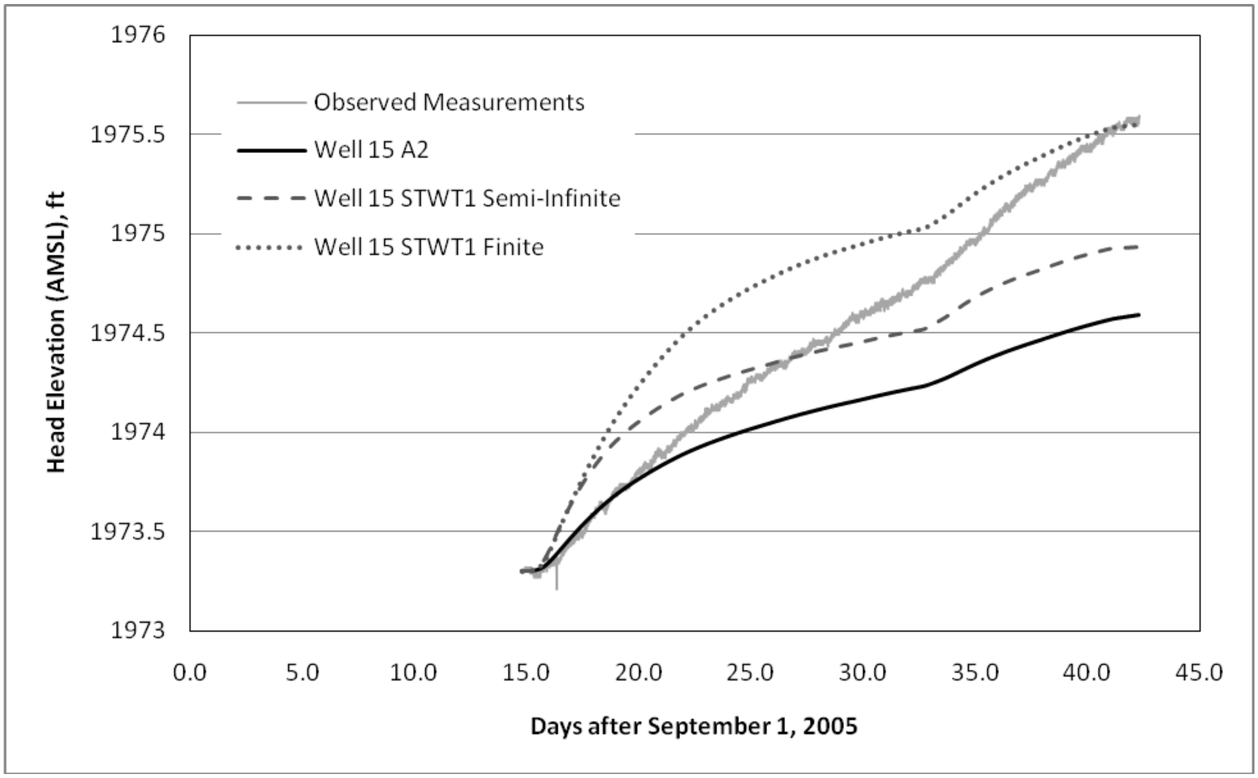


Figure 50. Comparison of Finite and Semi-Infinite assumptions on SVRP aquifer for Well 15.

4.4 Hantush (2005) (A3) Scenario Results

The Hantush solution models the interaction between groundwater and surface water by using the Muskingum Routing Method and adding or removing water to or from the aquifer. Only Areas 1 and 2 had flow data to determine the loss or gain from the Spokane River. This limits the equation to strictly losing reaches, which as the STWT1 and A2 solutions violates the initial condition of the system at rest. However, results for the first part of the time series are informative and can be considered to reproduce conditions similar to the assumptions used in the A3 solution.

4.4.1 Area 1 A3 Scenario Results

The A3 solution for Area 1 is determined from both the outflow at Harvard Road Gage and the seepage into the SVRP between the Post Falls Gage and the Harvard Road Gage. In Figure 51, outflow is the flow at Harvard Road Gage relative to the date and time 9/14/05, 22:15:00 (the starting date and time). By subtracting the flow at the starting date and time from each of the measurements, the initial outflow is zero. This is also done in the setup of solution A3 to define all variables relative to the initial value or to produce an initial condition of zero. Seepage (Figure 52) is also defined relative to the initial value. The MODFLOW K' and S_y values give an outflow at Harvard Road greater than the measured value and the calibrated value give an outflow less than the measured value. The fact that the calibrated curve is roughly the same distance from the measured values as the MODFLOW curve is due to the seepage for the two scenarios. Figure 52 shows that the seepage for the MODFLOW parameters is far less than the measured value. The calibrated parameters are able to reproduce the measured seepage values reasonably well. In order to get the seepage curve to match, the outflow parameters have to become less than the measured values. This is most likely due to the same problematic assumption of the system starting from rest for the SVRP aquifer.

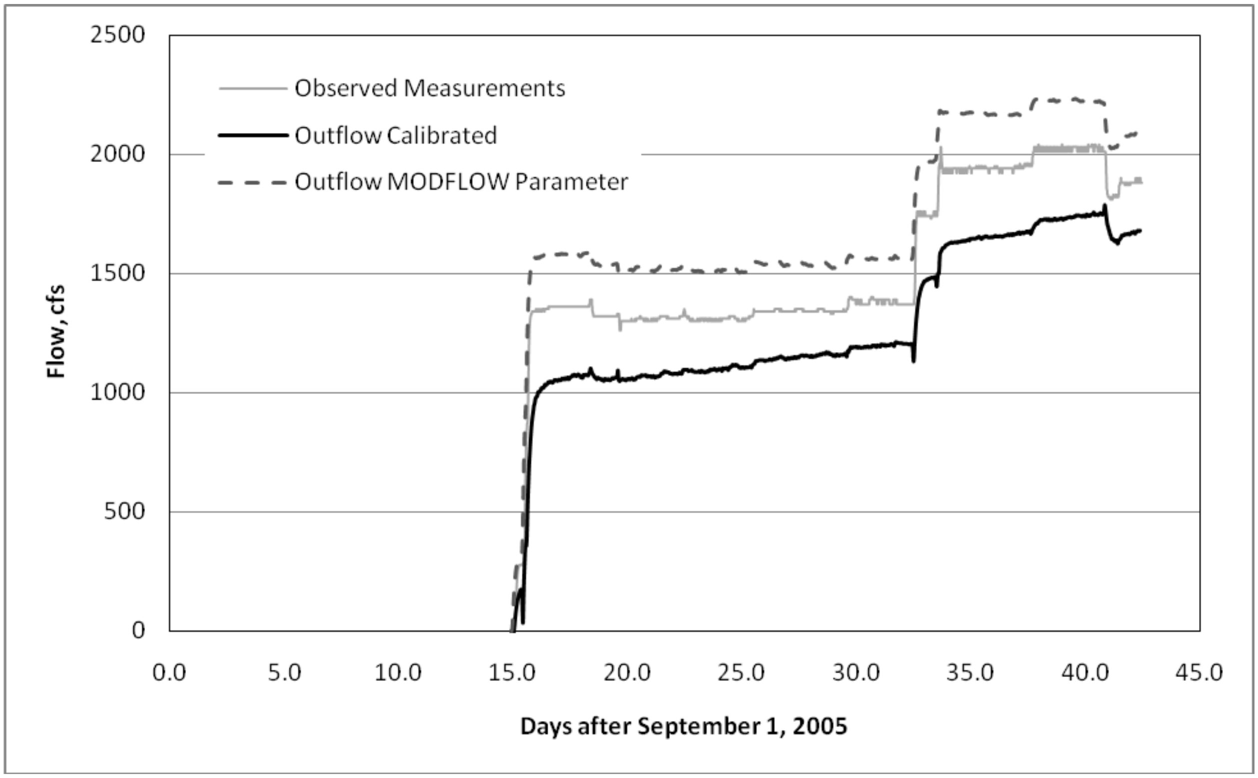


Figure 51. A3 results for Area 1 outflow using calibrated and MODFLOW K' and S_y values.

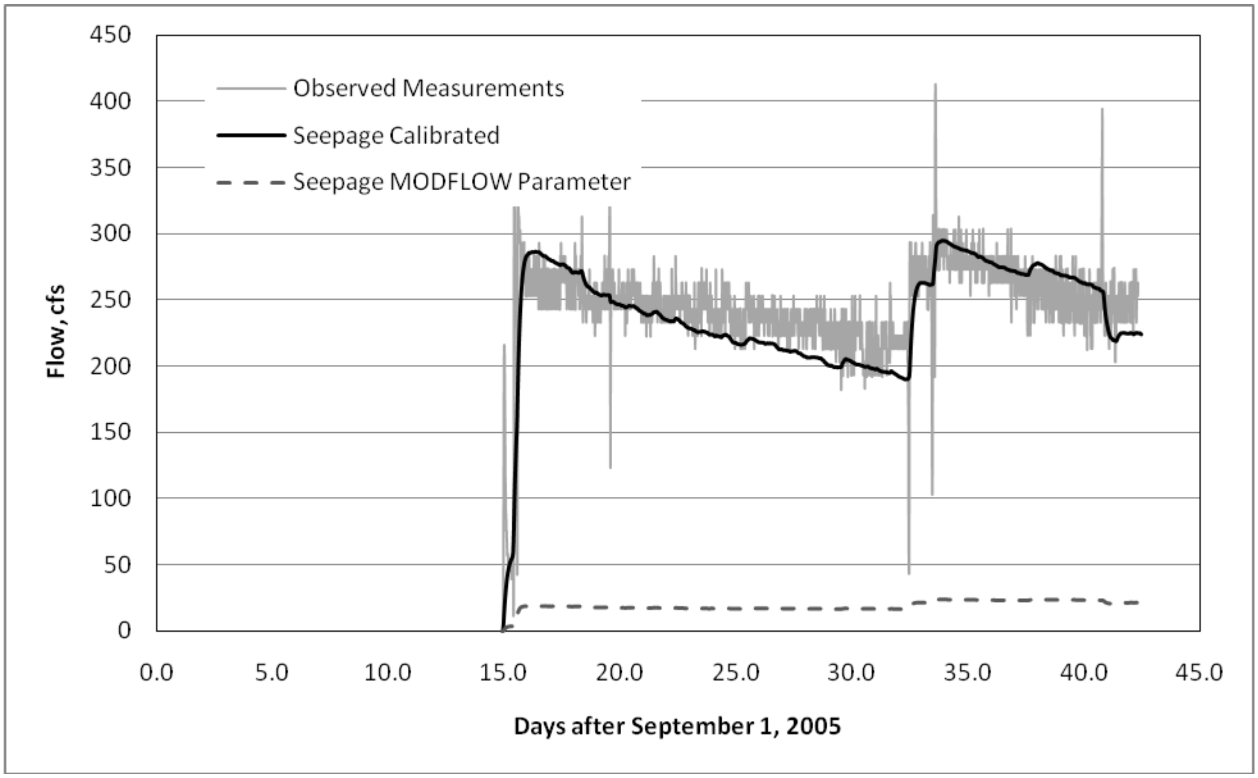


Figure 52. A3 results for Area 1 seepage using calibrated and MODFLOW K' and S_y values.

Table 18 shows two NSE values for both the calibrated and MODFLOW parameters. The first is the individual fit to the respective measured value (i.e. calibrated Outflow NSE = 0.489), the second is the average NSE value for both the outflow and the seepage. This average NSE value was used to determine the values of K', S_y, η, and ξ that best fit all the data. The bias (ē) for the outflow data is close to the same magnitude but opposite in sign for the calibrated and MODFLOW parameters. The value of η (Table 18) is representative of the time it takes for the time of travel of the flood wave through the reach (Chow et al., 1988). However, examining the hydrographs for Post Falls Gage and Harvard Road Gage the time of travel is about 2.5 hours. The difference between the observed η and the calibrated η indicates that to achieve the seepage values requires the flood wave to move more slowly through Area 1, allowing time for seepage to accumulate. This reduces the outflow and the result is the poor fit in Figure 51. The calibrated outflow curve can be made to be nearly identical to the measured values if η is set to 0.104 days (2.5 hours), K' set to 2, and ξ and S_y set to 0.3 (NSE = 0.975). Including seepage with the parameters just described brings the average NSE down to -8.67. The inability of the A3 solution to match both outflow and seepage is attributed to A3 solution assumptions.

Table 18. Area 1 descriptive statistics for A3 solution results

		NSE	ē*	R_b %	Avg NSE	K'	K	S_y	η (days)	ξ
Calibrated	Outflow	0.489	-243.649	-16.0%	0.398	2	22150	0.3	0.292	0.3
	Seepage	0.307	-7.045	-2.9%						
MODFLOW	Outflow	0.645	207.895	13.6%	-21.566	0.25	22150	0.19	0.104	0.14
	Seepage	-43.778	-227.341	-92.4%						

* ē is in the unit of the criteria variable (Outflow = cfs, Seepage = cfs)

In Figure 53 the seepage from A3 calibrated solution is compared to the STWT1 solution. The difference between the two solutions is seen after 17 days when the assumptions of both are not valid. However, the A3 solution overall does better at reproducing the observed measurements. Thus the method used to derive the solution plays a role in determining the ability of the solution to represent the interaction between a river and a groundwater system. It is noted that the calibrated solution for the STWT1 approach was determined to be K' = 1.8 ft/day and S_y = 0.3 for Area 1, which is close to the values reported by the A3 solution in Table 18.

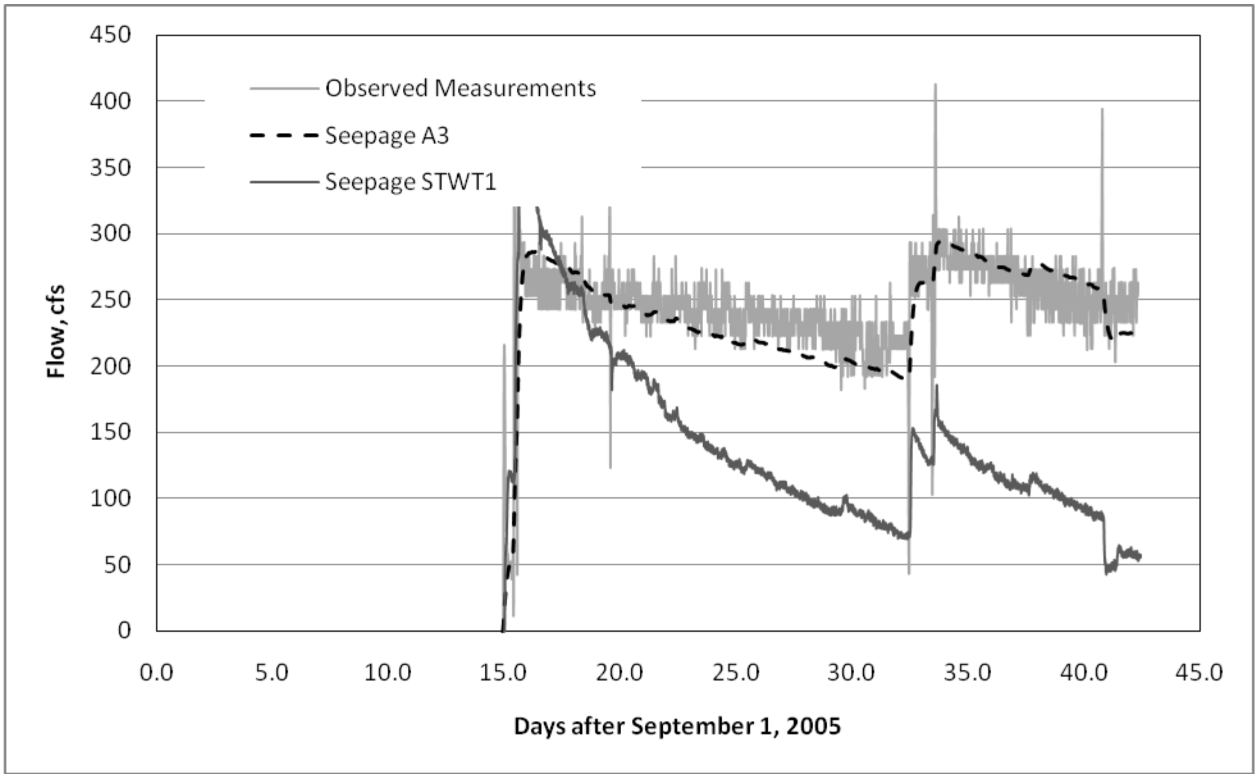


Figure 53. Comparison of seepage from A3 and STWT1 solutions for Area 1.

4.4.2 Area 2 A3 Scenario Results

Results for Area 2 are very similar to Area 1. Both are strictly losing reaches, and the same assumption causes difficulty in reproducing the observed measurements. Keeping that in mind the model for Area 2 is able to better capture the outflow (Figure 54) but the does not capture the seepage (Figure 55) and has a lower NSE value than Area 1 overall. Although the A3 solution does not predict the exact magnitude of the measured values, it is able to duplicate the changes in river surface elevation at the right time. The seepage curve from the A3 solution provides the magnitude of the initial seepage after the flood pulse pass by, but is not able to maintain the extended seepage of the losing reach. The model for Area 1 was able to maintain the seepage and it is slightly surprising that the model of Area 2 could not do the same.

However, the value of η is much less in Area 2 and if η was increased an extended seepage might be produced.

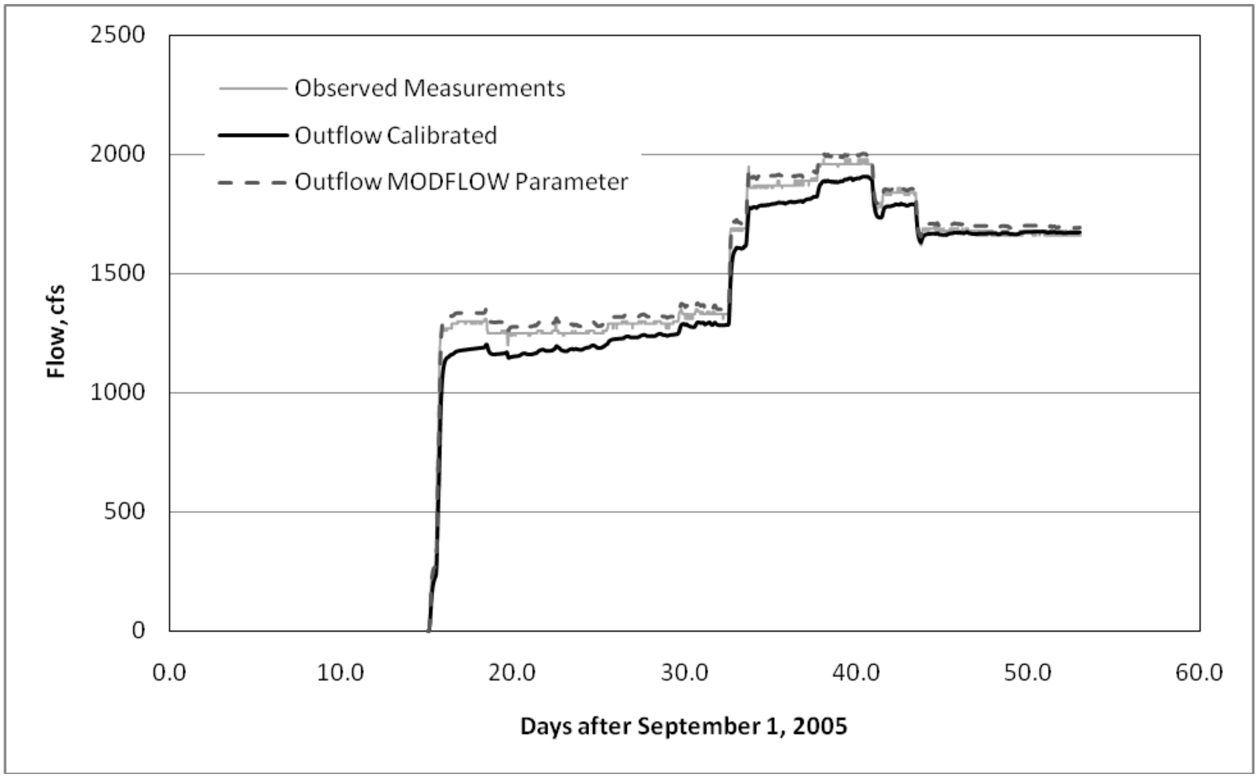


Figure 54. A3 results for Area 2 outflow using calibrated and MODFLOW K' and S_y values.

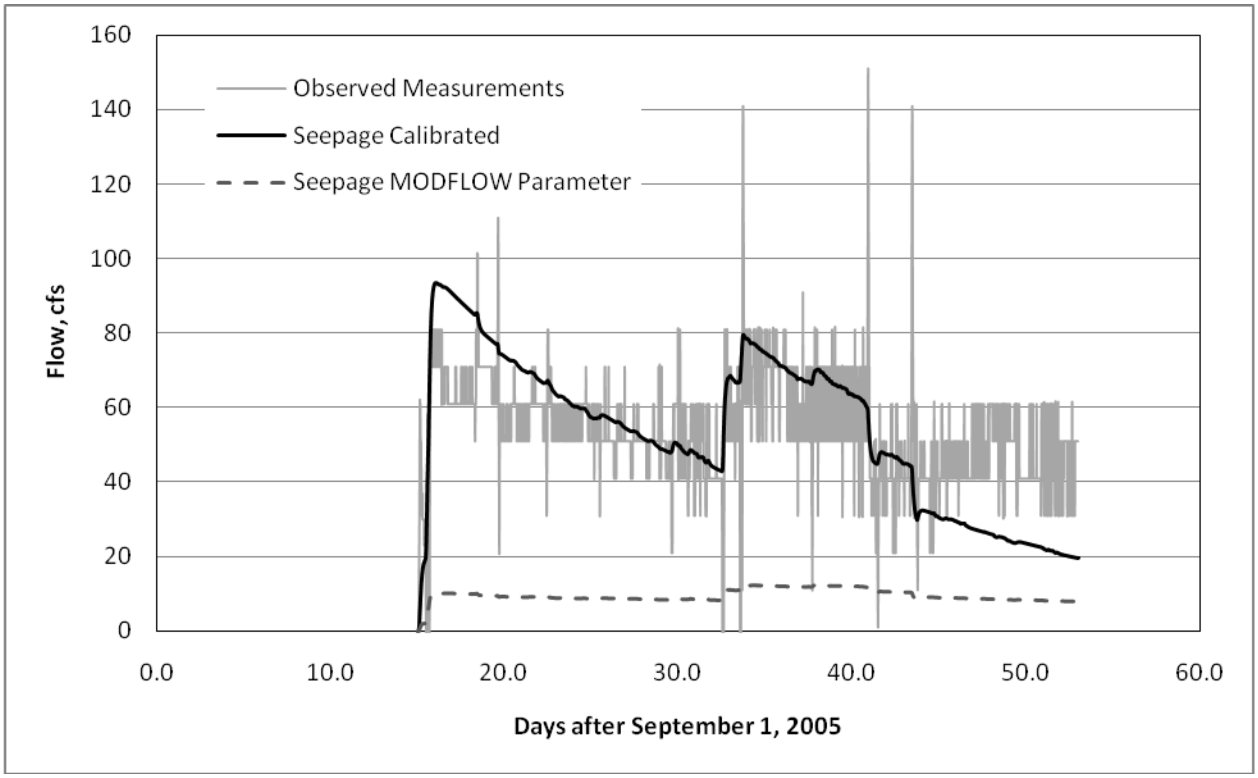


Figure 55. A3 results for Area 2 seepage using calibrated and MODFLOW K' and S_y values.

The marked difference in NSE values for outflow and seepage in Area 2 indicate a problem in the method used for this area. The mismatch can be attributed to the reasons discussed for Area 1. The value of ξ was reduced to achieve the calibrated values, which is contrary to Area 1. As ξ approaches 0 the Muskingum Method becomes similar to the level pool routing method. The artificial flood wave is unique as it does not follow one of the families of asymmetric sinusoidal curves defined by Cooper and Rorabaugh (1963). The artificial flood wave has a very steep front and a near constant value for a substantial time afterward. Given it has a dynamic nature as it passes followed by level pool characteristics could explain the decrease in ξ for Area 2.

Table 19. Area 2 descriptive statistics for A3 solution results

	NSE	$\bar{e}$*	R_b %	Avg NSE	K'	K	S_y	η (days)	ξ

Calibrated	Outflow	0.950	-56.201	-3.7%	0.267	1.5	19090	0.3	0.125	0.1
	Seepage	-0.417	0.433	0.8%						
MODFLOW	Outflow	0.986	34.172	2.2%	-3.312	0.22	19090	0.19	0.076	0.3
	Seepage	-7.611	-43.638	-81.8%						

* ē is in the unit of the criteria variable (Outflow = cfs, Seepage = cfs)

Figure 56 shows that for Area 2 the STWT1 solution is better able to maintain seepage late in the time series, but shows large declines in seepage after the initial pulse. From Area 2 the method used still influences the ability of the solution to simulate the observed conditions but again leads to similar values of K' and S_y. While the comparison in Figure 56 for the same values shows differences, the overall ability to determine river and aquifer parameters seems to be similar.

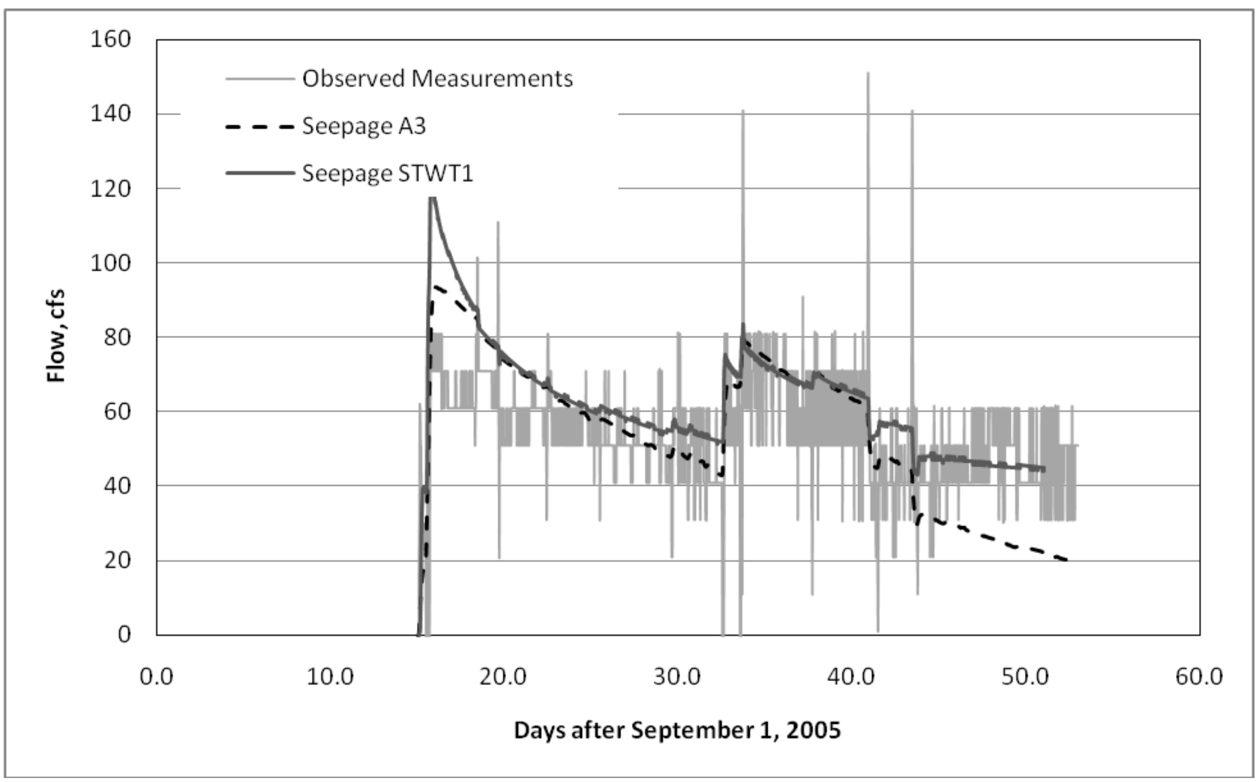


Figure 56. Comparison of seepage from A3 and STWT1 solutions for Area 2.

