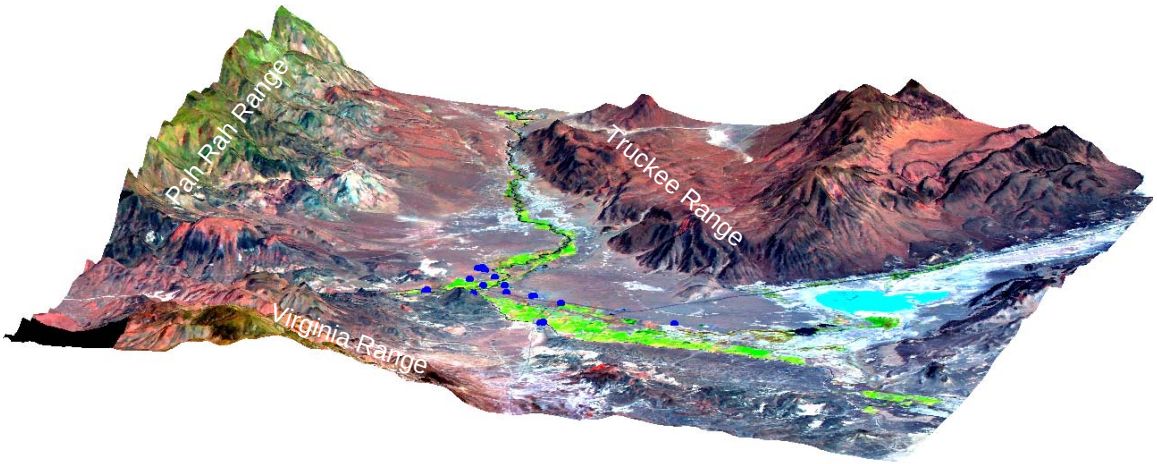
# *Regional Groundwater Model Development for the Fernley/Wadsworth Hydrographic Basins, Nevada*

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**ABSTRACT**

The Fernley/Wadsworth area is located in western Nevada, about 55 kilometers (34 miles) east of the cities of Reno and Sparks. The Truckee River flows from Lake Tahoe, through Reno/Sparks, and terminates in Pyramid Lake. Derby Dam diverts Truckee River water into the Truckee Canal, bringing water into the Fernley and Dodge Flat hydrographic basins. Fernley and Wadsworth are traditionally agricultural communities; however, as the population grows, agricultural land is decreasing. Less irrigated land decreases the amount of irrigation water available to recharge the basin. Municipal and domestic water usage and the amount of groundwater pumped from the aquifer are increasing with the population.

The objective of this study is to develop a predictive groundwater model that can be used to assess water supply scenarios regarding expected production well yield in the Wadsworth area, groundwater supply in the Fernley area with changes in diversions to the Truckee Canal, and the potential impacts of additional pumping on Truckee River flows. Both a steady-state and a transient model were constructed. Two transient simulations were performed: a six-year (2000 to 2005) calibration simulation, and a 20-year (2006 to 2025) predictive simulation that includes projected Fernley and Wadsworth groundwater pumping. Results show that the steady-state model provides a relatively accurate tool to quickly assess long-term changes to the groundwater system, while the transient model provides a more detailed look at the aquifer system by including temporal variations in groundwater pumping, recharge, and, Truckee River and Truckee Canal flows.

The major conclusions that can be drawn from this study regarding behavior of the system under current operating procedures include: increased seasonality of Truckee Canal flows leading to decreasing seepage amounts, decline of groundwater levels adjacent to new pumping centers with increased pumping, and decline of groundwater levels adjacent to older pumping centers with increased pumping. The area of significant water level decline, as determined by the model, is focused to the east of the Truckee River, which may cause poor-quality groundwater to encroach upon the new production wells. Although the area of significant drawdown is predicted to extend to the Truckee River, it appears the variations in river flow dominate the groundwater-surface water interactions. The management tool developed in this study can be used by the City of Fernley, the Pyramid Lake Paiute Tribe, and the U.S. Bureau of Reclamation to further assess the impacts of present and future water management decisions on the Fernley/Wadsworth area.

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# INTRODUCTION

Land-use changes in the Fernley and Wadsworth, Nevada, areas have affected the hydrologic system. During the last several decades, a shift from agricultural to domestic land use has occurred. The trend indicates that, over the decades, homes will cover land that was previously agricultural. The City of Fernley needs to develop additional capacity in its municipal water system to meet the needs of the growing population (Vpoint, 2005).

Previously, the Desert Research Institute (DRI) developed a groundwater model to help locate new production wells in the Fernley area (Pohll, 2004). Since the development of that model, data gaps have been identified. The purpose of this study was to collect more data in the Wadsworth area, and to expand the capacity of the previous model to accurately predict the impact of additional pumping in the Wadsworth area.

## Objective

The objective of this study is to develop a predictive groundwater model, which will simulate potential outcomes from various water-use decisions. Specifically, the model will be used to assess water supply scenarios regarding expected production well yield in the Wadsworth area, groundwater supply in the Fernley area with changes in diversions to the Truckee Canal, and potential impacts of additional pumping on Truckee River flows. This management tool is being created to allow the City of Fernley and the Pyramid Lake Paiute Tribe to evaluate present and future water management challenges.

## Report Organization

This report can be broken down into seven major components:

1. Introduction
2. Field Work
3. Modeling Approach
4. Model Calibration
5. Model Results
6. Conclusions
7. References

The Introduction section provides background information on the study area. The Field Work section provides information on new hydraulic testing and canal studies. The Modeling Approach section details the data used to construct the models and the modeling process. The Model Calibration section discusses how the models were calibrated and the results of the model calibration. The Model Results section presents selected output from the steady-state, transient calibration, and predictive simulations. The Conclusions section provides some general interpretation of the results. Last, the References section provides a comprehensive list of site-specific and more general literature used in this analysis.

## History of the Fernley/Wadsworth Area

The economy of the Fernley area has historically been agriculturally based (Van Denburgh and Arteaga, 1985). Derby Dam was completed in 1905 as part of the Newlands Reclamation Project. At one time, Derby Dam diverted almost half of the annual flow of the Truckee River into the Truckee Canal, approximately 800,000 $m^3$/day (211 million gallons

per day [mgd]; 236,900 acre-feet per year [afy]). Currently, diversions in the canal are generally less than 300,000 $m^3$/day (79 mgd; 88,800 afy) (Bratburg, 1980).

The Truckee Canal, located south of the Fernley farm district, is 52.3 km (32.5 mi) long and unlined for the majority of its length. Leakage from the canal provides approximately 40,000 $m^3$/day (11 mgd; 11,800 afy) of recharge to the Fernley and Wadsworth groundwater systems (Mihevc *et al*., 2002). Additional recharge due to seepage from lateral canals is estimated to be 12,000 $m^3$/day (3.2 mgd; 3,600 afy).

Irrigation is another major source of recharge in the Fernley area. It is estimated that irrigation recharge in the Fernley and Wadsworth areas is approximately 8,000 to 25,000 $m^3$/day (2.1 to 6.6 mgd; 2,400 to 7,400 afy). The population in the Fernley area has been steadily growing for the past 40 years and has doubled each decade from 1960 to 1980 (Van Denburgh and Arteaga, 1985). According to the U.S. Census Bureau, the population in Fernley grew from 5,188 to 8,543 between 1990 and 2000 (U.S. Census Bureau, 2001). The population increase in Fernley suggests that there is a shift from agriculture to a more urbanized land use. This economic shift results in fewer acres of irrigated land supplying surface water as recharge to the basin. Additionally, the population growth increases the municipal and domestic water usage, and therefore the amount of groundwater pumped from the aquifer.