Hantush (2005) used the A3 model to simulate a hypothetical story event, drought scenario, aquifer drawdown, and regulated reservoir to show the solutions versatility. In this research an actual flood wave was routed using the A3 model and an ok fit was found given the limitations of the model assumptions. If head in the aquifer was an output of the A3 solution, Area 3, which has site conditions that match the models assumptions, could have been simulated to determine the model responded to an actual situation.

4.5 HYDRUS Scenario Results

To complete the HYDRUS modeling three approaches were taken, as outlined in Chapter 3. However, the results of the three-dimensional model and the two-dimensional model with MODFLOW were qualitative, not quantitative. The qualitative results are therefore combined into one section (4.5.2).

4.5.1 Area 1 HYDRUS Scenario Results

HYDRUS is able to simulate the losing condition in Areas 1 and 2 for the entire time series without loss of strength that occurred with the analytical solutions due to the assumption of the system starting from rest. However, because of size constraints only the aquifer 65.6 feet (20 meters) adjacent to the river could be modeled, which did not include the observation wells. Therefore, the ability of HYDRUS to produce the heads in the aquifer is not known for this application. As shown in Figure 57 the HYDRUS solution is able to reproduce the seepage from Post Falls to Harvard Road. The calibrated curve, however, does not have the slight decline shown from the observed measurements. A statistical analysis was done using a model utility test on the slope of the observed measurements after the initial pulse of the flood wave passed (day 16 to day 30) where seepage was predicted by the flow (cfs) at the Post Falls Gage. It was found that the large number of data points lead to very small P-values ($P = 1.7*10^{-23}$), but the R^2 of 0.072 showed that the regression line had very little power to predict the seepage. Also, the USGS only reported values to three significant figures for flow rates on the Spokane River and the flow rate at both Post Falls Gage and Harvard Road Gage are over 1500 cfs. This means that flows are known only to the closest 10 cfs, so when taking the difference of the two gaging stations and including the lag time, the decrease may be artificial. The magnitude of the calibrated HYDRUS curve is also in good agreement with the observed measurements, while the MODFLOW curve produces much less seepage across the riverbed. Returning to the original model data from Hsieh et al. (2007), the zone budget value of seepage given for this reach was 148.5 cfs for the average over the month of September of 2005, which was the last time period modeled. In comparison with Figure 57 the value of seepage that HYDRUS determined for the same K' value of 0.25 is lower. This is interesting as one of the main criticisms of the linear approach used by MODFLOW to determine seepage is that it under predicts seepage rates for disconnected rivers (Brunner et al., 2010). In this case the width of the river used in MODFLOW is 184 feet on average in Area 1; in the HYDRUS scenario the width changes as the depth changes with an average value of 190 feet between the largest and smallest widths in the scenario, so the width would not be cause of the difference. The length of the contributing reach is very nearly the same as well. Any differences in head would have made little impact because MODFLOW assumes that after the water goes below the streambed the hydraulic gradient is unity. The only parameters left affecting the MODFLOW solution are thickness of the riverbed sediments and K', thus the difference must be in the combination of these parameters into one value. In the Hsieh et al. (2007) model the thickness was assumed and set at 1 foot. Any errors associated with the 1 foot assumption would be captured by the value of K'. Thus even though a riverbed sediment layer of 1 foot was used in HYDRUS the combined K’ and thickness value caused an unexpected seepage value. Also the values used in HYDRUS to model the soil (α and n) could have contributed to the difference.

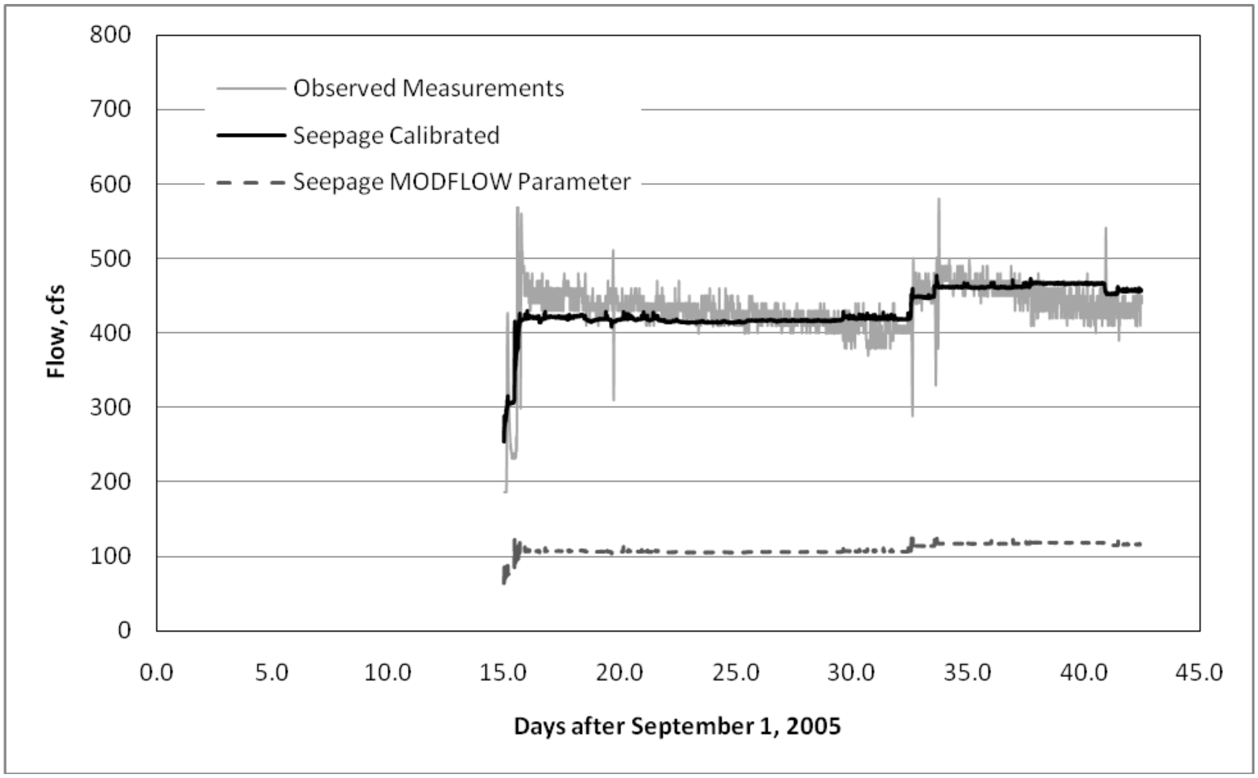


Figure 57. HYDRUS results for Area 1 seepage using calibrated and MODFLOW K' values.

Table 20 shows that the actual fit to the observed seepage rate for the calibrated curve does not have a high NSE value even though by visual inspection the fit is reasonable. Looking at the bias and relative bias the values are good; a bias of -1.5 cfs for seepage rates of 400 cfs is good. The denominator of the relative bias for the A3 solution uses the average of the observed data minus the minimum of the observed data (in this case Min = minimum observed seepage rates = 187 cfs). This put the relative bias in terms of the changes from the flood wave and amplifies the bias in for Area 1. The overall fit in Figure 57 appears very reasonable, while the low NSE value indicates a poor fit. The reason for the low NSE value is the noise of the observed data set and the strength of the Nash-Sutcliffe index for large data sets.

Table 20. Area 1 descriptive statistics for HYDRUS solution results.

		NSE	ē*	R_b %	Min (cfs)	K'	K	α(1/ft)	n
Calibrated	Seepage	0.373	-1.461	-0.3%	187	1.09	22150	1.52	2.08
MODFLOW	Seepage	-77.592	-323.179	-74.6%	187	0.25	22150	1.52	2.08

* ē is in the unit of the criteria variable (Seepage = cfs)

4.5.2 Area 2 HYDRUS Scenario Results

Area 2 showed an interesting feature in the HYDRUS solution. The initial jump in Figure 58 is not because of a jump in the input stream stage, but rather the way in which the solution responded to the initial increase in stream stage. The observed data does not have this same jump for a lag time of 1.5 hours. The overall fit to the observed data is reasonable and as discussed for Area 1 the horizontal part of the HYDRUS curve is likely valid. The September 2005 average MODFLOW seepage value is 98 cfs for the length of the Spokane River between Harvard Road Gage and Barker Road Gage. Similar to Area 1, this is greater than the HYDRUS seepage for the MODFLOW K'. The MODFLOW K' curve has numerous small jumps and declines and may indicate that slight changes in stream stage have a larger effect on K' after some threshold value in the model.

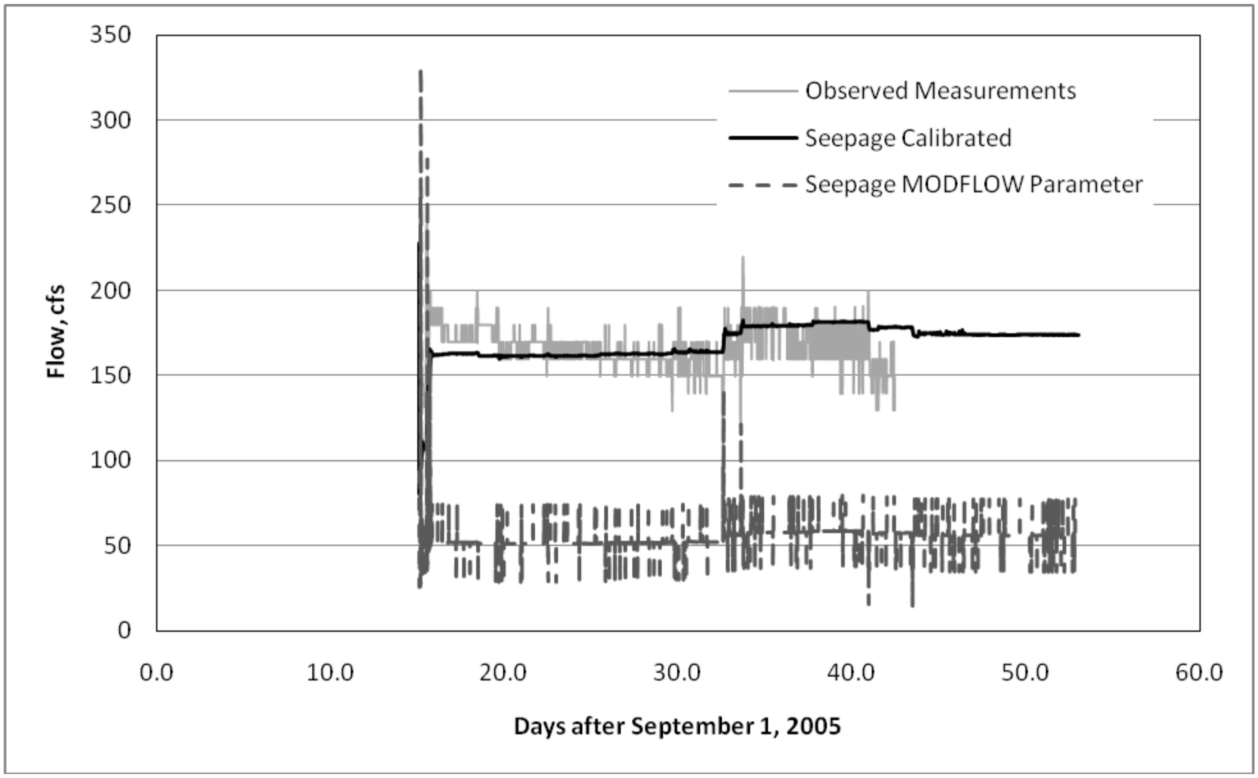


Figure 58. Area 2 Seepage, HYDRUS results for calibrated and MODFLOW K' values.

Both the NSE values are low for Area 2 but are reasonable when the bias and relative bias are considered. Outliners where not removed when the NSE values were determined, removing them would increase the NSE value.

Table 21. Area 2 descriptive statistics for HYDRUS solution results.

		NSE	ē*	R_b %	Min	K'	K	α(1/ft)	n
Calibrated	Seepage	-0.727	0.767	0.5%	100	0.76	19090	1.52	2.08
MODFLOW	Seepage	-79.968	-113.154	-67.9%	100	0.22	19090	1.52	2.08

* ē is in the unit of the criteria variable (Seepage = cfs)

An additional analysis in Area 2 was completed using the linear equation presented in Equation 6, the elevation of the lowest point in the Spokane Riverbed at Barker Road Gage, and the stage change of the Spokane River at the same gage. Two linear lines were established with one based on the constant width of 200 feet (the width for the Barker Road Gage location given by MODFLOW) and the second with a multiplier of the change in stage height added to the width to account for the increased seepage due to an increased width. Figure 59 shows that using the simple relationship of Equation 6 a good fit to the observed data can be established. The difference between the increased width and the constant width show the importance of including a width adjustment to the equation as each new width gives a new conductance factor. Given in Table 22, the value of K' used to achieve the fit shown was 0.48 ft/day, which is still higher than the value given in the calibrated version of MODFLOW but less than the K' needed to achieve seepage in HYDRUS. This simple analysis seems to indicate that the saturated condition assumption of MODFLOW allows for less resistance to flow and a lower value of K' to produce similar seepage rates for the unsaturated condition represented by HYDRUS. This may indicate that for a strictly losing reach the increased suction from the unsaturated conditions can be counterbalanced by the decrease in hydraulic conductivity associated with a decrease in soil moisture, to a degree where Equation 6 produces a higher resistance.

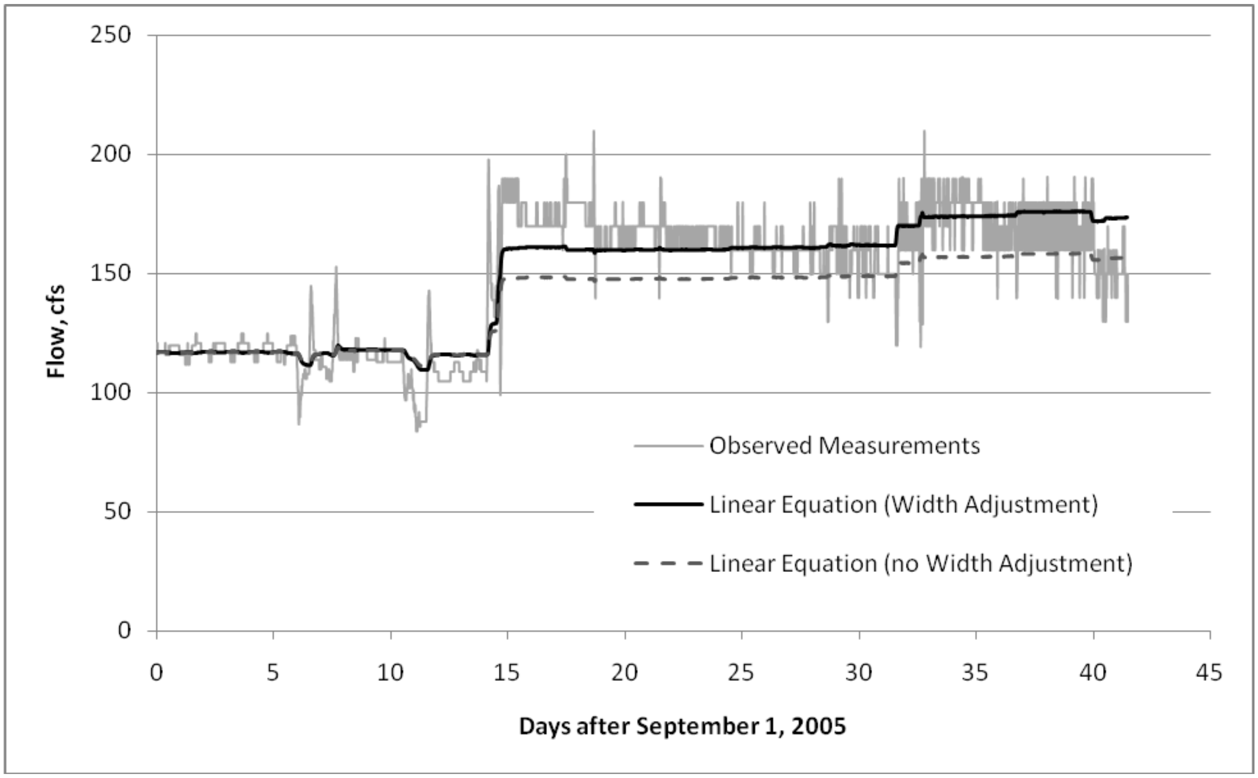


Figure 59. Comparison of Equation 6 with Observed Measurements

Table 22. Descriptive Statistics for Equation 6 comparison

	NSE	ē*	R_b	K' (ft/day)	Width Adj. Factor	Width(ft)
With Width Adj	0.700	-7.411	-5.88%	0.48	10.9	200
Without	0.841	0.029	0.02%	0.48	-	200

* ē is in the unit of the criteria variable (Seepage = cfs)

HYDRUS was successfully used in another study detailing the affects of the streambed sediment layer (Shawaqfah, 2002). In that study it was found that HYDRUS was able to match the STWT1 solution for simple aquifer/stream interaction almost exactly with or without a reduce hydraulic conductivity streambed layer. The assumptions used in were a connected stream and water levels in the stream and aquifer where the same before the change in stream stage (Shawaqfah, 2002). Very different conditions are present in the SVRP aquifer and

HYDRUS was able model high K aquifers and fast changes in seepage, with K as a function of water content.

4.5.3 2D and 3D with MODFLOW HYDRUS Scenarios

The purpose of using HYDRUS in a three-dimensional application was to be able to predict both head in the aquifer at the various observation wells and the seepage from the Spokane River. Calibrated values for K' and K could then be established with more realistic boundary conditions and without the limitations of the assumptions in other models. HYDRUS has the ability to model a fully three-dimensional setting, both unsaturated and saturated flow, and to represent losing, gaining, or transitional stream reaches in detail. However, in order to integrate the aquifer boundaries into the model the domain exceeded the available memory of the computer hardware used. Grid cells near the Spokane were set to about 2.5 feet and closer to the aquifer boundaries over 131 feet. Due to the highly non-linear nature of the Richards equation such large cells did not allow for solutions to converge and the three-dimensional model had to be abandoned. The result of the attempt is the knowledge that HYDRUS was good for looking at small areas, but could not be used for large-scale interaction problems.

The ability to model both head and seepage was a distinct advantage and a second attempt was made for a three-dimensional representation. HYDRUS was reduced to the two dimensional setup of the model as shown in Figure 22 and was linked to MODFLOW. Thus seepage from HYDRUS could be directly applied in MODFLOW through the recharge package. The recharge package avoided the linear seepage representation used in MODFLOW and cells could be selected that matched the path of the channel. HYDRUS and MODFLOW were both successfully run, but the appropriate determination of the boundary conditions became a problem. The aquifer boundary could be easily represented in MODFLOW (see Figure 23 Area

1 for delineation of 3D boundaries), but the eastern and western boundaries of Area 1 in the aquifer were a problem. The high hydraulic conductivity caused large amounts of groundwater flow to enter through the transient head boundary condition and that flow governed the head distribution. The result was an increased understanding of the head distribution near the Spokane River in Area 1 (the only area attempted). Flownets where used to give a best guess solution to what the actual existing head distribution might be. While crude, this approximation was later confirmed by Figure 7 in the Caldwell and Bowers (2003) report. Figure 7 (Caldwell and Bowers, 2003) shows the water levels in about 70 wells between Post Falls and Sullivan Road. The resulting head distribution shows a steep hydraulic gradient perpendicular to the Spokane River very close to the river and a much lower hydraulic gradient parallel to the river when moving toward the aquifer boundaries. Figure 23 shows the location of the Spokane River in the SVRP aquifer between Post Falls and Sullivan Road Bridge Gage.

4.6 MODFLOW Scenario Results

Overall the MODFLOW scenario served two purposes: 1) to provide a direct comparison between MODFLOW and the other solutions used for the wells in the 6-Minute study, and 2) to produce a solution that included three-dimensional flow to determine whether the timing and magnitude of upstream flow affected the artificial flood wave produced by Post Falls Dam. Of the thirty wells used in the 6-Minute Data study, selected wells will be shown to highlight different parts of the data, the rest can be found in Appendix A. Some wells not used in the analytical solutions are shown as they provided an analysis of the MODFLOW data for important areas of the regional model.

Figure 8 and Figure 23 show the location and the identification number of the wells used in the MODFLOW scenario. The results are shown in Table 23 and figures are given afterward to show visually how the results in Table 23 performed.

Table 23. MODFLOW Scenario Descriptive Statistics

Well Name	NSE	$\bar{e}$*	R_b	$\bar{Y}$	Δ Head (ft)	Count
Well 6	-1.35	1.52	39.0%	1944.42	3.90	4744
Well 9	-0.15	-0.59	-17.7%	1960.68	3.34	4745
Well 11	-18.97	-4.70	-152.7%	1953.72	3.07	4741
Well 12	-2164.67	42.96	1627.2%	1885.15	2.64	4752
Well 13	-1.10	-0.90	-37.0%	1977.68	2.43	4740
Well 14	-21.80	-4.53	-152.6%	1931.29	2.97	4749
Well 15	-0.78	0.83	32.4%	1974.37	2.57	4739
Well 16	-0.52	0.76	27.5%	1973.16	2.75	4738
Well 18	-1.40	1.07	48.1%	1877.57	2.23	4752
Well 19	-12.25	-4.24	-127.6%	1950.45	3.32	4743
Well 20	-10.39	-6.09	-111.1%	1759.48	5.49	4752
Well 22	-4235.19	26.26	834.9%	1694.12	3.32	4752
Well 28	-19.13	-5.32	-160.3%	1951.16	3.30	4743
Well 29	-3.57	-2.27	-68.7%	1961.30	2.11	4746
Well 31	-339.84	14.25	677.0%	1861.81	2.32	4752
Well 32	0.25	-0.30	-13.1%	1878.56	1.63	4752
Well 33	-48.17	3.61	221.7%	1908.94	3.38	4752
Well 34	-18.99	5.45	161.2%	1931.79	3.03	4747
Well 35	-16.64	4.30	142.0%	1864.11	2.00	4751
Well 36	-7.38	1.77	88.2%	1854.81	4.07	4752
Well 37	-9.30	1.99	48.7%	1883.19	3.14	4752
Well 39	-864.36	23.50	1002.0%	1852.20	2.34	4752
Well 42	-53.37	4.26	239.3%	1862.23	1.78	4749
Well 43	-0.88	0.64	38.2%	1889.89	1.67	4752
Well 44	-23.80	3.57	177.7%	1870.49	2.01	4752
Well 45	-115.55	-5.71	-317.3%	1842.33	1.80	4745
Well 46	-4.43	-2.78	-78.0%	1952.92	3.57	4744
Well 47	-676.12	3.14	602.9%	1846.01	0.52	4748
Well 48	-16.11	-4.81	-145.2%	1951.36	3.31	4742
Well 49	-15.27	4.44	145.0%	1932.54	3.06	4748

* $\bar{e}$ is in the unit of the criteria variable (head = ft)

In Table 23 the shaded rows are the wells in Areas 1-4 and the non-shaded rows are the others wells used in the 6-Minute Data study. The R_b values are often over 100%, due to the effort to relate $\bar{e}$ to a meaningful value in the observed data. Thus R_b is the value of $\bar{e}$ divided by the change in head at each well. This shows how the magnitude of the bias in the model compares to the actual change in the head during the scenario. Values over 100% indicate that the bias is greater than the change in the head during the scenario, and NSE values indicate poor fit for those values. The highest NSE value is 0.25 for Well 32 and the lowest NSE value is -4235.19 for Well 22 (graphs for both are shown in Figure 60 and Figure 61). In general the MODFLOW scenario does not fit the observed values well. For Wells 22 and 12 the fits are very poor and it would seem that an error in the height of the measurement might be possible; there is nearly a 25-foot and 44-foot difference between the values in the observation wells and the value produced by MODFLOW.

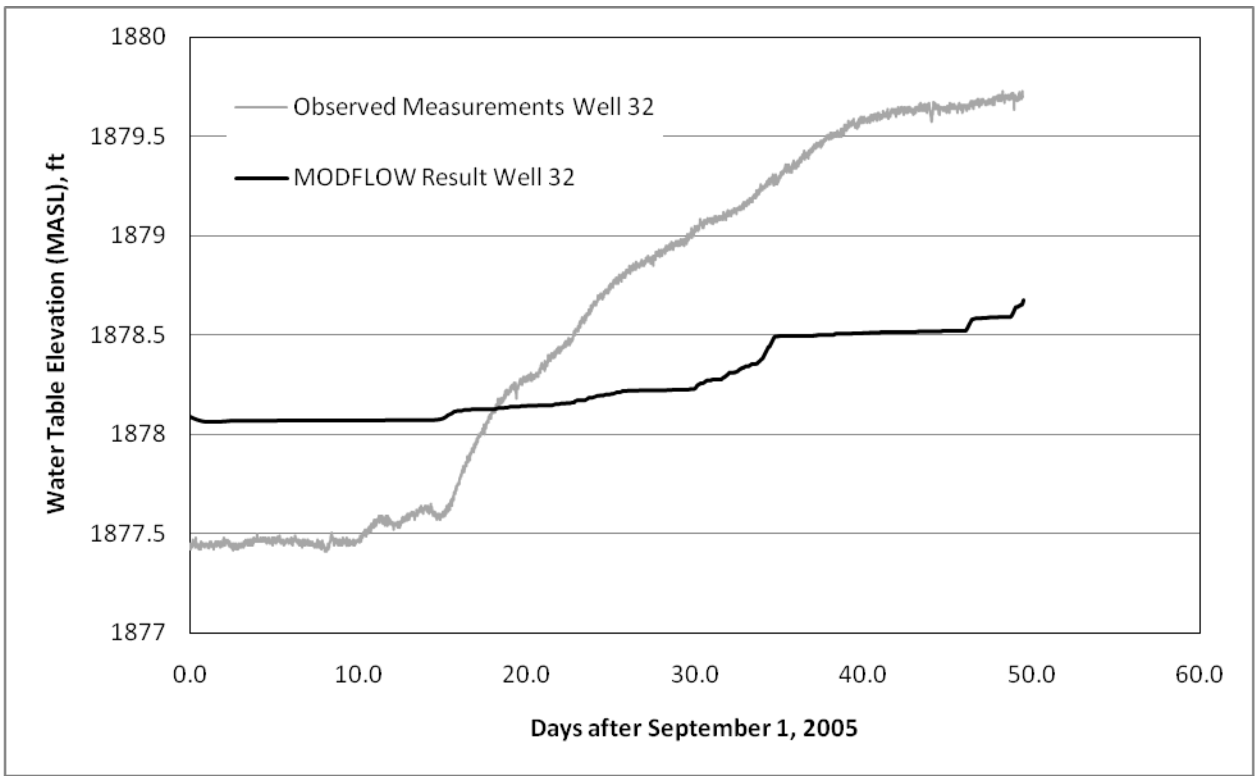


Figure 60. Best fit NSE value for Well 32 using MODFLOW.