Groundwater pumping for municipal and industrial purposes has increased since the 1960s in both Fernley and Wadsworth. The Nevada Cement Company uses groundwater to run its facility and has been the primary industry in the Fernley area since operation began in 1964 (Van Denburgh and Arteaga, 1985). Fernley and Wadsworth both rely on groundwater for their municipal water supply. Currently, groundwater for municipal and industrial purposes is estimated to be 28,000 $m^3$/day (7.4 mgd; 8,300 afy). There are also a number of domestic wells in the area, but much of this water is recycled via septic system return flows. The most dramatic increase in groundwater pumping has occurred in the Fernley area.

## Site Description

The Fernley/Wadsworth area is located within the Fernley and Dodge Flat hydrographic basins. Combined, the basins cover an area of approximately 588 $km^2$ (227 $mi^2$). The study area is bounded by the Pah Rah Range in the west, the Virginia Range in the south, the Truckee Range in north, and the Hot Spring Mountains in the northeast (Figure 1).

The climate of the area is described as a high, semi-arid desert region, with warm dry summers and cool moist winters. The Fernley/Wadsworth area is east of the Sierra Nevada Range, and lies within the rain shadow created by the mountains. Consequently, the annual precipitation in the basin floor is low, approximately 13 cm (5.12 in). The humidity ranges from 25 percent in the summer to 65 percent in the winter, with prevailing winds from the south.

## Geology

Mapping by Morrison and Davis (1984a,b) and Morrison and Frey (1965) extended the Lahontan-age allostratigraphic units of Morrison (1964) into the eastern part of the Dodge Flat area. Wilden and Speed (1974), and Bonham and Papke (1969) examined the

mineral resources and structure geology of Washoe, Churchill, and Lyon counties, portions of which are included in the field area.

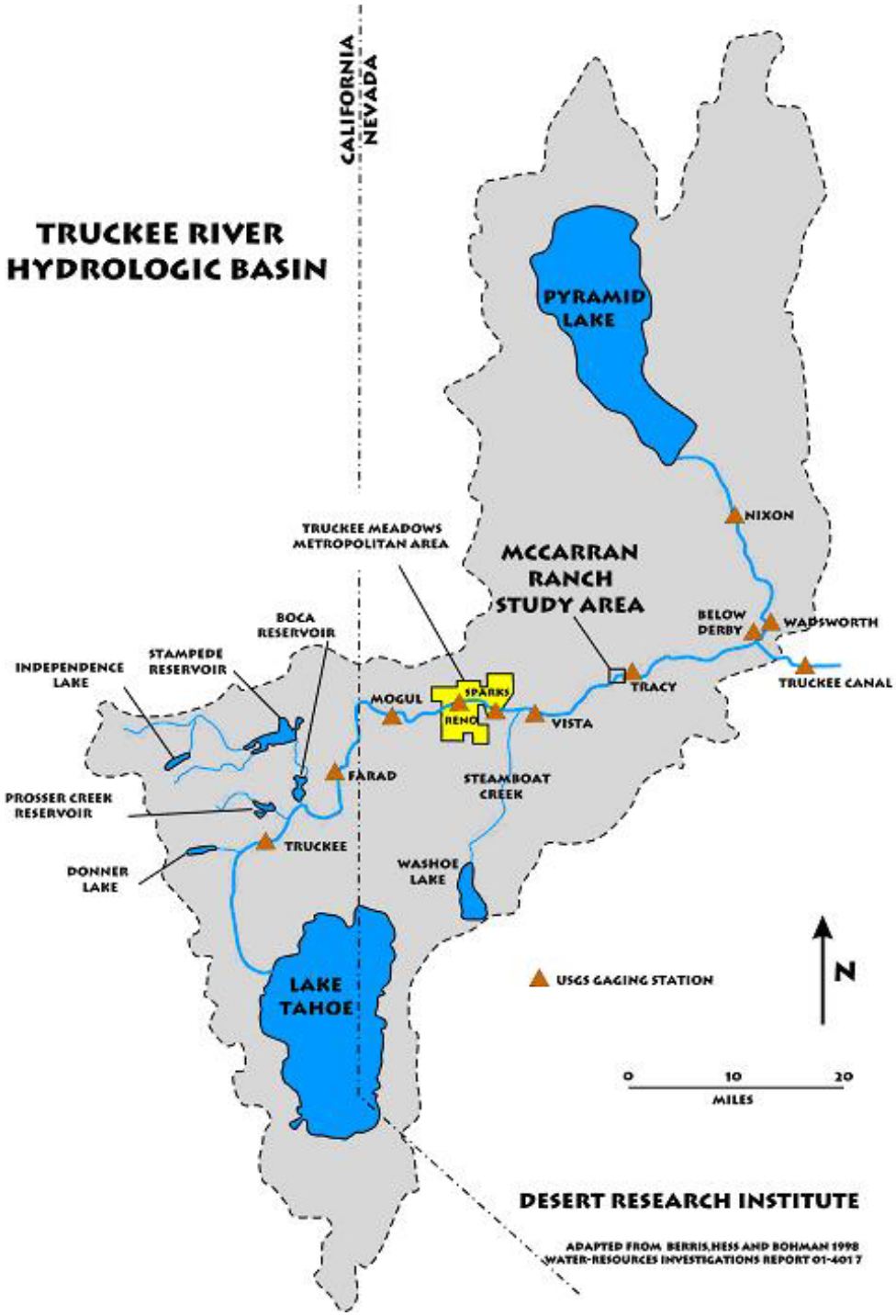



Figure 1. Study area location, including major faults in the area. Derby Dam diverts water from the Truckee River into the Fernley and Dodge Flat hydrographic basins via the Truckee Canal.

Bell (1984) mapped Quaternary faulting in the alluvium and described the relative age of the tectonic activity. Pleistocene faults extend into the alluvium in the extreme western part of the study area. Holocene faults transect large parts of the Dodge Flat hydrographic basin. More recent historical ground rupture, such as the Olinghouse fault that trails off into the Dodge Flat alluvium near its contact with mountain block, are also present within the study area (Watersource Consulting Engineers, 1998).

The geology of the area is characterized by the west-tilted, fault-block mountain range of Mesozoic-age granitic rocks with small amounts of metamorphic and volcanic rocks (Bonham and Papke, 1969). The primary rocks in these formations are andesite, rhyolite, tuff, and basalt (Willden and Speed, 1974). Erosion from the mountain ranges partially filled the surrounding basins with Tertiary sediments, which are assumed to be beneath the Quaternary alluvium (Sinclair and Loeltz, 1963). Pleistocene glacial Lake Lahontan, which once occupied more than 20,720 $km^2$ (8,000 $mi^2$) of northern Nevada, deposited over 305 m (1,000 ft) of lacustrine sediments on the basin floor, overlaying the original alluvium that was derived from the volcanic mountain range. These lacustrine deposits are comprised mostly of silt and clay in the middle of the basin, with sand and gravel along the base of the surrounding ranges. Because Lake Lahontan had no outlet, the water evaporated over time, causing soluble minerals to precipitate and the water to become saline. Pyramid Lake is the only remnant of Lake Lahontan in the Truckee River Basin (Peterson, 2003). During the last several thousand years, as the level of Pyramid Lake has fluctuated, the Truckee River has meandered through the Fernley and Dodge Flat basins, depositing gravels and sand over the older Lake Lahontan deposit (Sinclair and Loeltz, 1963).

An important geologic structure in the area is the Olinghouse fault, which is associated with uplift of mountains to the west and lowering of the valley floor. The Olinghouse fault extends from the Pah Rah Range towards Reno, and exhibits more than 365.8 m (1,200 ft) of dip-slip displacement (Bonham and Papke, 1969). The study area is also part of the Walker Lake Fault Zone, which is a large regional feature that consists of a number of discontinuous faults extending from south of Fernley to Pyramid Lake.