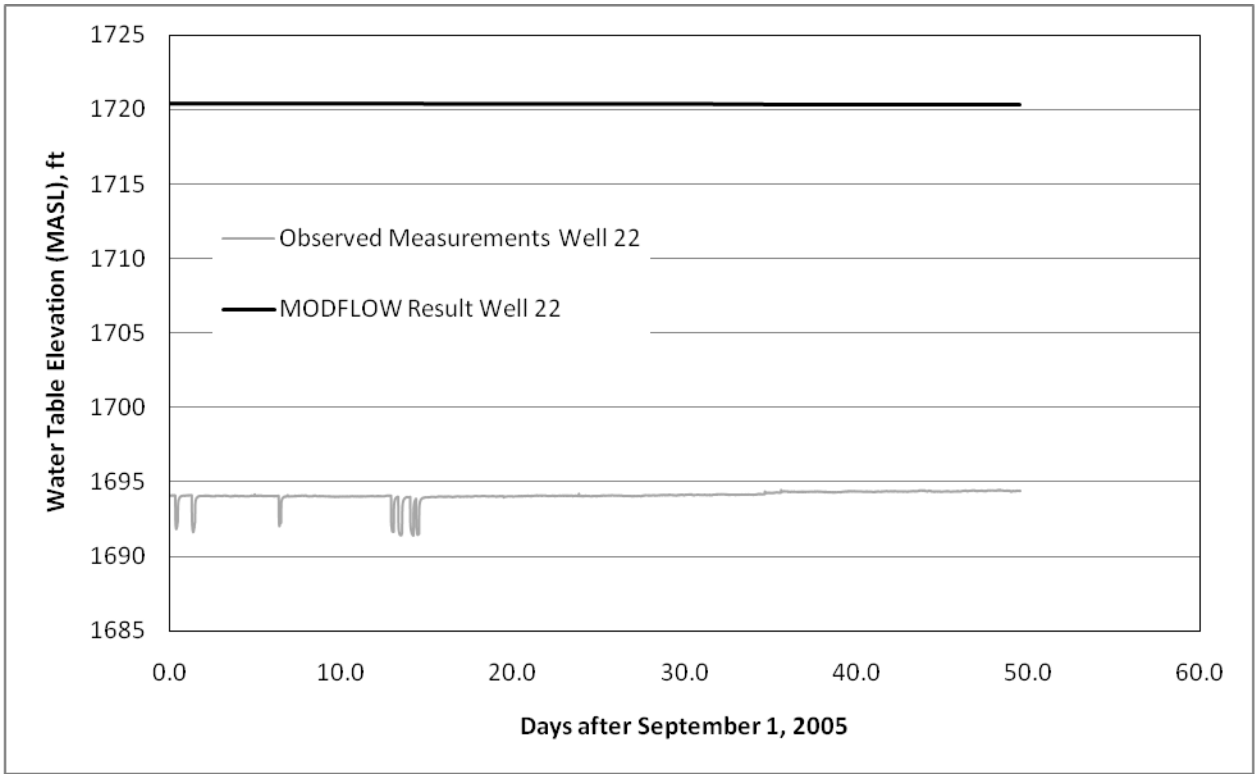


Figure 61. Worse fit NSE value for Well 22 using MODFLOW.

Figure 60 shows that the MODFLOW scenario values are able to capture the timing of the changes in the head due to changes in river stage but not the magnitude of the change. The difference between the initial elevation of the observation data and the MODFLOW values is not a good indicator of the model's ability to reproduce the artificial flood wave. The grid cells in the MODFLOW model are 0.25 miles by 0.25 miles and the value of head is only calculated at the center of the cell. Since the observation wells do not correspond directly to the center of the MODFLOW cells some difference in initial elevation is expected. Large differences such as in Figure 61 are too great to be accounted for by the spatial differences in observation well and MODFLOW head measurement location and represent error in one or both of the head values. The most important result shown in Figure 60 and Figure 61 is the difference in magnitude of the change in the MODFLOW values and the 6-Minute Data. This difference confirms the results of higher K' values in the analytical solutions and shows a misrepresentation of the interaction between the SVRP aquifer and the Spokane River during the low flow summer months in the calibrated MODFLOW model. Figures 62-65 show the best fitting well in each of the areas used for the analytical solutions.

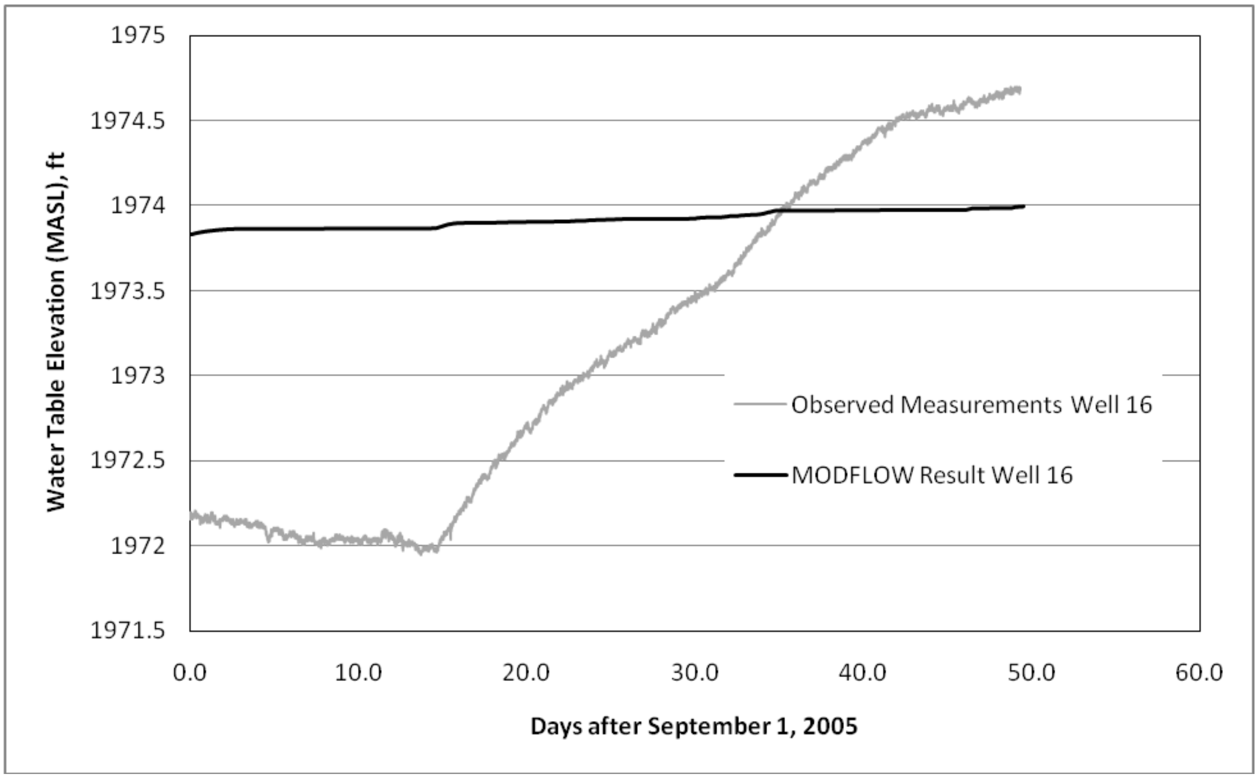


Figure 62. Comparison of observed and calculated head for Well 16 in Area 1 using MODFLOW.

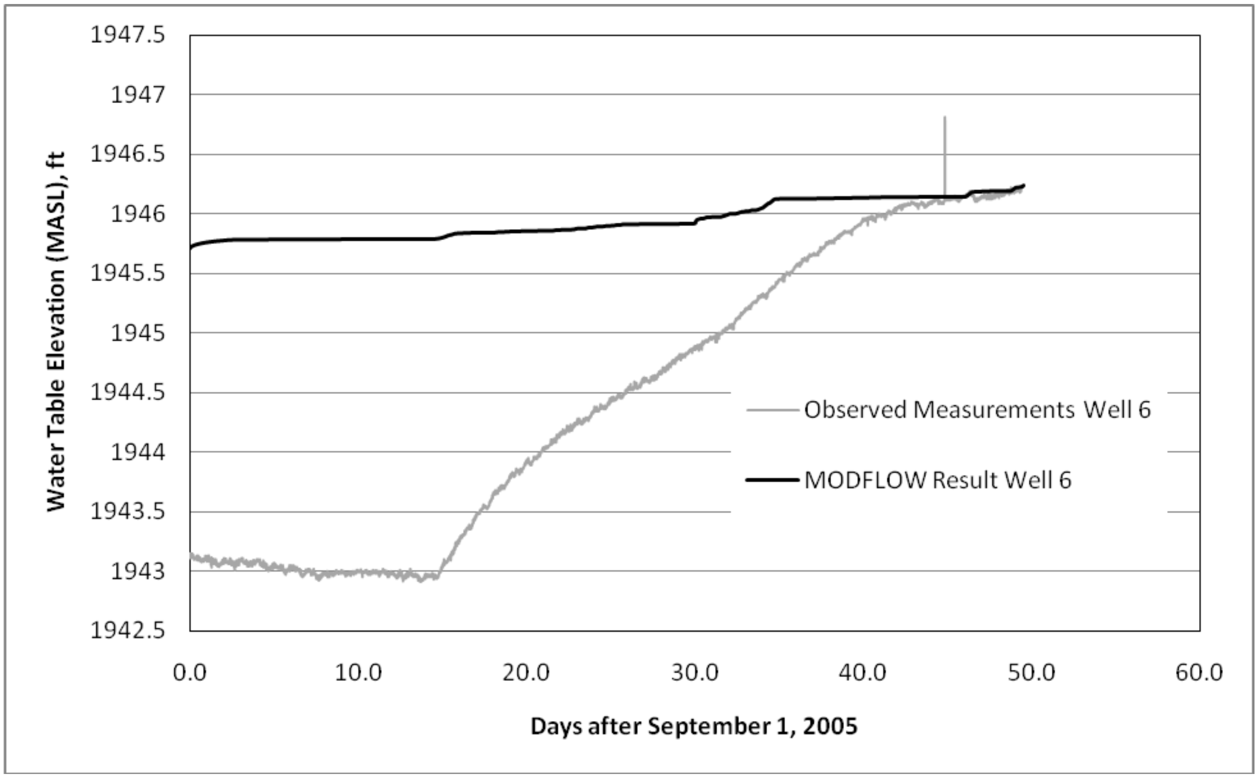


Figure 63. Comparison of observed and calculated head for Well 6 in Area 2 using MODFLOW.

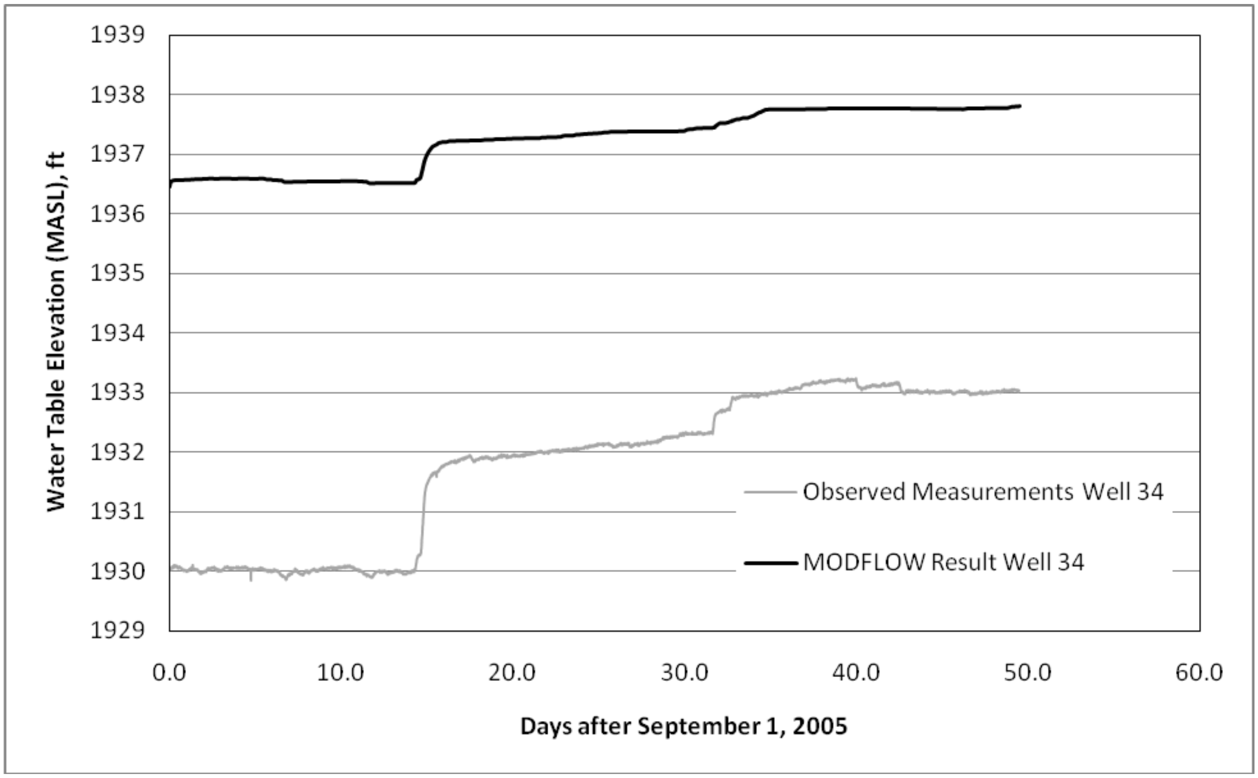


Figure 64. Comparison of observed and calculated head for Well 34 in Area 3 using MODFLOW.

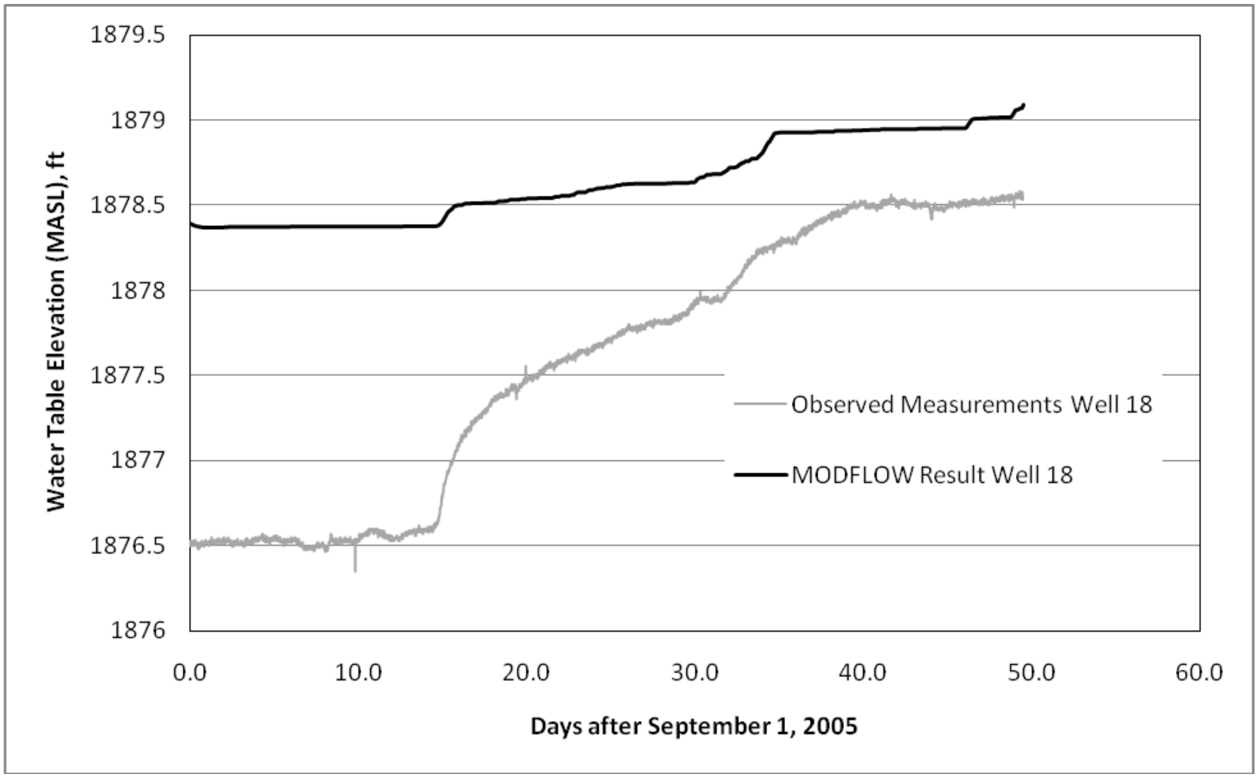


Figure 65. Comparison of observed and calculated head for Well18 in Area 4 using MODFLOW.

The timing of the river stage change is fairly well represented in Figures 62-65 and shows the ability of the currently calibrated MODFLOW model to correctly predict the distance the water has moved. In Areas 1 and 2 the magnitude of the change in the MODFLOW results was much less than given by the observed data in 6-Minute Data study. Wells down gradient in the aquifer of Areas 1 and 2 were not able to achieve the magnitude of change either. However, Areas 3 and 4 show a much greater response to the change in stream stage. Effects of three-dimensional flow are minor in MODFLOW, as seen by the flattening of each curve produced. This result could be due to the lack of flow from Areas 1 and 2. Together the difference in the magnitudes of the changes in head and the lack of three-dimensional response indicate that by changing the hydraulic conductivity of Areas 1 and 2, the response predicted by MODFLOW in the wells downstream may be improved.

4.7 Sensitivity Analysis

For the Sensitivity Analysis, if multiple wells were used in an Area only the well closest to the Spokane River will be used. Calibrated values of K' and S_y were often determined independently for each well in the respective area, however only one well is needed to establish sensitivity in an Area. The sensitivities for each model in each area will be compared and the most sensitive parameters will be discussed in the context of each solution.

Table 24. Sensitivity Analysis values

Solution	Area	Parameter	Increase			Decrease		
			NSE	$\bar{e}$	R_b	NSE	$\bar{e}$	R_b
STWT1	Area 1	a	0.880	-0.157	-11.6%	0.926	-0.075	-5.6%
		σ	0.930	-0.066	-4.9%	0.865	-0.175	-12.9%
		x_D	0.907	-0.117	-8.6%	0.906	-0.118	-8.7%
	Area 2	a	0.209	-0.668	-31.7%	0.266	-0.613	-29.1%
		σ	0.265	-0.616	-29.2%	0.205	-0.670	-31.8%
		x_D	0.239	-0.641	-30.4%	0.240	-0.640	-30.4%
	Area 3	a	0.980	-0.029	-1.2%	0.975	0.024	1.0%
		σ	0.979	0.018	0.7%	0.978	-0.026	-1.0%
		x_D	0.980	-0.004	-0.1%	0.980	-0.002	-0.1%
	Area 4	a	0.876	-0.156	-10.9%	0.883	-0.145	-10.2%
		σ	0.886	-0.142	-9.9%	0.871	-0.161	-11.3%
		x_D	0.880	-0.151	-10.6%	0.880	-0.150	-10.5%
A2	Area1	ξ	0.562	-0.317	-23.5%	0.438	-0.390	-28.9%
		xbar	0.461	-0.381	-28.2%	0.551	-0.320	-23.7%
	Area 2	ξ	-0.525	-0.954	-45.3%	-0.724	-1.036	-49.3%
		xbar	-0.619	-0.994	-47.3%	-0.612	-0.990	-47.1%
	Area 3	ξ	-1.344	0.749	44.6%	0.186	0.384	22.9%
		xbar	-0.577	0.593	35.3%	-0.596	0.598	35.6%
	Area 4	ξ	0.688	-0.252	-17.7%	0.658	-0.269	-18.9%
		xbar	0.658	-0.270	-18.9%	0.691	-0.250	-17.5%

A3 Outflow	Area 1	R	0.577	-221.111	-14.5%	0.382	-268.448	-17.6%
		D	0.535	-231.657	-15.2%	0.438	-256.144	-16.8%
		η	0.314	-283.218	-18.6%	0.643	-202.535	-13.3%
		ξ	0.472	-247.658	-16.2%	0.505	-239.701	-15.7%
	Area 2	R	0.957	-52.941	-3.5%	0.942	-59.550	-3.9%
		D	0.956	-49.826	-3.3%	0.943	-63.239	-4.1%
		η	0.933	-66.886	-4.4%	0.965	-45.415	-3.0%
		ξ	0.950	-56.259	-3.7%	0.950	-56.148	-3.7%
A3 Seepage	Area 1	R	0.110	-18.387	-7.5%	0.253	5.425	2.2%
		D	0.213	-13.108	-5.3%	0.320	-0.735	-0.3%
		η	0.124	11.933	4.9%	-0.174	-26.769	-10.9%
		ξ	0.321	-5.079	-2.1%	0.286	-8.983	-3.7%
	Area 2	R	-0.217	-1.195	-2.2%	-0.723	2.101	3.9%
		D	-0.523	-2.767	-5.2%	-0.397	3.963	7.4%
		η	-0.713	5.506	10.3%	-0.346	-4.691	-8.8%
		ξ	-0.422	0.463	0.9%	-0.412	0.404	0.8%
HYDRUS	Area 1	α	0.255	-12.978	-3.0%	0.219	14.508	3.3%
		n	0.223	-14.514	-3.4%	–	–	–
		Ksat	0.318	0.735	0.2%	0.359	-4.918	-1.1%
		K'sat	-0.778	38.996	9.0%	-0.939	-42.252	-9.8%
		Grid Size	0.376	-1.753	-0.4%	0.378	-0.862	-0.2%
		Depth to Water Table	0.377	-1.501	-0.3%	0.377	-1.463	-0.3%
	Area 2	α	-0.800	-9.631	-3.0%	-1.026	11.487	3.5%
		n	-0.811	-11.165	-3.4%	–	–	–
		Ksat	-0.778	4.357	1.3%	-0.703	-1.049	-0.3%
		K'sat	-2.677	33.072	10.2%	-2.105	-30.487	-9.4%
		Grid Size	-0.730	-2.097	-0.6%	-0.756	1.265	0.4%
		Depth to Water Table	-0.730	1.514	0.5%	-0.721	1.511	0.5%

For the STWT1 solution σ produced the largest range of NSE values followed closely by α, where σ and α are defined below.

$$\sigma = \frac{S_S b}{Sy} \qquad \text{Equation 18}$$

$$\alpha = \frac{Kd}{K'} \qquad \text{Equation 19}$$

where S_S is specific storage [], b is saturated thickness of the aquifer [L], and d is the thickness of the riverbed material [L]; S_y, K, and K' are as defined previously. The combination of the variables therefore, requires that four out of the six be defined prior to the solutions with a strong degree of certainty and that two of the six could be determined from calibration. X_D is the ratio of distance to the well divided by the width of the river, which for this solution does not change the result significantly.

For the A2 solution ξ usually produces a larger range that Xbar. ξ is a combination of most of the input parameters in the A2 solution and is therefore not surprisingly the most sensitive. Xbar is the ratio of distance to the well divided by the width of the river, which was shown by Zlotnik and Huang (1999) to be a sensitive parameter. Thus for the A2 solution width of the river is important and should be known with a strong degree of certainty.

For the A3 solution η and ξ produced the largest range of NSE values and R (retardation (Tb/PK') [L]) and D (aquifer diffusivity (T/S_y) [L^2/t]) less so. To get accurate values of K' and S_y, η and ξ should be known prior to a strong degree of certainty. As was discussed η and ξ can be established in the calibration process, but values may not match the physical conditions and can skew the values of K' and S_y.

For the HYDRUS model the value of K'sat was the most sensitive parameter. The value of n was also sensitive and decreases below 2 caused the solution to not converge. This was also demonstrated by Ippisch et al. (2006) with the van Genuchten-Mualem model which is limited with the air entry value. Increases in n were the second most sensitive and α was the third. Thus for HYDRUS, good estimates of n and α are needed for accurate estimates of the other parameters. The value of the thickness of the riverbed sediment layer was not tested even though it affects the estimate of the value of K'sat. Lastly, changing the value to the depth of the water table had little influence on the fit to the observed data. This means that the water table elevation produced by MODFLOW at quarter-mile grids cells is acceptable for use in the finer grid mesh used for a strictly losing condition.

In addition to the sensitivity reported above, the STWT1 solution also used a numerical inversion technique (Stehfest algorithm) to get from the Laplace domain to the real time domain. The number of terms in the Stehfest algorithm was changed from 8 to 6 and from 8 to 12 while the factor used to determine the number of terms in the finite sums for head and seepage was changed from 20 to 30 to 40. These changes produced differences in the final value of head seepage and bank storage of less than 0.5% in all cases. The low differences established the recommendation given by Moench and Barlow (2000) for those parameters as acceptable for the SVRP aquifer. Likewise in HYDRUS changes in the precision of the head and water content tolerance values from 0.001 to 0.0001 only changed the value of the NSE parameter by 2.2% or less. This established the tolerance of 0.001 as small enough to produce results that had a low influence on numerical error.

By establishing the range in which the K' and S_y are values valid for a given solution, the certainly that must be achieved to give meaning to the calibrated parameters is known. Without understanding the sensitivity of each calibrated parameter value to the given model assumptions and collected data, conclusions of analyses could be overstated or misrepresented. Thus the sensitivity analysis is essential in presenting which calibrated parameter values are extremely sensitive to alterations of model parameters.

Chapter 5: Conclusions and Recommendations

5.1 Conclusions

A greater understanding of groundwater/surface water interaction in the SVRP aquifer has been developed using five approaches to analyze the artificial flood wave produced by the opening of the Post Falls Dam at 22:30 on September 14, 2005. Four different areas were investigated with wells nearly perpendicular to the gaged locations in the Spokane River. Of specific concern was how the conceptualized flow method affected the resulting heads and seepage rates into the SVRP aquifer. Conclusions will be presented for the main result of each objective.

Objective 1) The three analytical solutions gave different resulting values of K' and S_y for each area and, if applicable, each well in the area. The assumptions in each model were in most cases very similar, which confirms the differences were from the model and not the initial assumptions. Thus the method of conceptualization of GW/SW interaction affects the calibrated parameters values. Specifically, the governing equation (top-down, or bottom-up), the ability to incorporate partially penetrating streams, and finite or infinite boundary conditions were the main differences in model conceptualization that affected the calibrated parameters values. Table 25 shows the average K', S_y, and NSE value for Areas 1-4 for each model. The seepage rates are given in Figures 30, 36, 52, 53, 55, and 56.

Table 25. K' and S_y values for Areas 1-4

Area	Model	K'	S_y	NSE*
1	STWT1 (Head)	1.8	0.3	0.945
	STWT1 (Seep)	1.8	0.3	-10.300
	A2	7.1	0.3	0.394

	A3	2	0.3	0.398
2	STWT1 (Head)	1.05	0.23	0.251
	STWT1 (Seep)	1.05	0.23	-7.591
	A2	6.2	0.3	-0.654
	A3	1.5	0.3	0.267
3	STWT1 (Head)	4	0.3	0.980
	A2	11	0.19	0.185
	A3	N/A	N/A	N/A
4	STWT1 (Head)	6	0.3	0.917
	A2	24.5	0.25	0.519
	A3	N/A	N/A	N/A

* NSE values are the average value for all wells in the given area

Objective 2) It was seen that the unsaturated zone produced a value of K' in the HYDRUS model to 1.09 for Area 1 and 0.76 for Area 2, with NSE values of 0.373 and -0.727, respectively. These values are lower than in Table 25 but not as low as the calibrated MODFLOW model for Areas 1 and 2. Thus the calibrated MODFLOW model is under predicting the values of K' into Areas 1 and 2 which are strictly losing reaches. Both the STWT1 and A3 analytical models give K' values that are close to the HYDRUS Model, which indicates that the conceptualization of GW/SW interaction in those models is less affected by the unsaturated zone.

Objective 3) The flood wave was studied in the MODFLOW model of the SVRP aquifer. The results at all the observation wells show mismatches in head values and in some cases a different trend in the increase in head at those wells. Large grid cell size contributes to the errors near the Spokane River but the analysis revealed that some areas in the model have flow patterns different from those observed in the field. The culmination of the aforementioned errors lead to very poor accuracy of the

MODFLOW model, with errors of over 100% of the measured change in head from the flood wave (Table 23).

Overall, the approach of using three analytical models and two numerical models has increased the knowledge of the GW/SW interaction in the SVRP aquifer and the influence of the model conceptualization of GW/SW interaction. In the SVRP aquifer 3D flow patterns strongly influence the measured data after 5-10 days and limit the application of analytical model of less dimensionality. Each analytical model was limited by the assumption of the aquifer and river initially having the same water surface elevation. The conceptualization of the leakage relationship caused the value of hydraulic conductivity to be up to an order of magnitude different. For unsaturated flow HYDRUS was found to be very practical for small scale interactions but could not incorporate regional scale dimensions.

The significance of understanding the conceptualization of the leakage relationship used in each model is how the calibrated values are to be used. If they are a preliminary estimate, each model will provide a K' value or S_y value that will work as an initial guess in a numerical model or to model a flood wave with similar initial flow and total change in flow. If the calibrated value is used in another model with a different conceptualization of the leakage relationship, erroneous results should be expected. Since both K' and S_y are important parameters for aquifer response to stresses, the above analysis helps establish the need for a conceptualization of the leakage relationship that accounts for the mechanisms of the exchange. For the SVRP aquifer the STWT1 solution does not represent the physical conditions of partial penetration in the aquifer. While calibration of the STWT1 model can establish a response that matches head changes in the aquifer, using the same values for the spring runoff period in the SVRP aquifer would likely require additional calibration. The A2 solution correctly modeled the interaction between the GW/SW, but had a semi-finite boundary. The incorrect boundary condition produced significant differences from measured aquifer response and overall should not be used for predictions in the SVRP aquifer. The A3 solution best represented of the physical condition in the SVRP aquifer of the interaction between the GW/SW for the three analytical models. The A3 model could be used to make predictions, but with the understanding of the limitations of the initial assumptions. Both HYDRUS and MODFLOW present realistic conceptualization of the GW/SW interaction and aquifer boundary conditions. Prediction in both models could be used for future planning activities. With the main purpose of a model being the ability to predict changes in response to future conditions, both city planners and irrigation managers could benefit from the results and discussion of the conceptualization of GW/SW interaction presented in this Thesis.

5.2 Recommendations for future work

The study presented has explored all of the above-mentioned cases and conditions in modeling the interaction between the SVRP aquifer and the Spokane River. Having completed the goals of this study, additional areas of research and additional data that would help further the understanding of the conceptualization of the leakage relationship and determining hydraulic conductivity and specific yield in the SVRP are discussed below.

Additional areas of research:

a) Using HYDRUS, complete a study that looks at the value of K’ as the elevation of the water surface in the river increases to the high flow value. This would help establish if seasonal changes in K’ were present. Also it could establish if the K’ value was the same for the whole river channel or if portions of the banks not continuously saturated have different properties from those that are.

b) Using HYDRUS, consider the affect of the riverbed thickness to determine the overall impact of lumping uncertainties in K' and the riverbed thickness.

c) Using HYDRUS, develop a 3D model that looks at changed in head in the aquifer for just the changes in the stage of the Spokane River. Compare head changes in the HYDRUS model to the observed measurements from the 6-Mintue Study to establish the timing and magnitude of increase in regional groundwater movement from the artificial flood wave.

d) Using MODFLOW, complete a study with grid size equal to the width of the Spokane River (or less) to produce more accurate flow fields and head values near the Spokane River. Incorporate both the seasonal high flow and low flow to establish K' values and if the linear leakage relationship can reproduce seepage rates and head in the aquifer.

e) Develop an analytical model that considered both a finite aquifer width and a partially penetrating stream exactly; such a solution would be desirable for regional aquifers such as the SVRP.

f) Modify the A3 model to output head changes to determine how the top-down model reproduces measurements in the aquifer.

g) Analyze the 6-Mintue Study data for three dimensional flow patterns in the timing and total change in head to understand groundwater movement.

Additional data:

a) Since head values alone are not enough to calibrate an analytical solution, a higher density of river gages in the Spokane River is needed. Strategic points have been identified from the low flow condition where the Spokane River changes between a losing or gaining reach. Stream stage gages at these transitional points and at locations where the hydraulic conductivity of the riverbed sediments changes are crucial.

b) Elevations of the gages in the Spokane River need to be established so that direct comparisons to the elevation in the aquifer can be determined.

c) River gages near dams should be established near the spillway, the start of the slack water from the dam, and downstream of the dam as near as possible to the spillway but still providing accurate measurements.

d) Establish observations wells closer to the river that are part of perpendicular lines to gaged sections of the Spokane River at regular intervals. This would enable the propagation of the response to the flood wave in the aquifer to established and determine changes in the hydraulic conductivity of the aquifer materials as move away from the Spokane River.

e) Install observation wells adjacent to the impoundments of the to add in predicting heads in those locations.

f) Develop current cross-sections at each of the gages in the river so that properties of the interaction dependent on the riverbed geometry can be determined.

g) Observational wells between Sullivan Road and Trent Street Bridge need be spatially increased and should run both perpendicular to the direction of Spokane River and parallel to the regional groundwater direction of flow.

References

Akylas, E., Koussis, A.D., 2007. Response of sloping unconfined aquifer to stage changes in adjacent stream. I. Theoretical analysis and derivation of system response functions. Journal of Hydrology 338, 85-95.

Alley, W.M., Healy, R.W., LaBaugh, J.W., Reilly, T.E., 2002. Flow and Storage in Groundwater Systems. Science 296, 1985-1990.

Anderson, E.I., 2003. An analytical solution representing groundwater-surface water interaction. Water Resources Research 39 (3), pp. 7.

Anderson, E.I., 2005. Modeling groundwater-surface water interactions using the Dupuit approximation. Advances in Water Resources 28, 315-327.

Anderson, M.P., Woessner, W.W., 2002. Applied groundwater modeling: simulation of flow and advective transport. Academic Press, London.

Arntzen, E.V., Geist, D.R., Dresel, P.E., 2006. Effects of Fluctuating River Flow on Groundwater/surface water Mixing in the Hyporheic zone of a Regulated, Large Cobbel Bed River. River Research and Applications 22, 937-946.

Barlow, P.M., Desimone, L.A., Moench, A.F., 2000. Aquifer response to stream-stage and recharge variations. II. Convolution method and applications. Journal of Hydrology 230, 211-229.

Barlow, P.M., Moench, A.F., 1998. Analytical Solutions and Computer Programs for Hydraulic Interaction of Stream-Aquifer Systems. , 1-98.

Bartolino, J.R., 2007. Assessment of Areal Recharge to the Spokane Valley-Rathdrum Prairie Aquifer, Spokane County, Washington, and Bonner and Kootenai Counties, Idaho. Scientific Investigations Report 2007-5038 , 1-38.

Birkhead, A.L., James, C.S., 2002. Muskingum river routing with dynamic bank storage. Journal of Hydrology 264, 113-132.

Blaschke, A.P., Steiner, K., Schmalfuss, R., Gutknecht, D., Sengschmitt, D., 2003. Clogging Processes in Hyporheic Interstices of an Impounded River, the Danube at Vienna, Austria. International Review of Hydrobiology 88 (3-4), 397-413.

Bolke, E.L., and Vaccaro, J.J., 1981, Digital-model simulation of the hydrologic flow system, with emphasis on ground water, in the Spokane Valley, Washington and Idaho: U.S. Geological Survey Open-File Report 80-1300, 43 p.

Bolster, C.H., Genereux, D.P., Saiers, J.E., 2001. Determination of specific yield for the Biscayne Aquifer with a canal-drawdown test. Ground Water 39 (5), 768-777.

Brunner, P., Cook, P.G., Simmons, C.T., 2009. Hydrogeologic controls on disconnection between surface water and groundwater. Water Resources Research 45 (W01422), pp. 13.

Brunner, P., Simmons, C.T., Cook, P.G., 2009. Spatial and temporal aspects of the transition from connection to disconnection between rivers, lakes and groundwater. Journal of Hydrology 376, 159-169.

Brunner, P., Simmons, C.T., Cook, P.G., Therrien, R., 2010. Modeling Surface Water-Groundwater Interaction with MODFLOW: Some Considerations. GROUND WATER 48 (2), 174-180.

Buchanan, J.P., 2000, Unified groundwater flow model of the Rathdrum Prairie-Spokane Valley aquifer system: Prepared for Water Quality Management Program, Spokane County Public Works and Idaho Division of Environmental Quality: Cheney, Eastern Washington University, 23 p.

Carslaw, H.S., Jaeger, J.C., 1959. Conduction of heat in solids, 2nd ed. Oxford at the Clarendon Press, London.

CH2M Hill, 1998, City of Spokane wellhead protection program phase I—Technical assessment report: CH2M Hill report for the City of Spokane Wellhead Protection Program, 2 vols., variously paged, 12 appendixes.

Chen, X., Chen, X., 2003. Stream water infiltration, bank storage, and storage zone changes due to stream-stage fluctuations. Journal of Hydrology 280, 246-264.

Chow, V.T., Maidment, D.R., Mays, L.W., 1988. Applied Hydrology. McGraw-Hill.

Christensen, S., 2000. On the estimation of stream flow depletion parameters by drawdown analysis. Ground Water 38 (5), 726-734.

Christensen, S., Zlotnik, V.A., Tartakovsky, D.M., 2010. On the use of analytical solutions to design pumping tests in leaky aquifers connected to a stream. Journal of Hydrology 381, 341-351.

Constantz, J., 2008. Heat as a tracer to determine streambed water exchanges. Water Resources Research 44, 1-20.

Cooper, H.H., Rorabaugh, M.I., 1963. Ground-water movements and bank storage due to flood stages in surface streams. U.S. Geological Survey Water Supply Paper 1536-J 1963, 343-366.

Covert, J.J., Band, T.L., Gregory, G.J., 2005. Data Report, 2005 “Six minute” Study, Spokane Valley Rathdrum Prairie Aquifer Area; WA. Dept. of Ecology.

Cunge, J.A., 1969. On The Subject Of A Flood Propagation Computation Method (Muskingum Method). Journal of Hydraulic Research 7 (2), 205-230.

Desilets, S.L., Ferre, T.P., Troch, P.A., 2008. Effects of stream-aquifer disconnection on local flow patterns. Water Resources Research 44, 1-6.

Dever, R.J., Cleary, R.W., 1979. Unsteady-State, Two-Dimensional Response of Leaky Aquifers to Stream Stage Fluctuations. Advances in Water Resources 2, 13-18.

Domenico, P.A., Schwartz, F.W., 1990. Physical and chemical hydrogeology. Chp 3:61-62.

Doppler, T., Franssen, H.H., Kaiser, H., Kuhlman, U., Stauffer, F., 2007. Field evidence of dynamic leakage coefficient for modelling river-aquifer interactions. Journal of Hydrology 347, 177-187.

Federal Register. Vol 43 (28). Doc 78-3475, Filed 2-8-78

Fetter C.W., 2001. Applied Hydrogeology 4th Edition. Chp. 3:66-112.

Fleckenstein, J.H., Niswonger, R.G., Fogg, G.E., 2006. River-Aquifer Interactions, Geologic Heterogeneity, and Low-Flow Management. Ground Water 44 (6), 837-852.

Fox, G.A., 2006. Estimating streambed conductivity: guidelines for stream-aquifer analysis tests. American Society of Agricultural and Biological En 50 (1), 107-113.

Fox, G.A., DuChateau, P., Durnford, D.S., 2002. Analytical model for aquifer response incorporating distributed stream leakage. Ground Water 40 (4), 378-384.

Fox, G.A., Heeren, D.M., Kizer, M.A., 2011. Evaluation of a stream-aquifer analysis test for deriving reach-scale streambed conductance. American Society of Agricultural and Biological En 54 (2), 473-479.

Garcia, L.D. 2010. Estimating groundwater recharge via surface infiltration in cold semi-arid regions. Master of Science, Civil and Environmental Engineering, Washington State University, Pullman, WA.

Glover, R.E., 1952. Methods of estimating possible depletion of flows in the Smoky Hill and North Solomon rivers in Kansas resulting from well pumping. Bur. Reclam. Tech. Memo. 657, Sect. I, 88-97.

Glover, R.E., 1964. Ground-water movement. U.S. Bureau of Reclamation Engineering Monograph Series, no. 31. Washington, DC: U.S. Bureau of Reclamation.

Golder Associates, Inc., 2004, Final report to the Little and Middle Spokane watershed WRIA 55 and 57 planning unit, level 2 technical assessment—Watershed simulation model: Seattle, Golder Associates, Inc., prepared under grant no. 9800300 from the Washington Department of Ecology, February 14, 2004, 51 p., 4 appendixes.

Guo, W., 1996. Transient groundwater flow between reservoirs and water-table aquifers. Journal of Hydrology 195, 370-384.

Haitjema, H.M., Kelson, V.A., de Lange, W., 2001. Selecting MODFLOW Cell Sizes for Accurate Flow Fields. Ground Water 39 (6), 931-938.

Hall, F.R., Moench, A.F., 1972. Application of the Convolution Equation to Stream-Aquifer Relationships. Water Resources Research 8 (2), 487-493.

Hantush, M.M., 2005. Modeling stream-aquifer interactions with linear response functions. Journal of Hydrology 311, 59-79.

Hantush, M.M., Harada, M., Marino, M.A., 2002. Hydraulics of Stream Flow Routing with Bank Storage. Journal of Hydrologic Engineering 7 (1), 76-89.

Hantush, M.S., 1959. Analysis of data from pumping wells near a river. Journal of Geophysical Research 64 (11), 1921-1932.

Hantush, M.S., 1965. Wells near Streams with Semipervious Beds. Journal of Geophysical Research 70 (12), 2829-2838.

Hantush, M.S., 1967. Growth and Decay of Groundwater-Mounds in Response to Uniform Percolation. Water Resource Research 3 (1), 227-234.

Harada, M., Hantush, M.M., Marino, M.A., 1999. Hydraulic Analysis on Stream-Aquifer Interaction by Storage Function Models. Proceedings of JSCE - (628), 189-194.

Harbaugh, A.W., Banta, E.R., Hill, M.C., McDonald, M.G., 2000. MODFLOW-2000, the U.S. Geological Survey modular ground-water model-User guide to modularization concepts and the Ground-Water Flow Process: U.S. Geological Survey Open-File Report 00-92, 121 p.

Hossain, A., 2011. Civil and Environmental Engineering - Washington State University. Personal Communication, May.

Hseih, P.A., M.E. Barber, B.A. Contor, M.A. Hossain, G.S. Johnson, J.L. Jones, and A.H. Wylie, 2007. Ground-Water flow model for the Spokane Valley-Rathdrum Prairie aquifer, Spokane County, Washington, and Bonner and Kootenai Counties, Idaho," USGS Scientific Investigations Report 2007-5044, Reston, VA.

Hunt, B., 1999. Unsteady stream depletion from ground water pumping. Ground Water 37 (1), 98-102.

Hunt, R.J., Anderson, M.P., Kelson, V.A., 1998. Improving a Complex Finite-Difference Ground Water Flow Model Through the Use of an Analytic Element Screening Model. Ground Water 36 (6), 1011-1017.

Ippisch, O., Vogel, H.-J., Bastian, P., 2006. Validity limits for the van Genuchten-Mualem model and implications for parameter estimation and numerical simulation. Advances in Water Resources 29, 1780-1789.

Jackson, C., 2005. Modelling leakage from perched rivers using the unsaturated flow model VS2DTI. British Geological Survey IR/05/019, 1-38.

Kabala, Z.J., Thorne, B., 1997. Hydraulics of one- and two-dimensional flow fields in aquifers. Hydrological Sciences 42 (1), 1-14.

Kahle, S.C., and Bartolino, J.R., 2007, Hydrogeologic framework and ground-water budget of the Spokane Valley-Rathdrum Prairie aquifer, Spokane County, Washington, and Bonner and Kootenai Counties, Idaho: U.S. Geological Survey Scientific Investigations Report 2007-5041, 48 p., 2 pls., accessed May 6, 2007 at http://pubs.water.usgs.gov/sir20075041.

Kennedy, C.D., Genereux, D.P., Mitasova, H., Corbett, D.R., Leahy, S., 2008. Effect of sampling density and design on estimation of streambed attributes. Journal of Hydrology 355, 164-180.

Konikow, L.F., Bredehoeft, J.D., 1992. Ground-water models cannot be validated. Advances in Water Resources 15, 75-83.

Konrad, C.P., 2006. Longitudinal hydraulic analysis of river-aquifer exchanges. Water Resources Research 42, 1-14.

Koussis, A.D., Akylas, E., Mazi, K., 2007. Response of sloping unconfined aquifer to stage changes in adjacent stream. II. Applications. Journal of Hydrology 338, 73-84.

Kumar, C.P., 2006. Groundwater flow models. National Institute of Hydrology, 1-20.

Lal, A.M.W., 2001. Modification of canal flow due to stream-aquifer interaction. Journal of Hydraulic Engineering, 127(7), 567-576.

Larkin, R.G., Sharp, J.M. Jr., 1992. On the relationship between river-basin geomorphology, aquifer hydraulics, and ground-water flow direction in alluvial aquifers. Geol Soc Am Bull 104, pp. 1608-1620.

Marino, M.A., 1975. Artificial Groundwater Recharge, II. Rectangular Recharging Area. Journal of Hydrology 26, 29-37.

Mays, L.W., 2005. Water resources engineering – 2005 ed. John Wiley & Sins.

McCuen, R.H., Knight, Z., Cutter, A.G., 2006. Evaluation of the Nash-Sutcliffe Efficiency Index. Journal of Hydrologic Engineering ASCE 11 (6), 597-602.

McDonald, M.G., Harbaugh, A.W., 1988. A modular three-dimensional finite-difference ground-water flow model: U.S. Geological Survey Techniques of Water-Resources Investigations, Book 6, Chap. A1, 586 p.

Mehl, S., Hill, M.C., 2010. Grid-size dependence of Cauchy boundary conditions used to simulate stream-aquifer interactions. Advances in Water Resources 33, 430-442.

Moench, A.F., Barlow, P.M., 2000. Aquifer response to stream-stage and recharge variations. I. Analytical step-reponse functions. Journal of Hydrology 230, 192-210.

Moench, A.F., Kisiel, C.C., 1970. Application of the convolution relation to estimating recharge from an ephemeral stream. Water Resources Research 6 (4), 1087-1094.

Moench, A.F., Sauer, V.B., Jennings, M.E., 1974. Modification of Routed Streamflow by Channel Loss and Base Flow. Water Resources Research 10 (5), 963-968.

Nemeth, M.S., Solo-Gabriele, H.M., 2002. Evaluation of the reach transmissivity to quantify exchange between groundwater and surface water. Journal of Hydrology 274, 145-159.

Neuman, S.P., 1972. Theory of flow in unconfined aquifers considering delayed response of the water table. Water Resources Research 8 (4), 1031-1045.

Neuman, S.P., 1974. Effect of partial penetration on flow in unconfined aquifers considering delayed gravity response. Water Resources Research 10 (2), 303-312.

Neuman, S.P., 1981. Delayed drainage in a stream-aquifer system. American Society of Civil Engineers 107 (IR4), 407-410.

NWIS Site Information for USA: Site Inventory. USGS 12419000 Spokane River NR Post Falls ID, retrieved: 2010-08-26. http://waterdata.usgs.gov/nwis/inventory?agency_code=USGS&site_no=12419000.

Osman, Y.Z., Bruen, M.P., 2002. Modelling stream-aquifer seepage in an alluvial aquifer: an improved loosing-stream package for MODFLOW. Journal of Hydrology 264, 69-86.

Patrick, M., 2011. Avista Cooperation. Personal Communication, May.

Rao, N.H., Sarma, P.B., 1981. Ground-Water Recharge from Rectangular Areas. Ground Water 19 (3), 271-274.

Rosenberry, D.O., Pitlick, J., 2009. Effects of sediment transport and seepage direction on hydraulic properties at the sediment-water interface of hyporheic settings. Journal of Hydrology 373, 377-391.

Rushton, K.R., 2007. Representation in regional models of saturated river-aquifer interaction for gaining/losing rivers. Journal of Hydrology 334 (12), 262-281.

Rushton, K.R., Tomlinson, L.M., 1979. Possible mechanisms for leakage between aquifers and rivers. Journal of Hydrology 40, 49-65.

Schaefli, B., Gupta, H.V., 2007. Do Nash values have value?. Hydrological Processes 21, 2075-2080.

Schaller, M.F., Fan, Y., 2009. River basins as groundwater exporters and importers: Implications for water cycle and climate modeling. Journal of Geophysical Research 114, 1-21.

Schlumberger Water Services., 2009. Visual MODFLOW Premium User's Manual 2009.1 River Package. Chp 4, pp. 245-248.

Schwarzenbach, R.P., Westall, J., 1981. Transport of Nonpolar Organic Compounds from Surface Water to Groundwater. Laboratory Sorption Studies. Environmental Science & Technology 15 (11), 1360-1367.

Selker, J.S., Keller, C.K., McCord, J.T., 1999. Vadose zone processes. CRC Press, New York.

Serrano, S.E., Workman, S.R., Srivastava, K., Cleave, B.M., 2007. Models of nonlinear stream aquifer transients. Journal of Hydrology 336, 199-205.

Sharp, J.M., 1977. Limitations of Bank-Storage Model Assumptions. Journal of Hydrology 35, 31-47.

Shawaqfah, Mo'ayyad., 2002. Numerical investigation of the role of muck layer in contaminant migration at stream/aquifer interface. Diss. Washington State University, Pullman.

Singh, S.K., Mishra, G.C., Swamee, P.K., Ojha, C.S., 2002. Aquifer Diffusivity and Stream Resistance from Varying Stream Stage. Journal of Irrigation and Drainage Engineering 128 (1), 57-61.

Šimůnek, J., M. Šenja and M. Th. Van Genuchten. 2007. Software package for simulating the two- and three-dimensional movement of water, heat, and multiple solutes in variably-saturated media. Users Manual, Version 1. http://igwmc.mines.edu/software/igwmcsoft/HYDRUS3D_User_Manual.pdf

Solinst Canada Ltd, 2001. Solinst Model 3001 Levelogger Junior. www.solinst.com.

Sophocleous, M., 2002. Interactions between groundwater and surface water: the state of the science. Hydrogeology Journal 10 (1), 52-67.

Sophocleous, M., Koussis, A.D., Martin, J.L., Perkins, S.P., 1995. Evaluation of Simplified Stream-Aquifer Depletion Models for Water Rights Administration. Ground Water 33 (4), 579-588.

Spalding, C.P., Khaleel, R., 1991. An Evaluation of Analytical Solutions to Estimate Drawdown's and Stream Depletions by Wells. Water Resources Research 27 (4), 597-609.

Tang, G., Alshawabkeh, A.N., 2006. A semi-analytical time integration for numerical solution of Boussinesq equation. Advances in Water Resources 29, 1953-1968.

The PRISM Group at Oregon State University, 2006. Contours of Idaho Average Annual Precipitation 1971-2000. Idaho Department of Water Resources.

Theis, C.V., 1940. The source of water derived from wells: essential factors controlling the response of an aquifer to development. Civil Engineer 10, 277-280.

Toth, J., 1963. A Theoretical Analysis of Groundwater Flow in Small Drainage Basins. Journal of Geophysical Research 68 (16), 4795-4812.

Toth, J., 1970. A conceptual model of the groundwater regime and the hydrogeologic environment. Journal of Hydrology 10, 164-176.

Walton, W.C., 1979. Progress in Analytical Groundwater Modeling. Journal of Hydrology 43, 149-159.

Wang, H.F., Anderson, M.P., 1982. Introduction to groundwater modeling. W. H. Freeman and Company, New York.

Ward, J.V., Stanford, J.A., 1995. The Serial Discontinuity Concept: Extending the Model to Floodplain Rivers. Regulated Rivers: Research & Management 10, 159-168.

Washington State Department of Ecology, 2006. Washington State Water Law: A Primer. Publication # WR 98-152, Revised July 2006, 1-7.

Washington State Department of Ecology, 2004. Stormwater management manual for eastern Washington. Publication # 04-10-076, 1-715.

Winter, T.C., 1999. Relation of streams, lakes, wetlands to groundwater flow systems. Hydrogeology Journal 7, 28-45.

Winter, T.C., Harvey, J.W., Franke, O.L., Alley, W.M., 1998. Ground Water and Surface Water A Single Resource. U.S. Geological Survey Circular 1139 , 1-79.

Woessner, W.W., 2000. Stream and Fluvial Plain Ground Water Interactions: Rescaling Hydrogeologic Thought. Ground Water 38 (3), 423-429.