**Hydrogeology**

There are several studies that have investigated the hydrogeology of the Fernley/Wadsworth area. Among the first was Sinclair and Loeltz (1965). Their investigation explored the ability of the basin sediments to transmit groundwater, as well as the occurrence and movement of groundwater. Sinclair and Loeltz (1965) were also the first to examine the quality of groundwater and the possible origins of high total dissolved solids (TDS) concentrations in the groundwater. Pohll *et al*. (2001) performed an analysis of the impact of the high TDS groundwater on the salt loading to the Truckee River. Rollins (1965) continued this research by investigating the quality of irrigation and drainage water in the Fernley/Wadsworth basins. The University of Nevada, Reno, undertook a 3-year study during the early 1970s that examined the soil types, irrigation, and water requirements in the study area (Guitjens and Mahannah, 1971, 1972, 1973). A cooperative study prepared by the U.S. Geological Survey (USGS) for the State of Nevada Department of Water Resources evaluated the water resources of the Truckee River basin taking into account the factors influencing the inflow and outflow rates for the Fernley area (Van Denburgh *et al*., 1973), and was subsequently revised in 1979 (Van Denburgh and Arteaga, 1985). Katzer *et al*. (1998) evaluated three resource options for the City of Fernley: (1) a floodplain aquifer next

to the Truckee River; (2) use of the Dodge Flat groundwater system for artificial recharge, storage, and distribution of the Truckee River water through wells; and (3) use of the Dodge Flat aquifer as a groundwater resource. Others, such as Pohll (2004), have investigated the potential areas in the Fernley/Wadsworth area for production wells with a low probability of excessive drawdown and TDS plume encroachment.

**Trends in the Fernley/Wadsworth Area**

As part of the Newlands Project, water from the Truckee River was imported to the Fernley area. The Fernley/Wadsworth area developed as a primarily agricultural and ranching community, the principal crop being alfalfa hay.

There has been steady growth in population for the past 40 years in the Fernley area. According to Van Denburgh and Arteaga (1985), the population more than doubled each decade from 1960 to 1980. From 1990 to 2000, the population of Fernley grew from 5,188 to 8,543 (U.S. Census Bureau, 2001). An updated master plan (Vpoint, 2005) for the community projects the population could reach 35,000 in the next 20 years. As the population grows, housing developments are increasing and agricultural land is decreasing. This shift results in fewer acres of irrigated land supplying surface water as recharge to the basin. Also, the population growth increases the municipal and domestic water usage and the amount of groundwater pumped from the aquifer.

The local groundwater supply is extremely dependent on infiltration from surface and irrigation water. Natural recharge, derived from precipitation, provides only minor amounts of recharge. Surface water rights are implemented under the Apline and Orr ditch decrees, and the Operating Criteria and Procedures (OCAP) associated with the Newlands Project. When implemented, the Truckee River Operating Agreement (TROA) will modify operations of Truckee River reservoirs to enhance coordination and flexibility, while ensuring that existing water rights are served, and that flood control and dam safety requirements are met. Although TROA will supersede any agreements concerning the operation of all reservoirs, TROA is prohibited from adversely affecting the decreed water rights. The principal activities of TROA are intended to:

1. Enhance water management flexibility, reservoir recreational opportunities, and reservoir efficiency
2. Improve water quality and conditions for Pyramid Lake fish
3. Increase municipal and industrial drought supply
4. Minimize reservoir releases and the capacity for carryover storage
5. Allocate Truckee River water between California and Nevada
6. Decrease water use conflicts

In short, TROA creates opportunities for storing and managing categories of credit water. Signatories are generally allowed to accumulate credit water in reservoir storage by retaining or capturing water that otherwise would have been released. Such storage could only take place after a transfer in accordance with state water law and federal permits. Once accumulated, credit water would be classified by category with a record kept of its storage, exchange, and release. Credit water would be retained in storage or exchanged among the reservoirs until needed to satisfy its beneficial use. The City of Fernley would use its changed diversion rights and privately owned water to accumulate credit water in federal reservoirs.