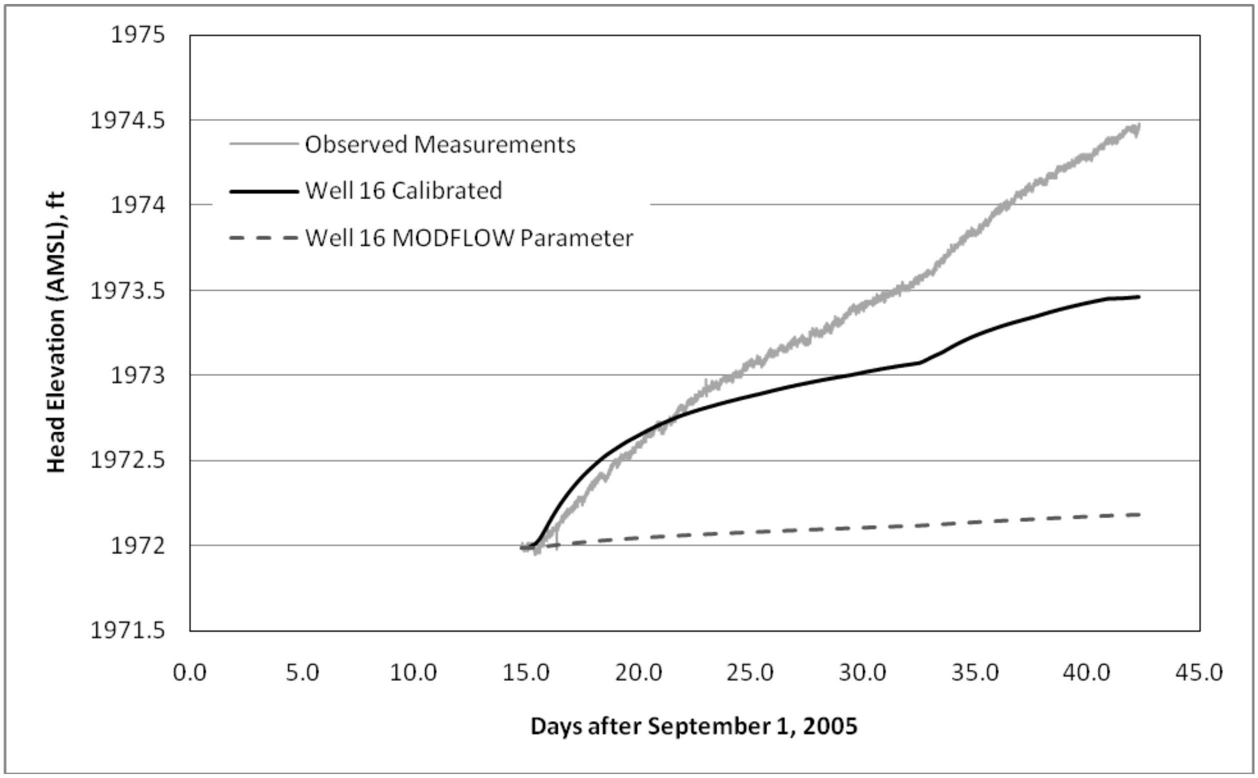
Yeh, H., Chang, Y., Zlotnik, V.A., 2008. Stream depletion rate and volume from groundwater pumping in wedge-shape aquifer. Journal of Hydrology 349, 501-511.

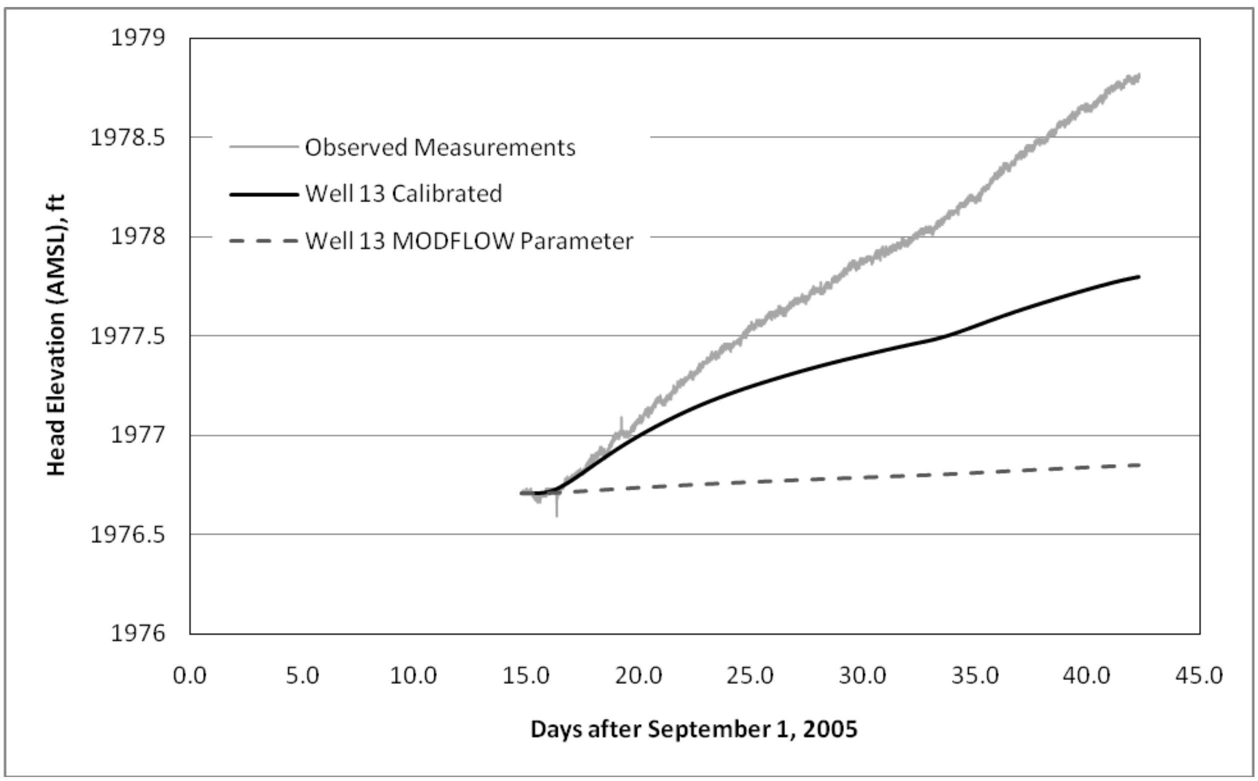
Zlotnik, V.A., Huang, H., 1999. Effect of Shallow Penetration and Streambed Sediments on Aquifer Response to Stream Stage Fluctuation (Analytical Model). Ground Water 37 (4), 599-605.

Appendix

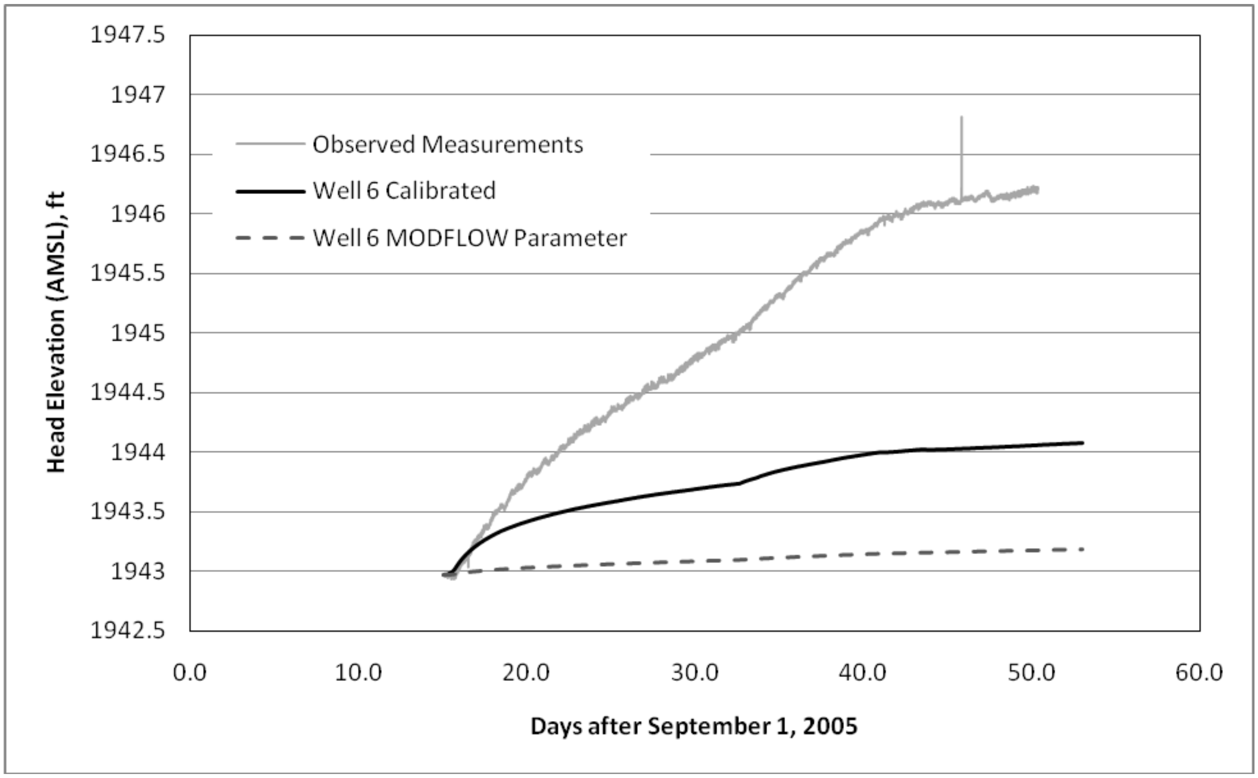
Additional Graphs for Zlotnik and Huang (1999) A2 Scenario

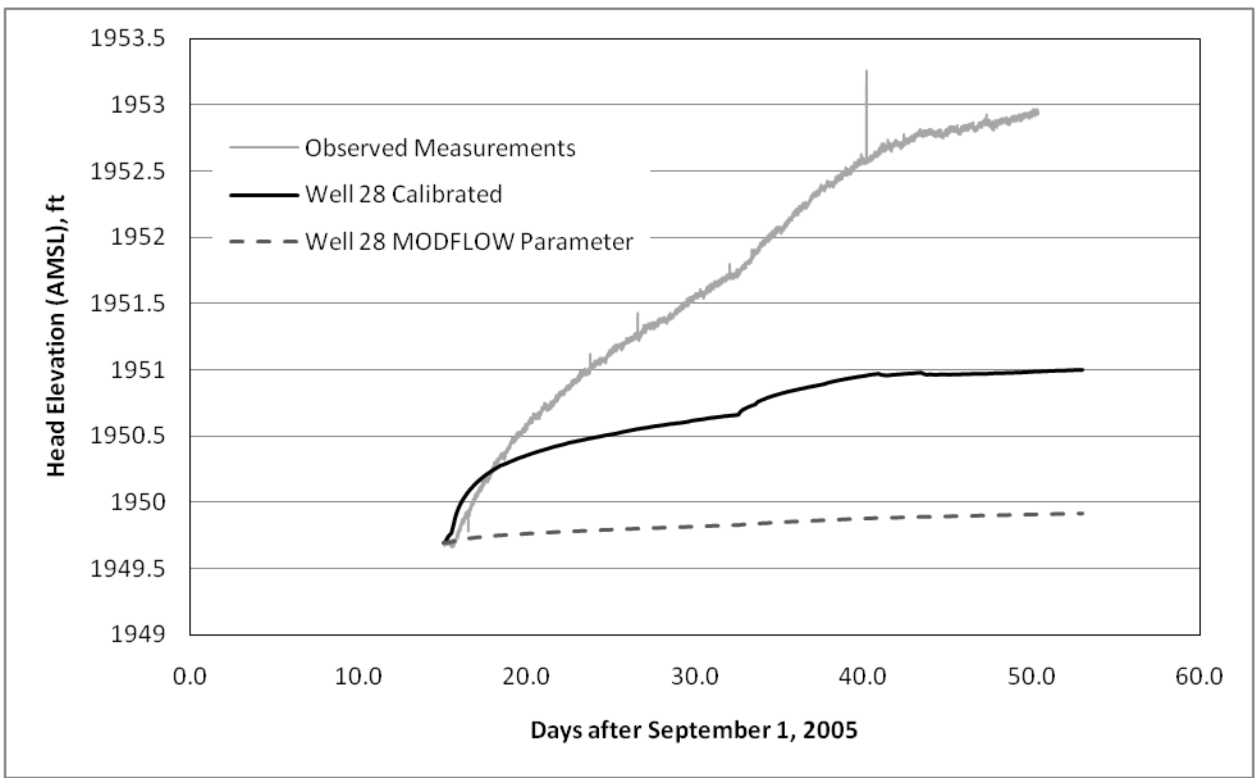
Area 1 A2 Additional Graphs

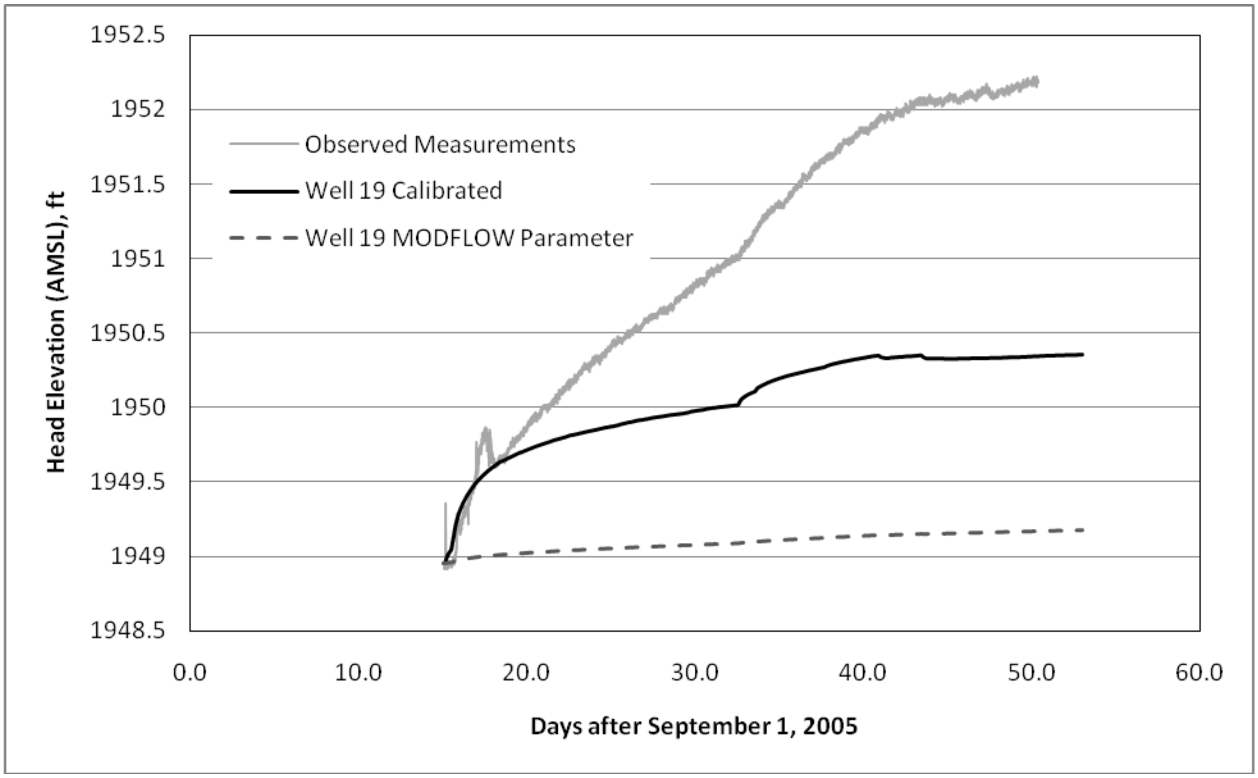


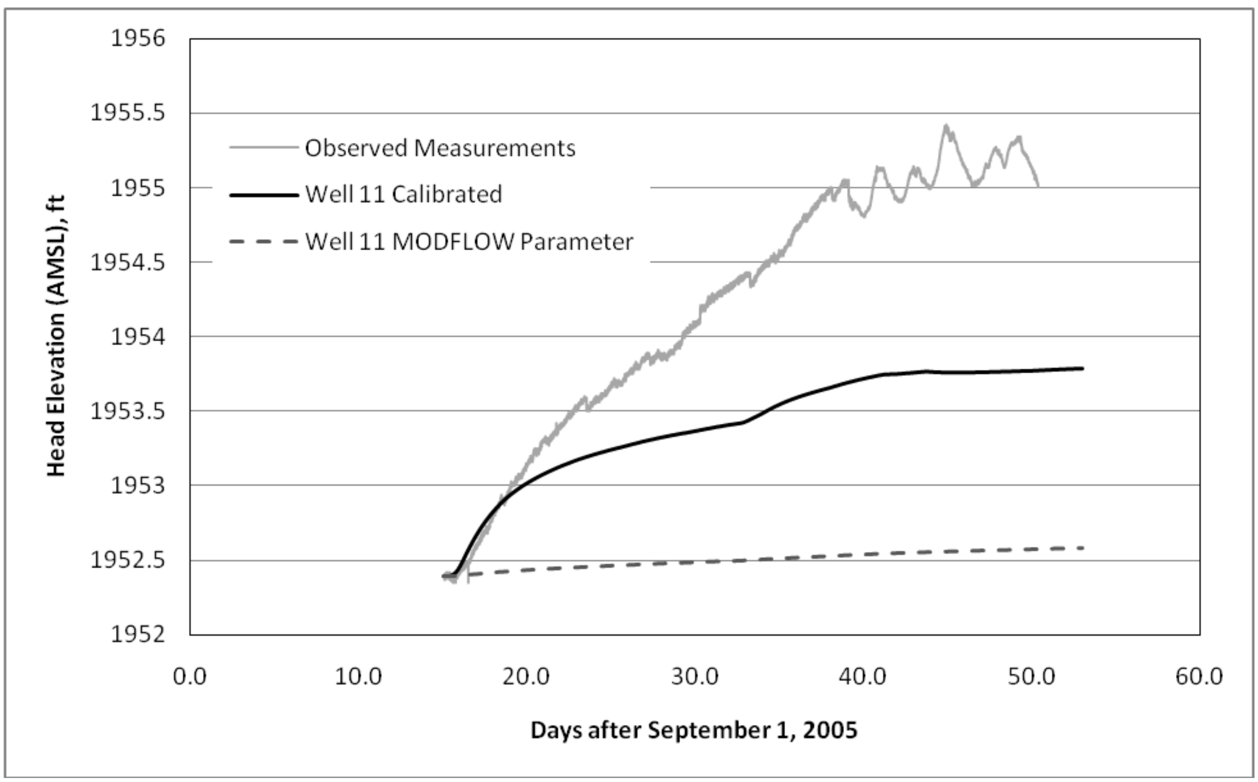


Area 2 A2 Additional Graphs

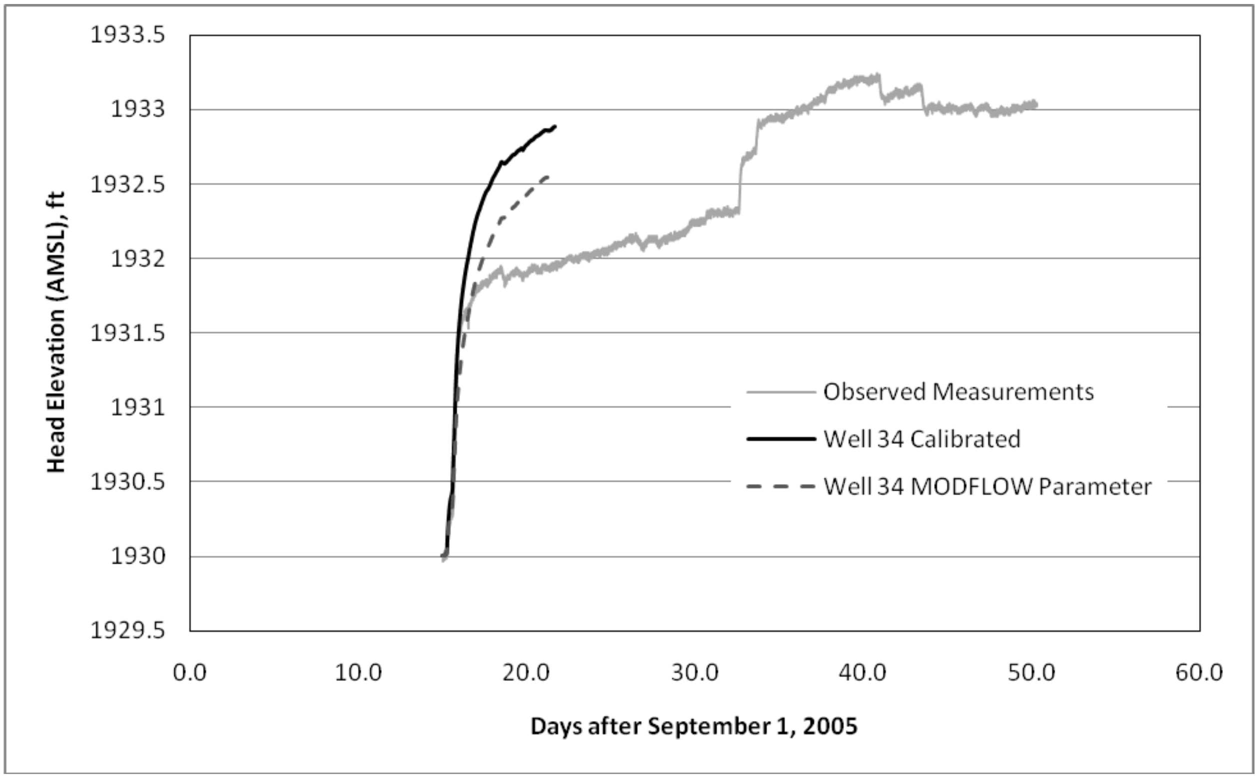

1947.5
1947
1946.5
1946
1945.5
1945
1944.5
1944
1943.5
1943
1942.5
Head Elevation (AMSL), ft
Observed Measurements
Well 6 Calibrated
Well 6 MODFLOW Parameter
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005


1953.5
1953
1952.5
1952
1951.5
1951
1950.5
1950
1949.5
1949
Head Elevation (AMSL), ft
Observed Measurements
Well 28 Calibrated
Well 28 MODFLOW Parameter
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

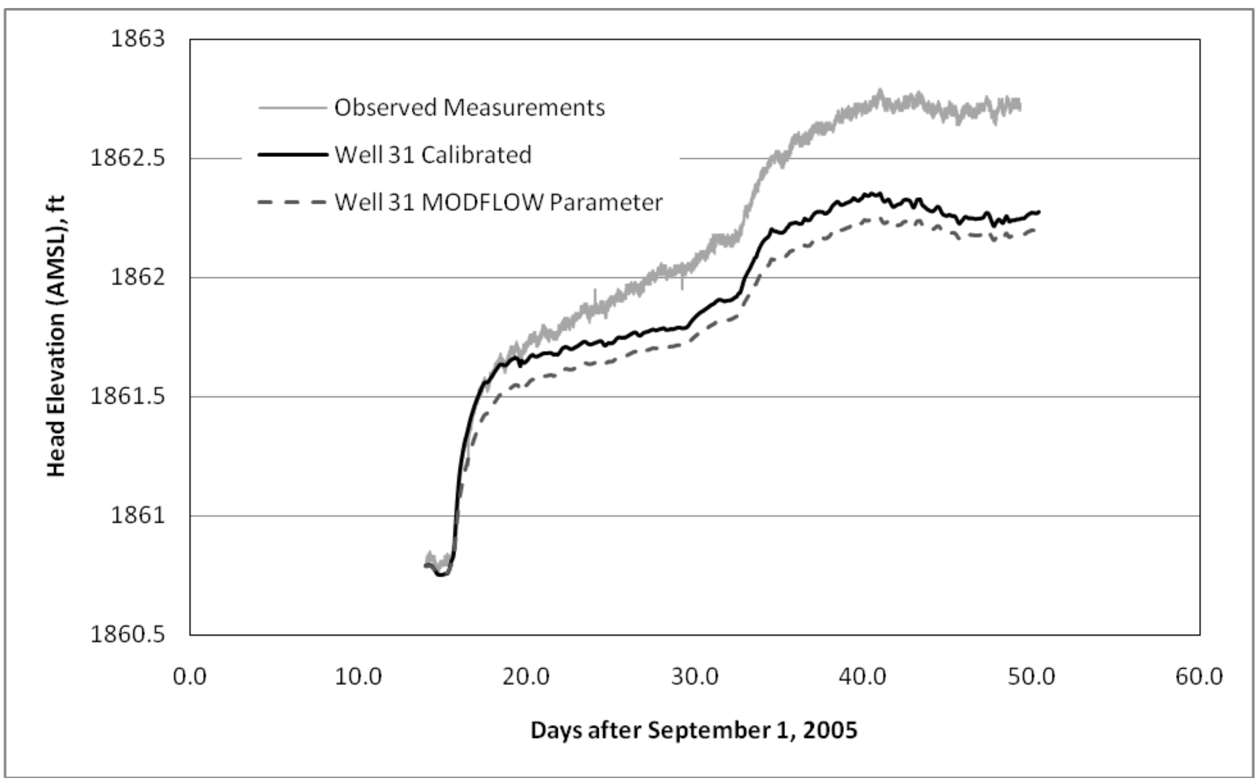


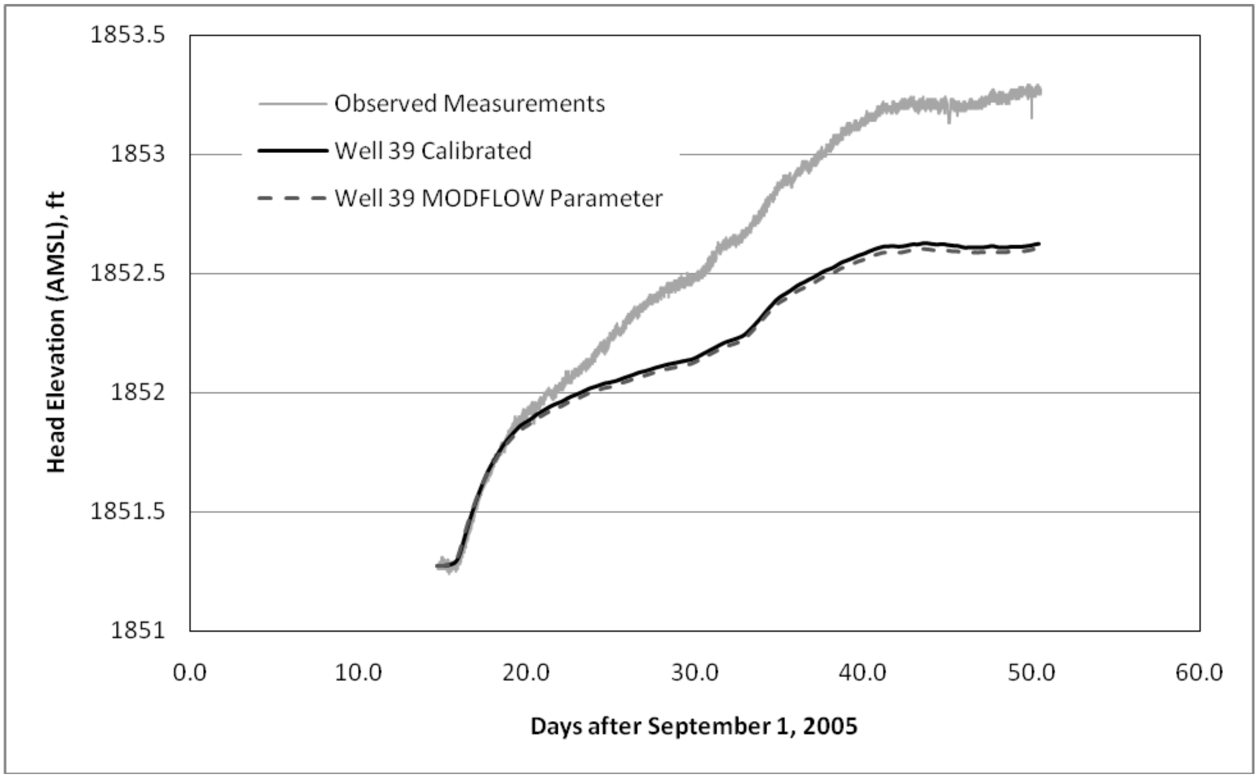


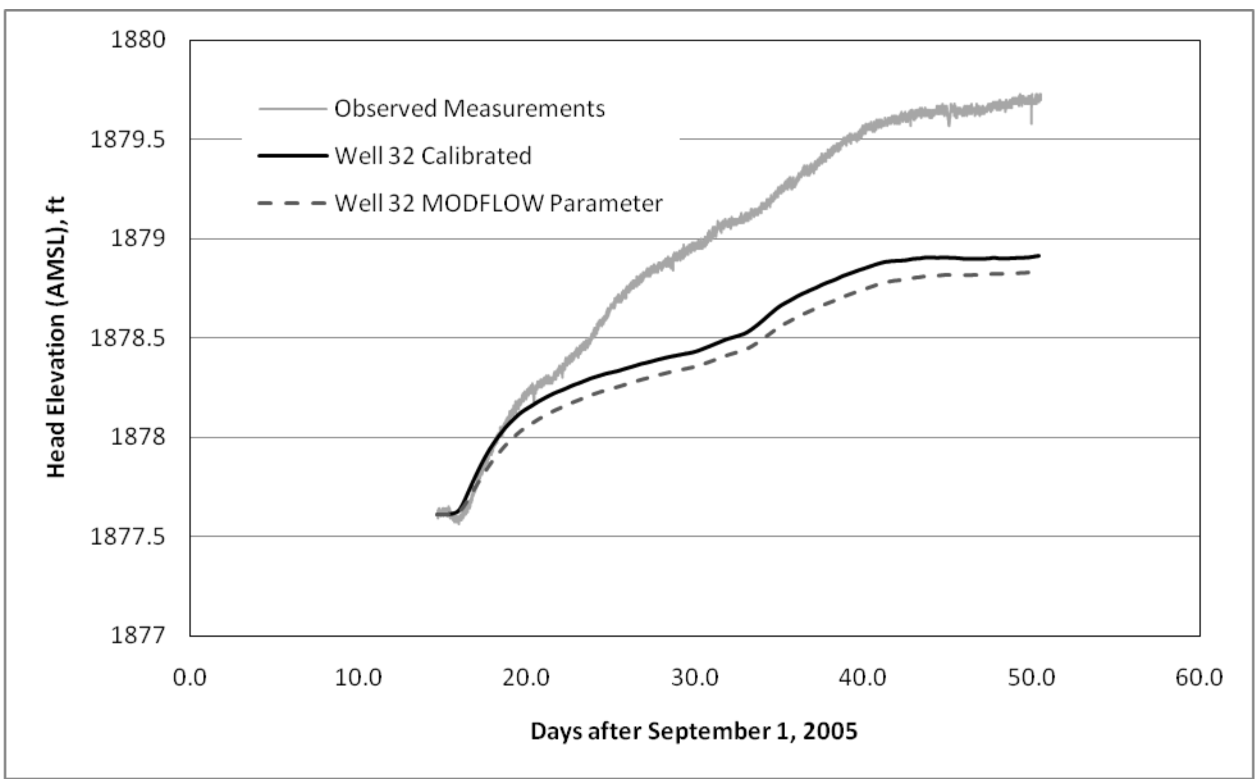
Area 3 A2 Additional Graphs



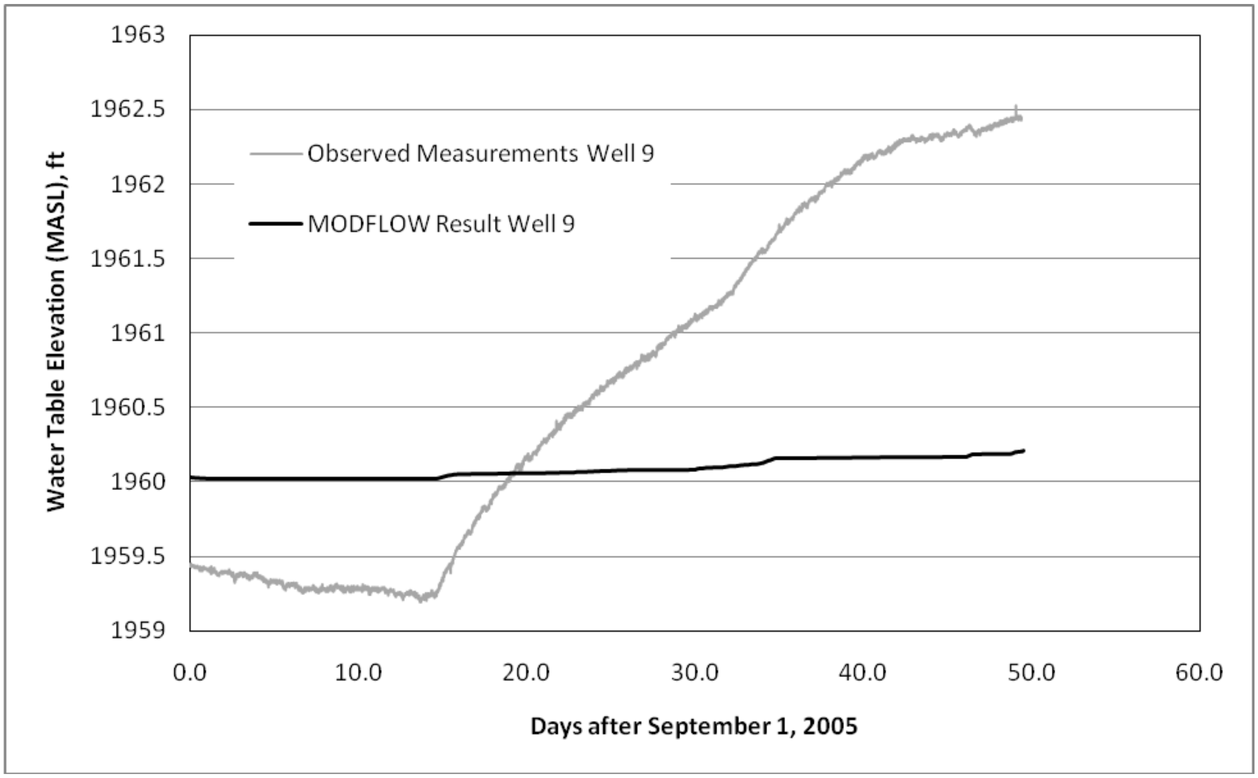
Area 4 A2 Additional Graphs

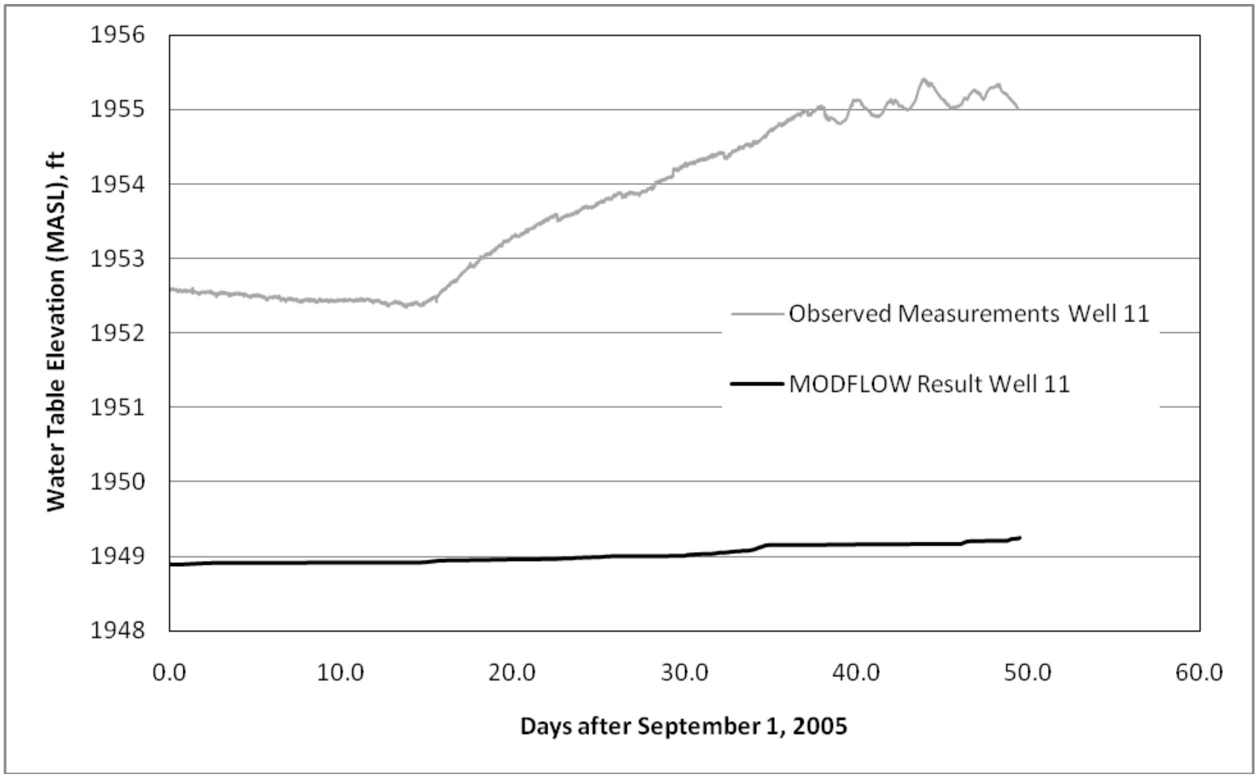


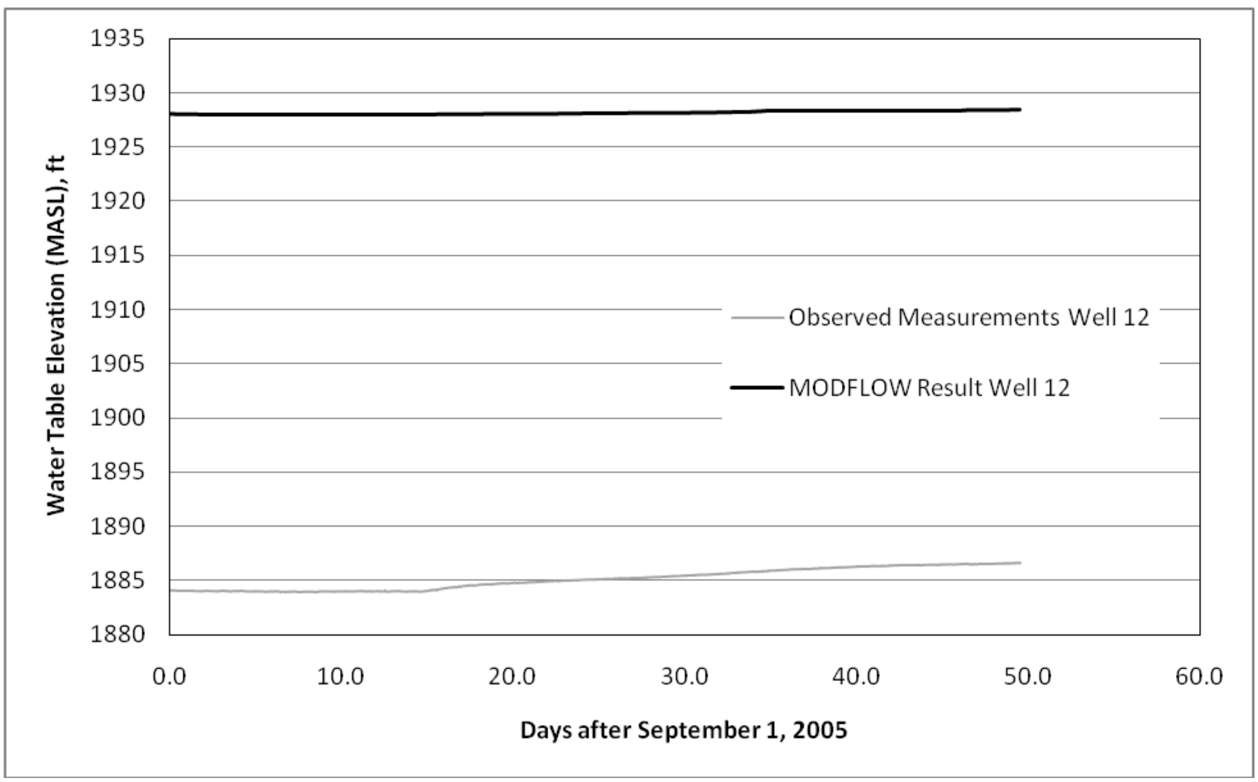

1853.5
1853
1852.5
1852
1851.5
1851
Head Elevation (AMSL), ft
Observed Measurements
Well 39 Calibrated
Well 39 MODFLOW Parameter
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

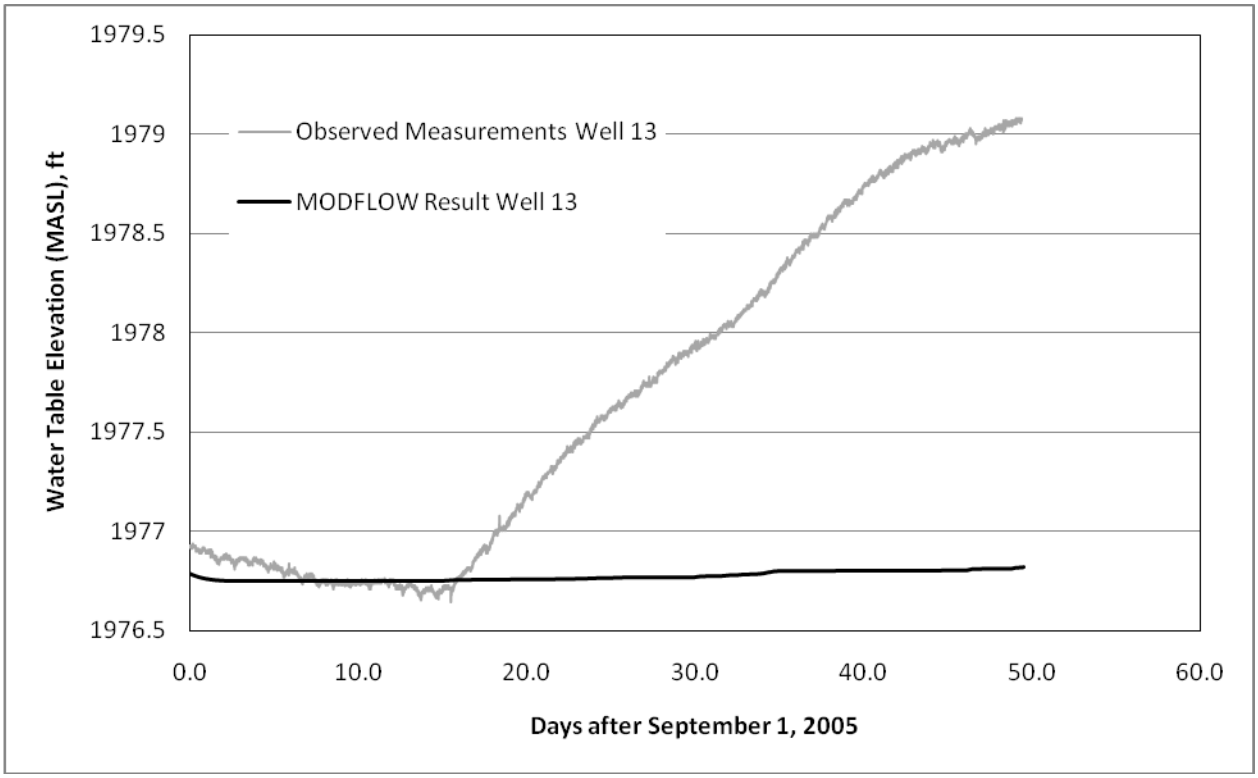

1880
1879.5
1879
1878.5
1878
1877.5
1877
Head Elevation (AMSL), ft
Observed Measurements
Well 32 Calibrated
Well 32 MODFLOW Parameter
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

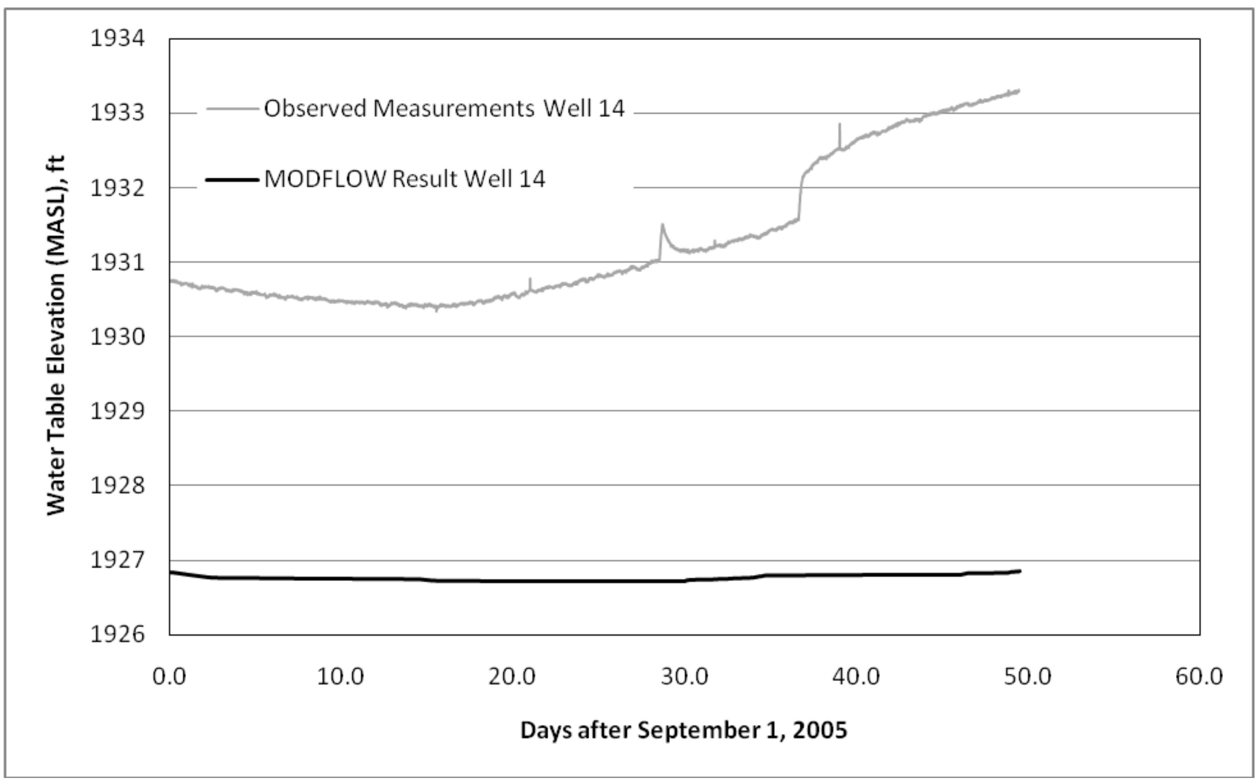
Additional Graphs MODFLOW Scenario

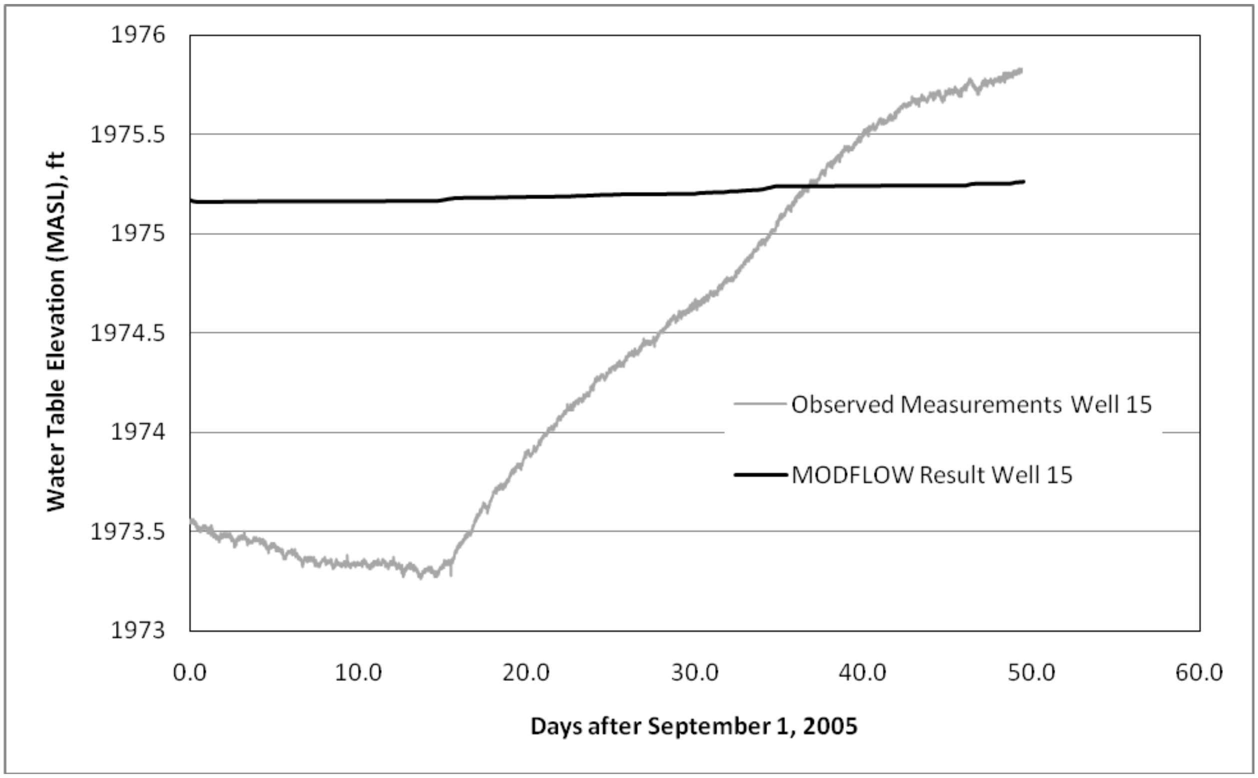


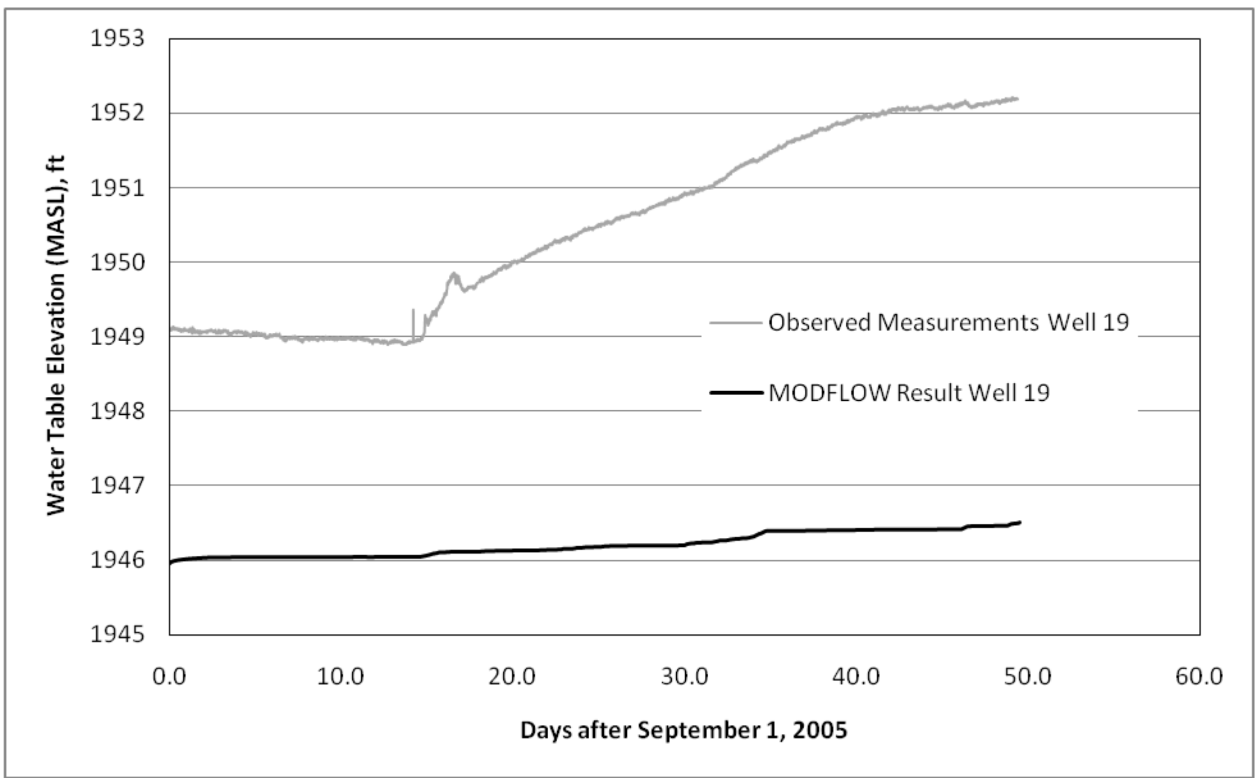

1956
1955
1954
1953
1952
1951
1950
1949
1948
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 11
MODFLOW Result Well 11

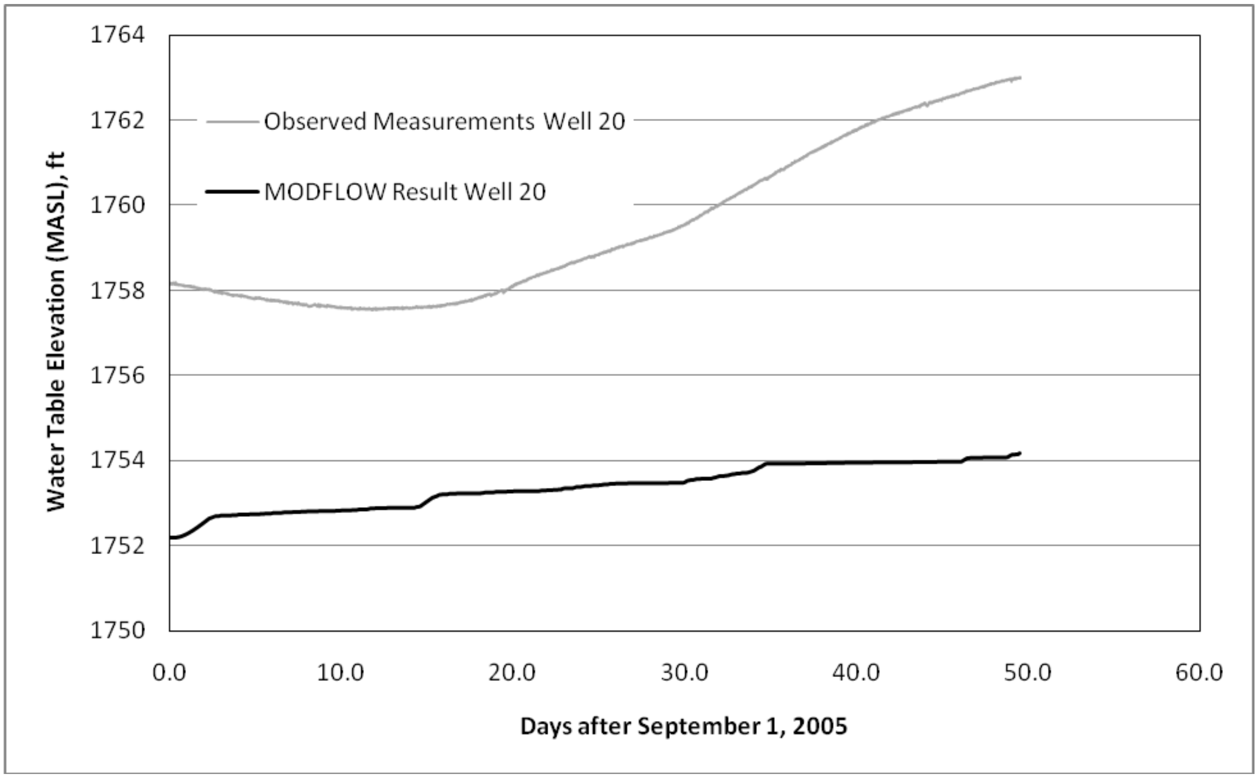

1935
1930
1925
1920
1915
1910
1905
1900
1895
1890
1885
1880
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 12
MODFLOW Result Well 12


1979.5
1979
1978.5
1978
1977.5
1977
1976.5
Water Table Elevation (MASL), ft
Observed Measurements Well 13
MODFLOW Result Well 13
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

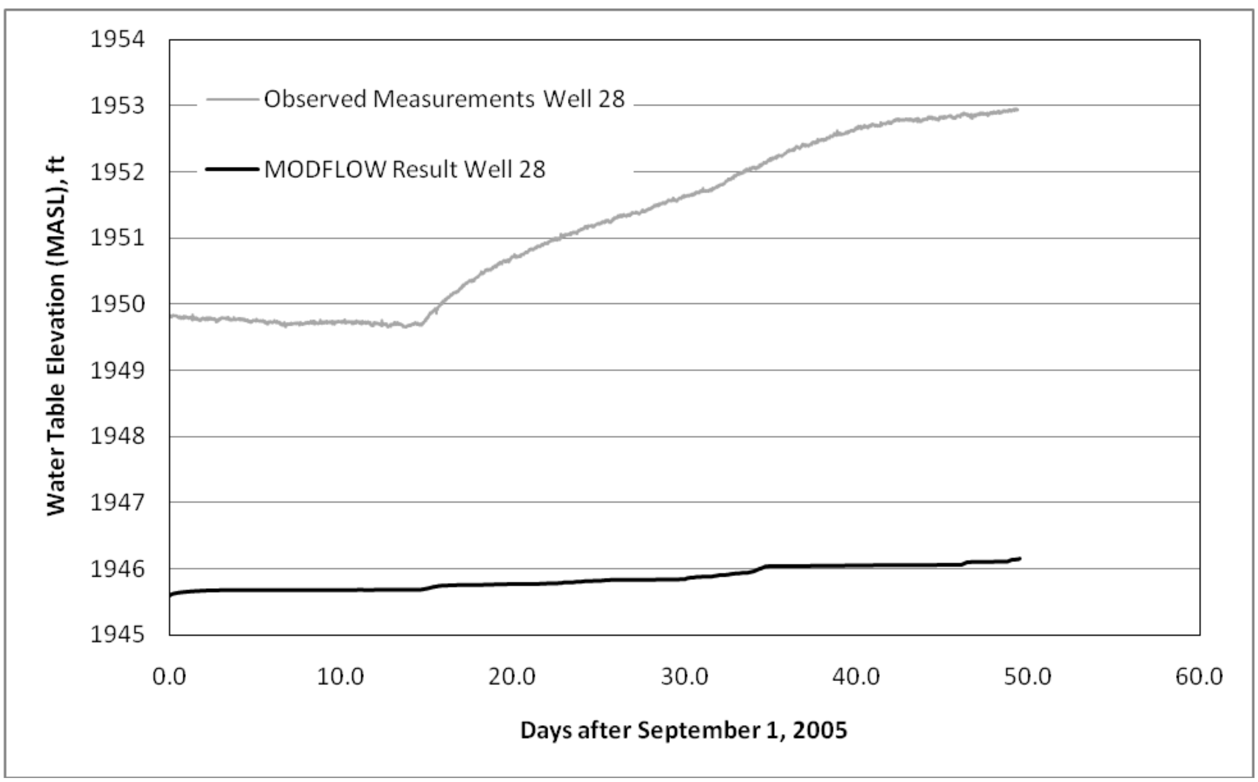

1934
1933
1932
1931
1930
1929
1928
1927
1926
Water Table Elevation (MASL), ft
Observed Measurements Well 14
MODFLOW Result Well 14
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005


1976
1975.5
1975
1974.5
1974
1973.5
1973
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Water Table Elevation (MASL), ft
Days after September 1, 2005
Observed Measurements Well 15
MODFLOW Result Well 15

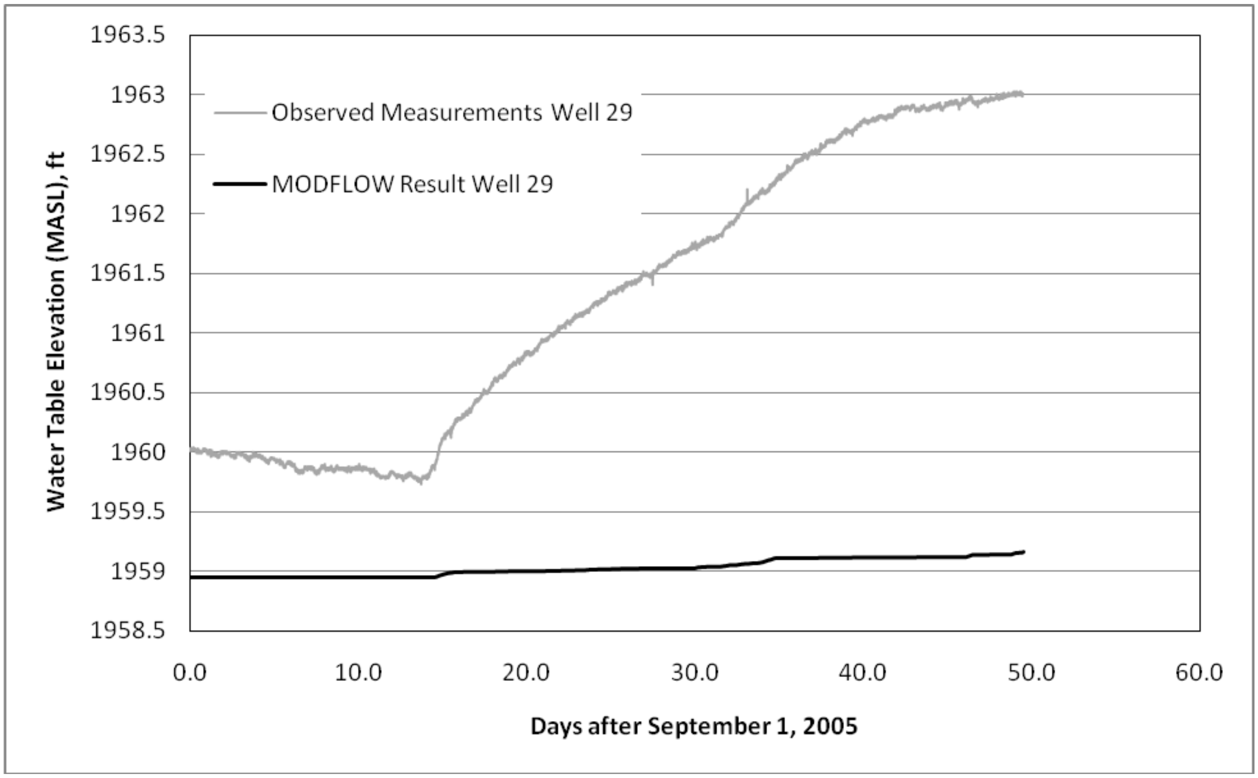

1953
1952
1951
1950
1949
1948
1947
1946
1945
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Water Table Elevation (MASL), ft
Days after September 1, 2005
Observed Measurements Well 19
MODFLOW Result Well 19

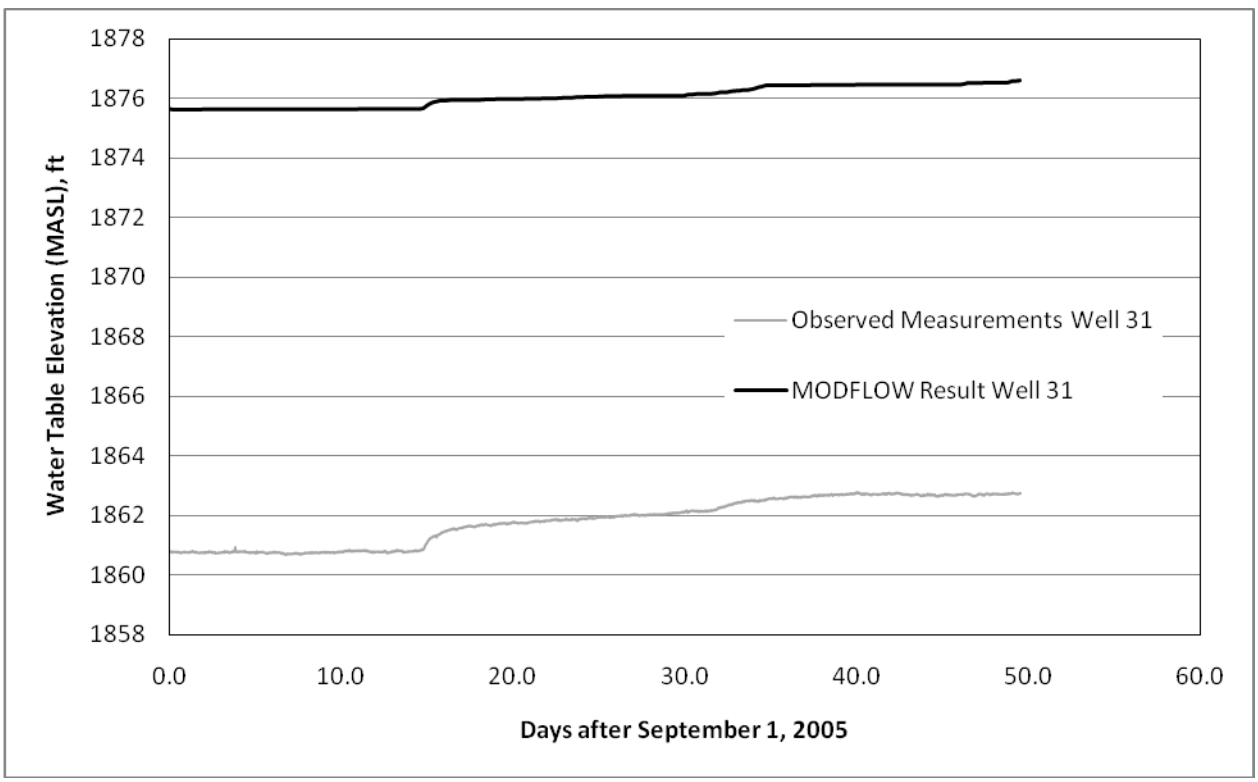


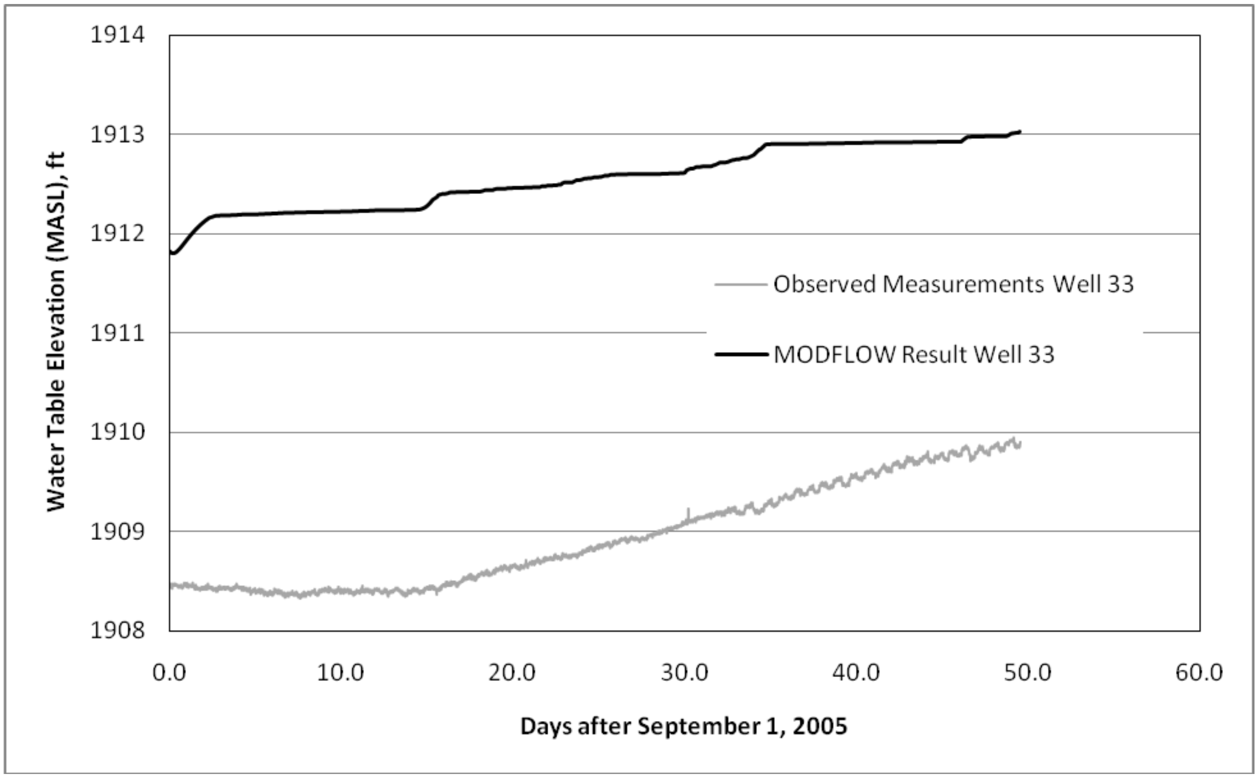
1764
1762
1760
1758
1756
1754
1752
1750
Observed Measurements Well 20
MODFLOW Result Well 20
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

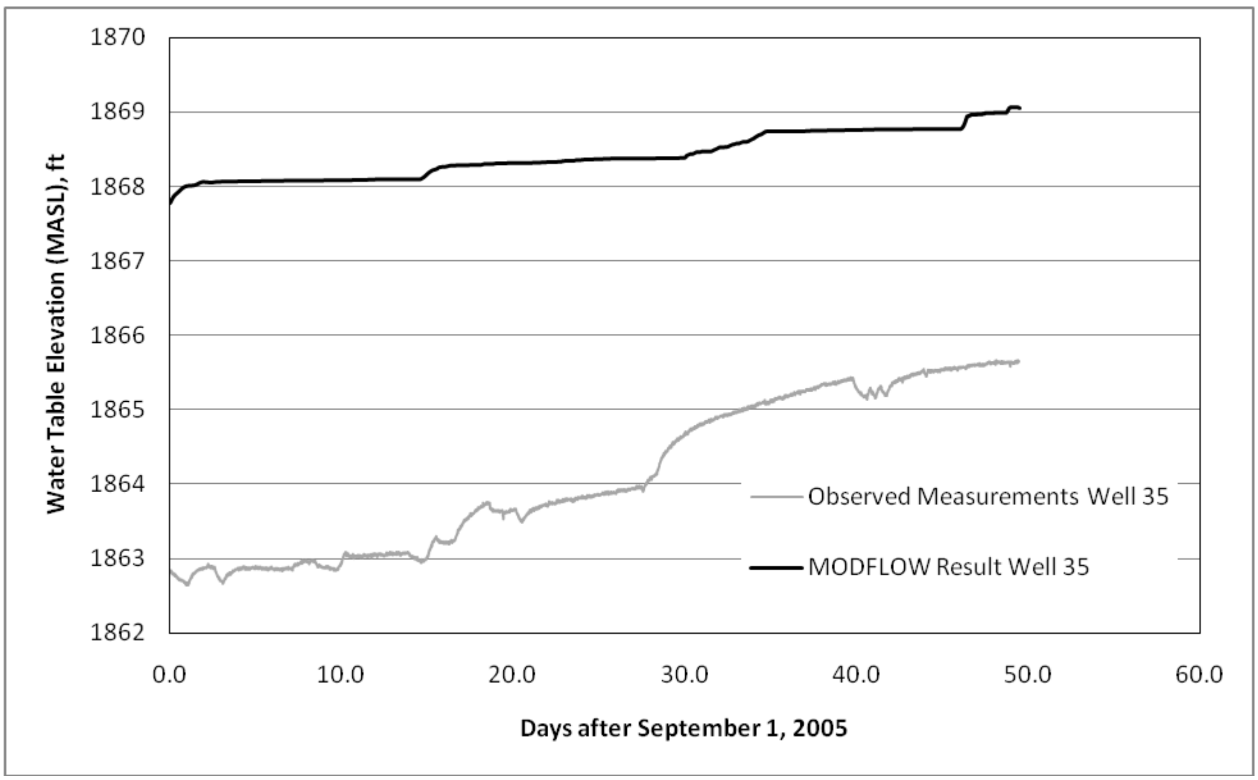


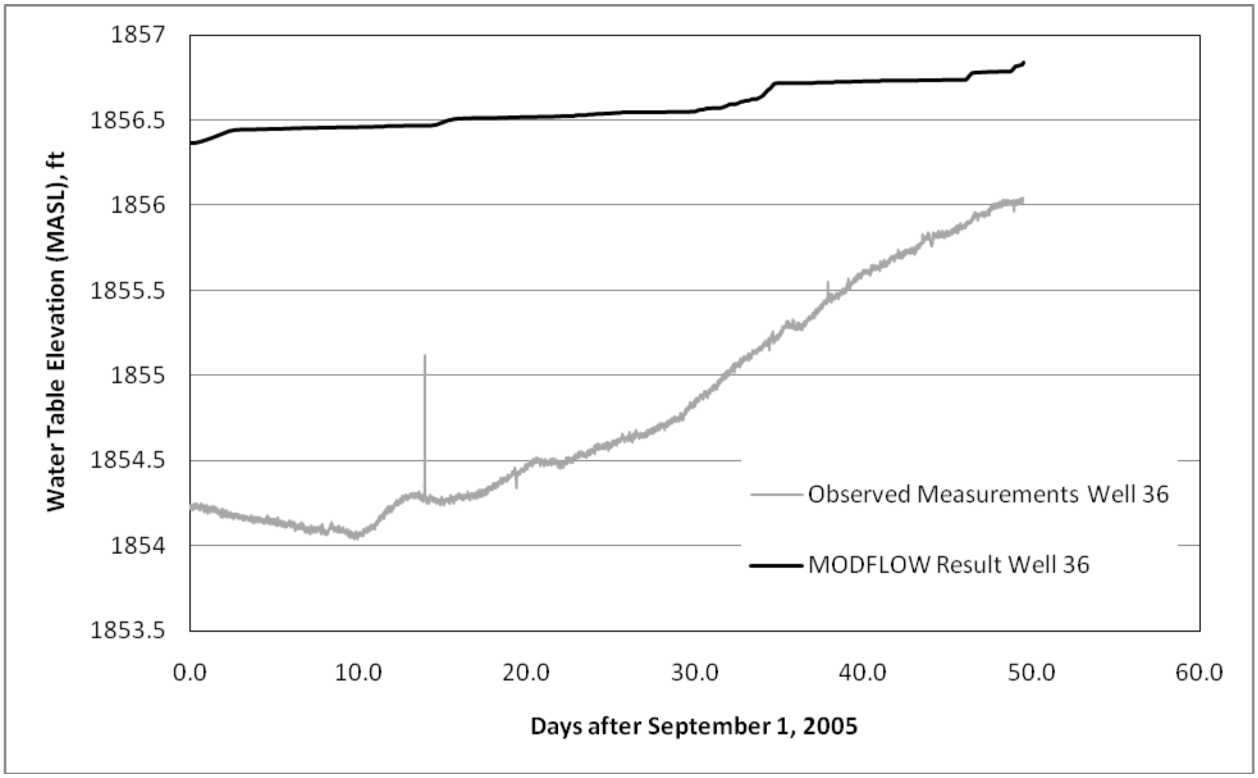
1954
1953
1952
1951
1950
1949
1948
1947
1946
1945
Observed Measurements Well 28
MODFLOW Result Well 28
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

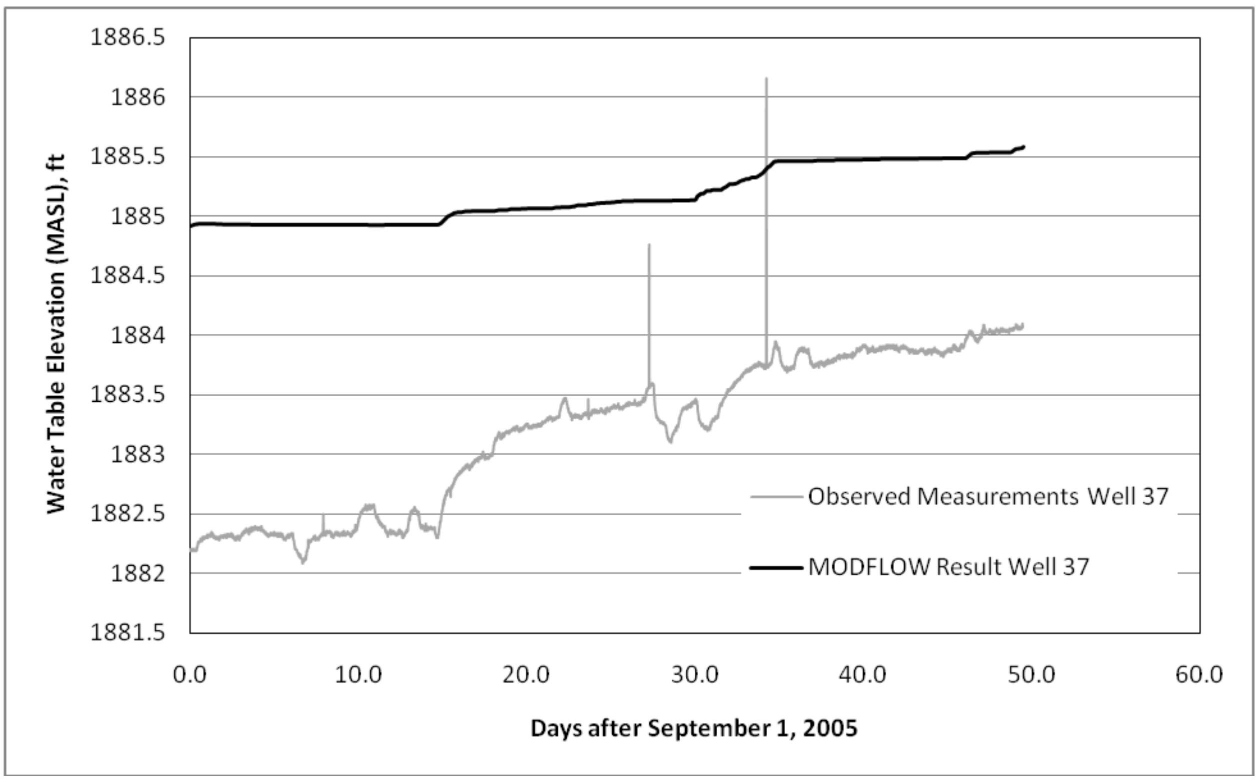

1963.5
1963
1962.5
1962
1961.5
1961
1960.5
1960
1959.5
1959
1958.5
Water Table Elevation (MASL), ft
Observed Measurements Well 29
MODFLOW Result Well 29
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

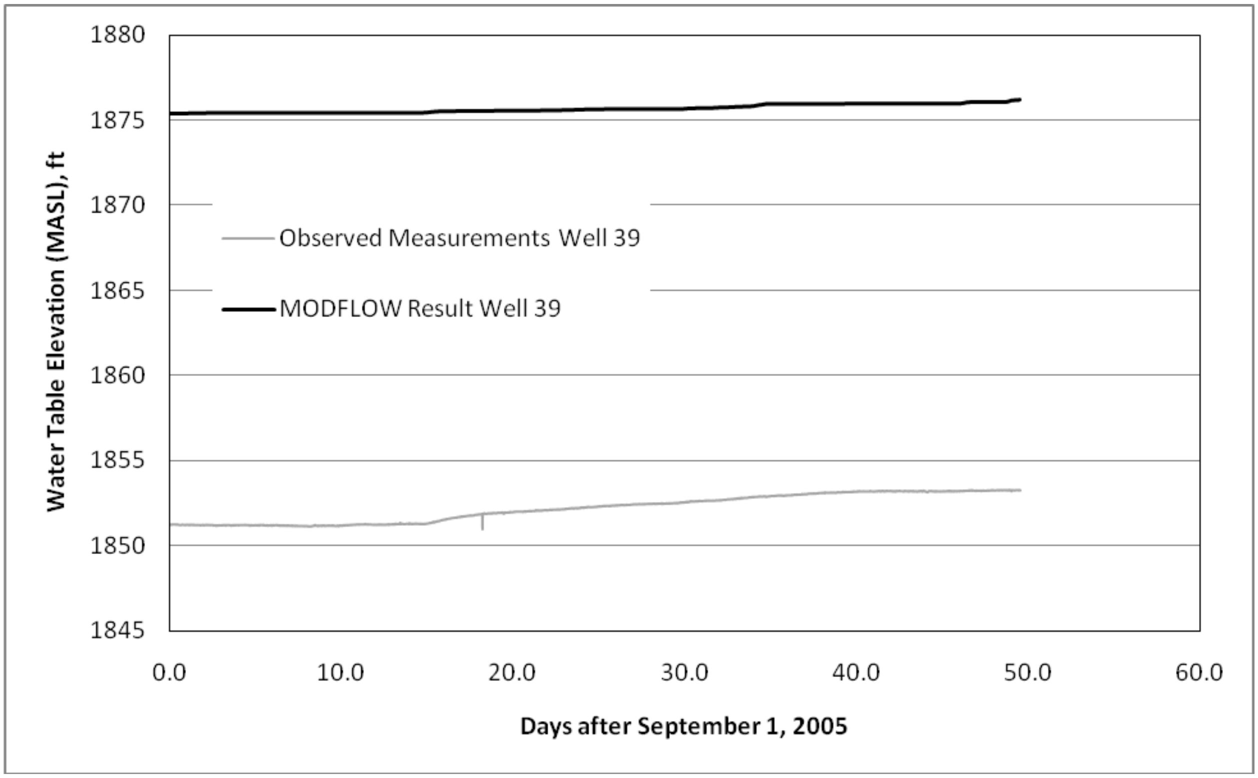

1878
1876
1874
1872
1870
1868
1866
1864
1862
1860
1858
Water Table Elevation (MASL), ft
Observed Measurements Well 31
MODFLOW Result Well 31
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005


1914
1913
1912
1911
1910
1909
1908
Water Table Elevation (MASL), ft
Observed Measurements Well 33
MODFLOW Result Well 33
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

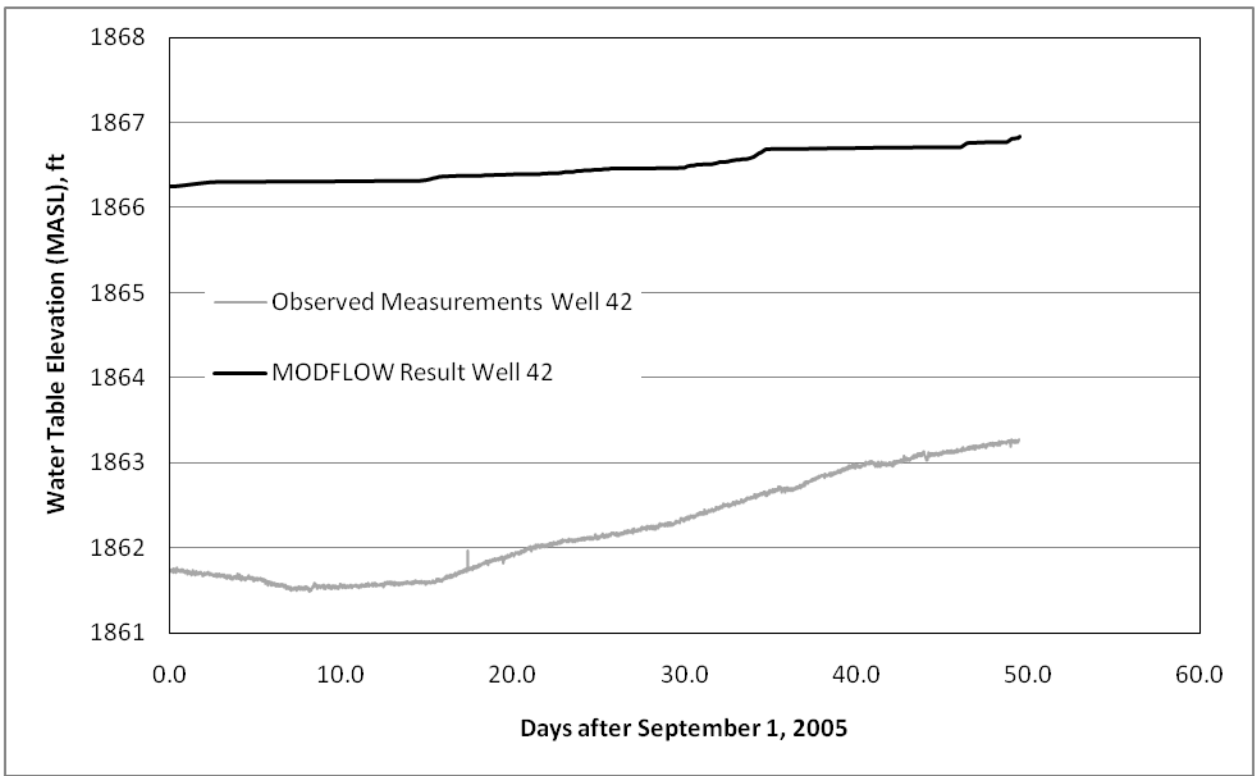

1870
1869
1868
1867
1866
1865
1864
1863
1862
Water Table Elevation (MASL), ft
Observed Measurements Well 35
MODFLOW Result Well 35
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005


1857
1856.5
1856
1855.5
1855
1854.5
1854
1853.5
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 36
MODFLOW Result Well 36

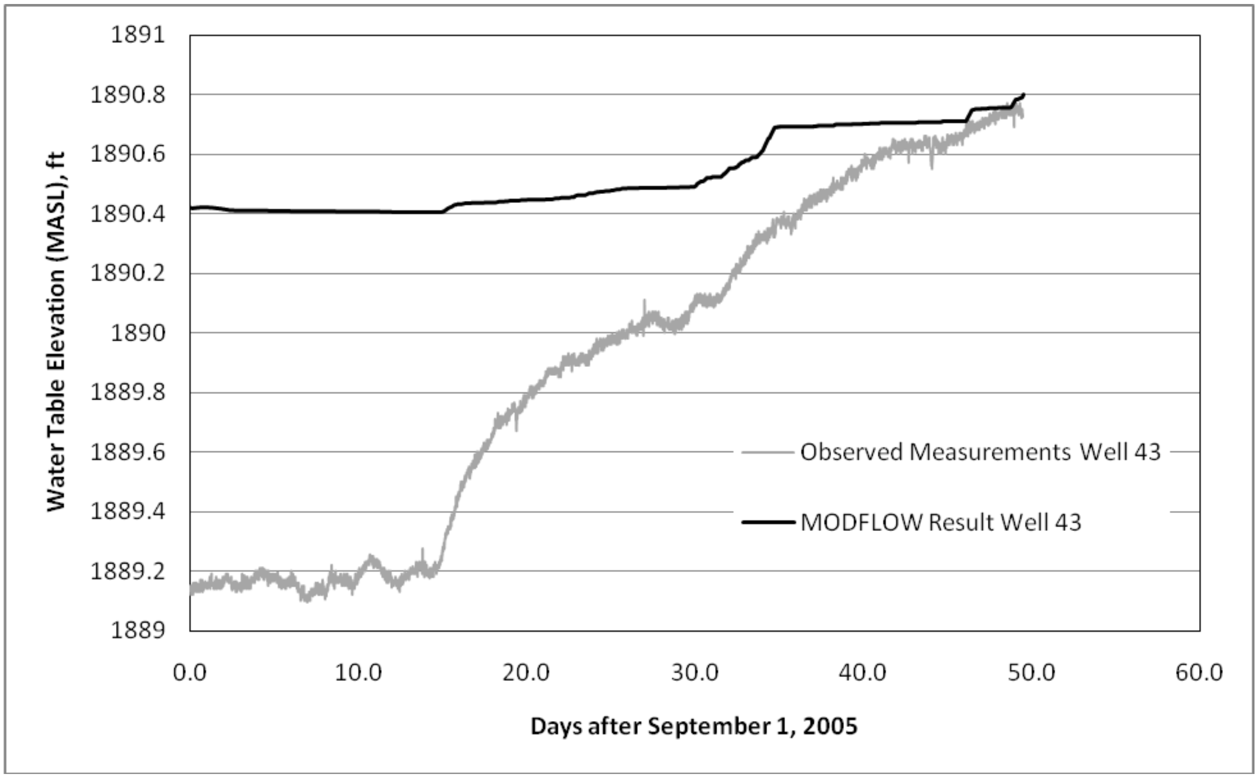

1886.5
1886
1885.5
1885
1884.5
1884
1883.5
1883
1882.5
1882
1881.5
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 37
MODFLOW Result Well 37

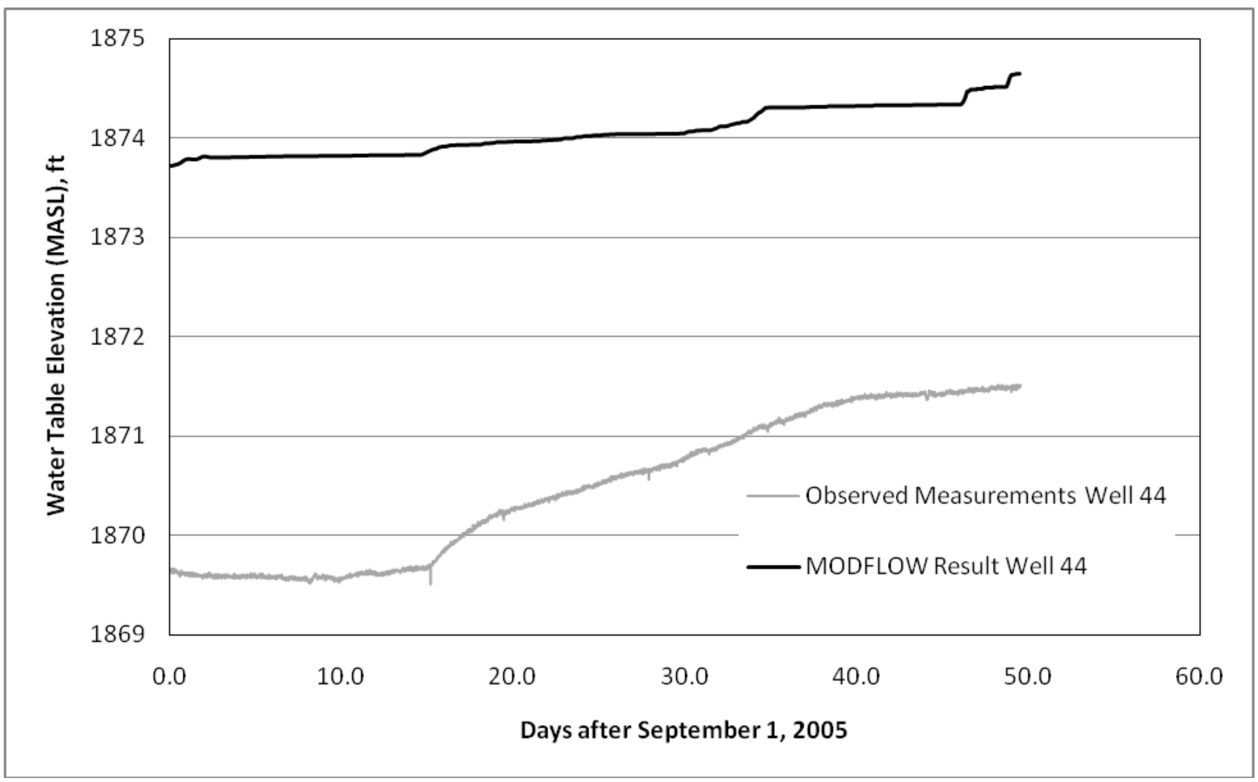


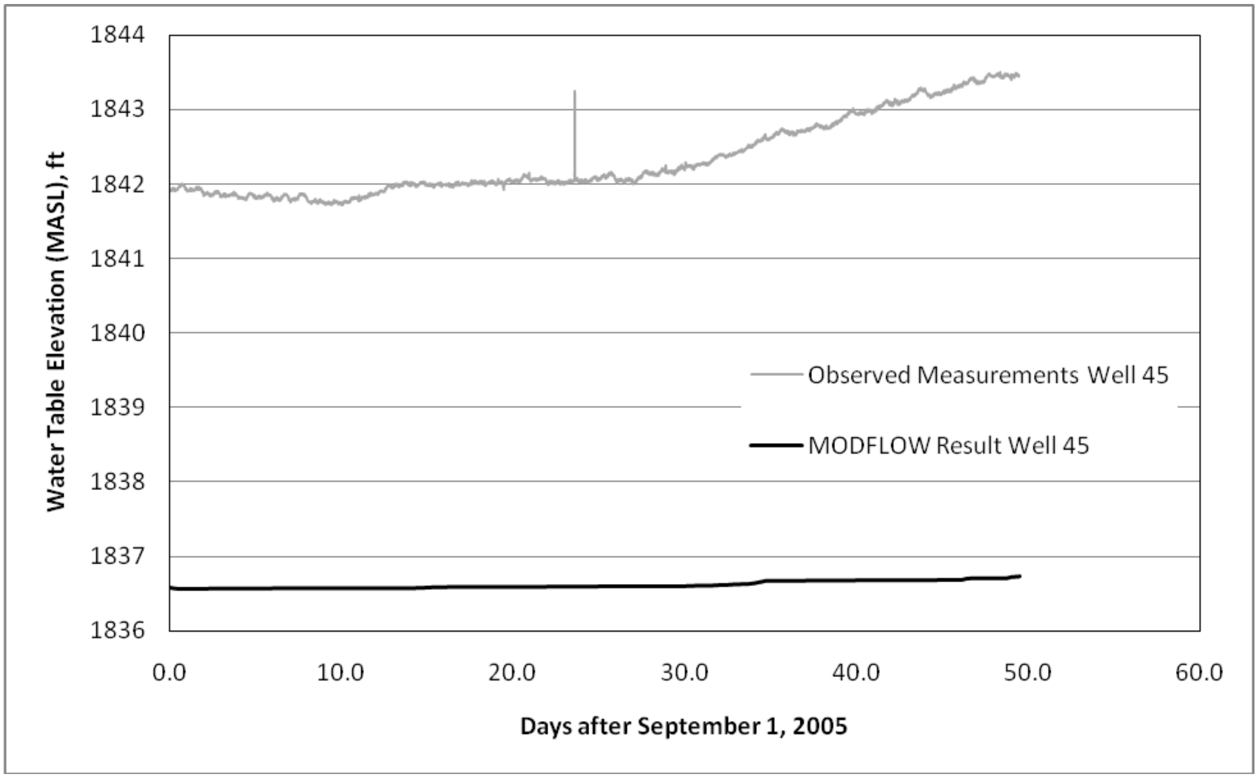
1880
1875
1870
1865
1860
1855
1850
1845
Water Table Elevation (MASL), ft
Observed Measurements Well 39
MODFLOW Result Well 39
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

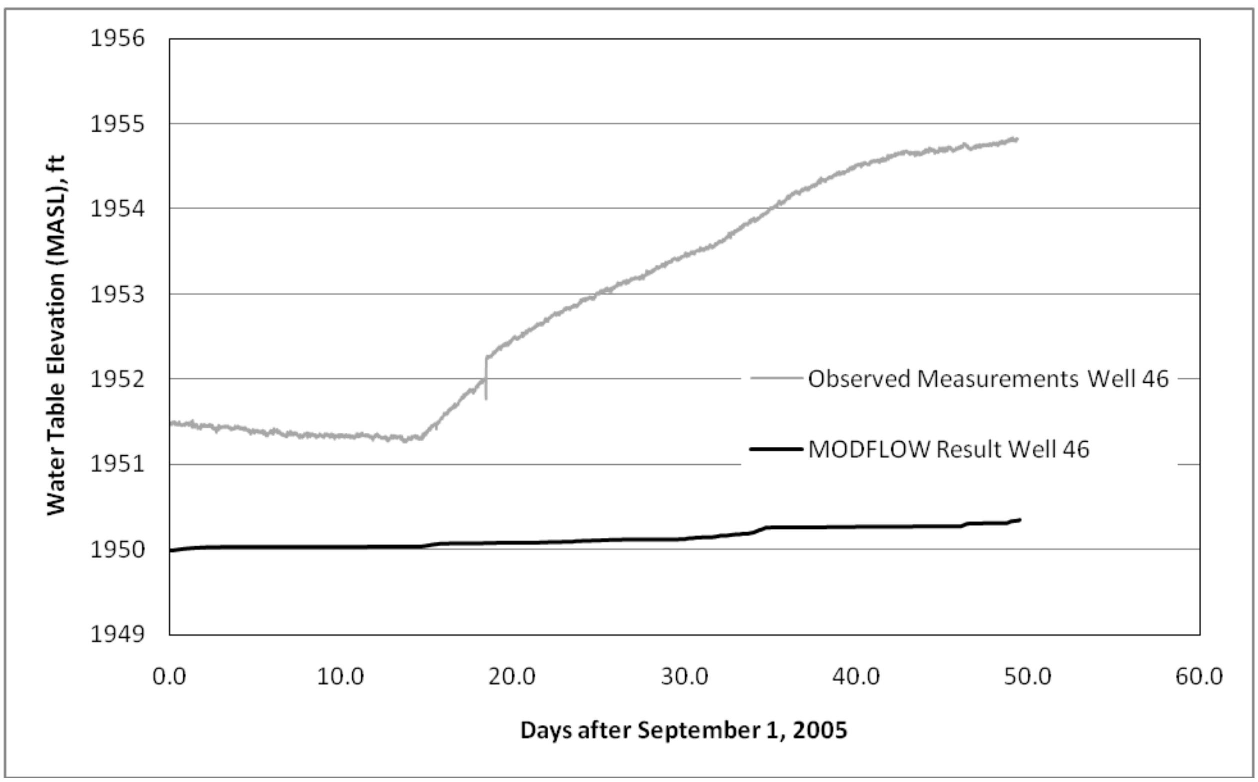


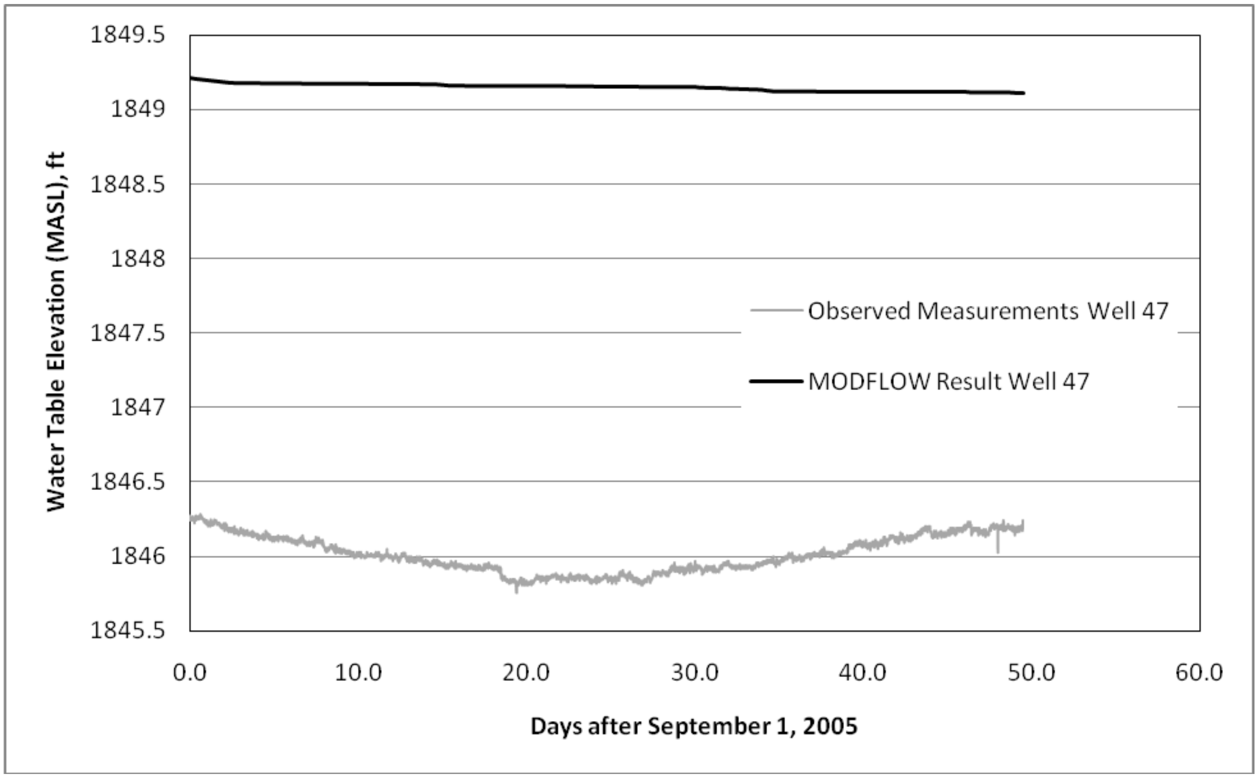
1868
1867
1866
1865
1864
1863
1862
1861
Water Table Elevation (MASL), ft
Observed Measurements Well 42
MODFLOW Result Well 42
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005

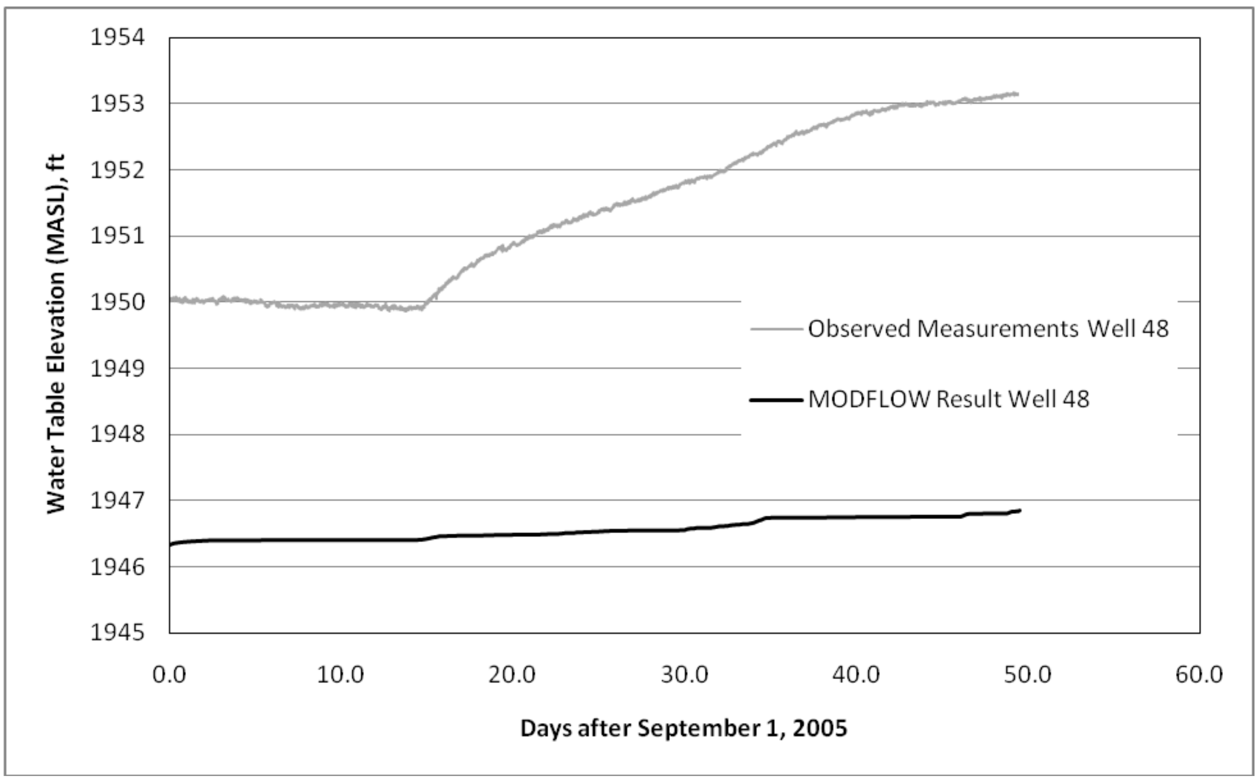

1891
1890.8
1890.6
1890.4
1890.2
1890
1889.8
1889.6
1889.4
1889.2
1889
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 43
MODFLOW Result Well 43

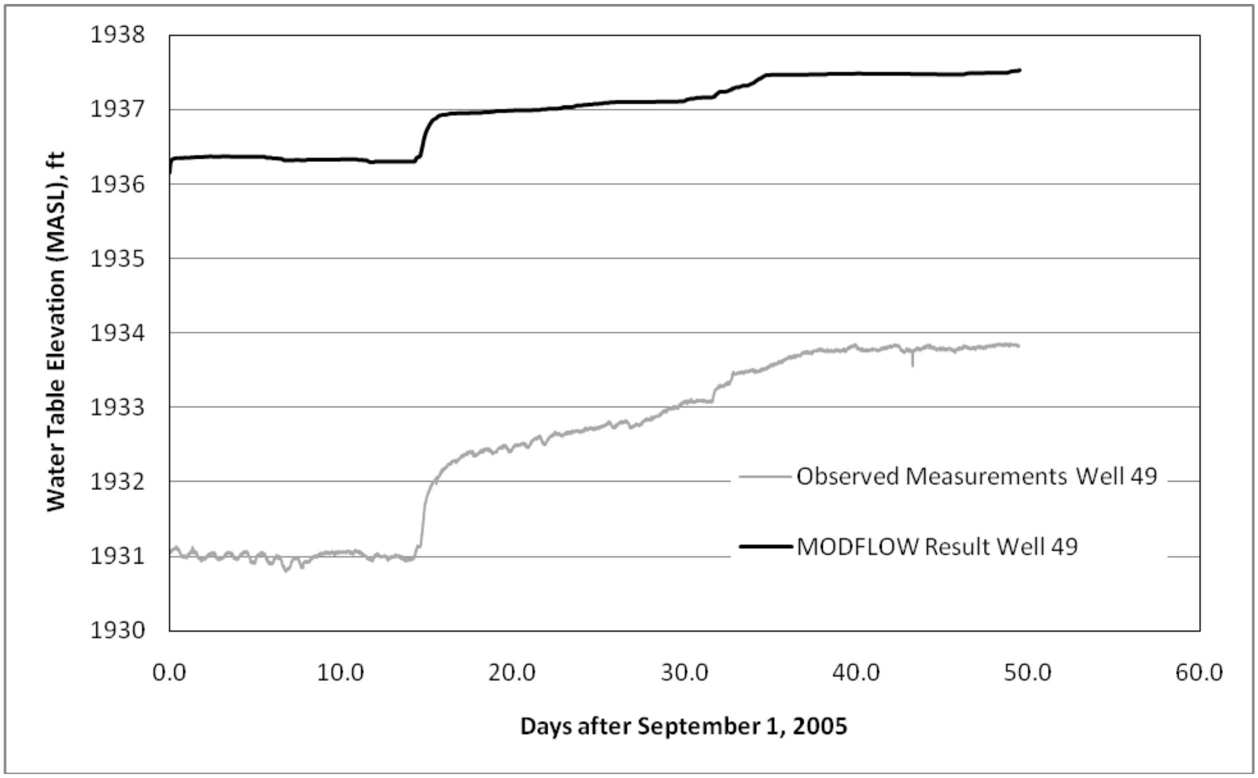

1875
1874
1873
1872
1871
1870
1869
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 44
MODFLOW Result Well 44


1844
1843
1842
1841
1840
1839
1838
1837
1836
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 45
MODFLOW Result Well 45


1956
1955
1954
1953
1952
1951
1950
1949
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 46
MODFLOW Result Well 46


1849.5
1849
1848.5
1848
1847.5
1847
1846.5
1846
1845.5
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 47
MODFLOW Result Well 47


1954
1953
1952
1951
1950
1949
1948
1947
1946
1945
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 48
MODFLOW Result Well 48


1938
1937
1936
1935
1934
1933
1932
1931
1930
Water Table Elevation (MASL), ft
0.0
10.0
20.0
30.0
40.0
50.0
60.0
Days after September 1, 2005
Observed Measurements Well 49
MODFLOW Result Well 49