## FIELD WORK

### Wadsworth Area Hydraulic Tests

Hydraulic testing in the Wadsworth area has been performed by DRI and Stetson Engineers (Stetson Engineers, 2006). The testing was conducted to gain a better understanding of the hydraulic properties along the Truckee River and to revise the groundwater flow model for the Fernley/Wadsworth area. A 3.5-day, constant-discharge pumping test was conducted from April 19, 2005, to April 22, 2005, on three wells, TW-5, Crosby, and BB-Irrigation in the Wadsworth area (Figure 2). However, lack of information on the construction and design on a fourth well, DP-Irrigation, limited the pumping test to two days. The pumping and recovery data plots for the wells are shown on Figure 3. Table 1 summarizes the locations, flow rates and estimated hydraulic parameters for these four wells.

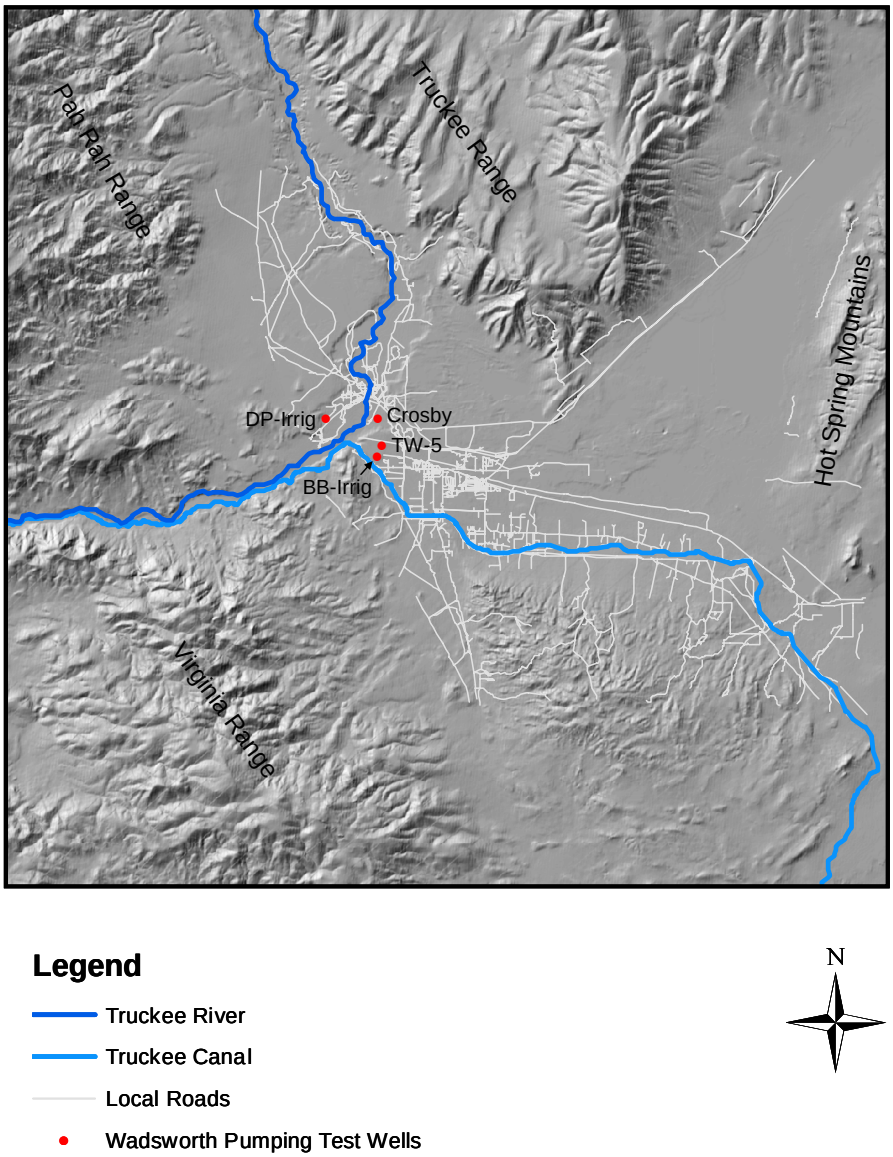



Figure 2. Location of pumping test wells in the Wadsworth area.

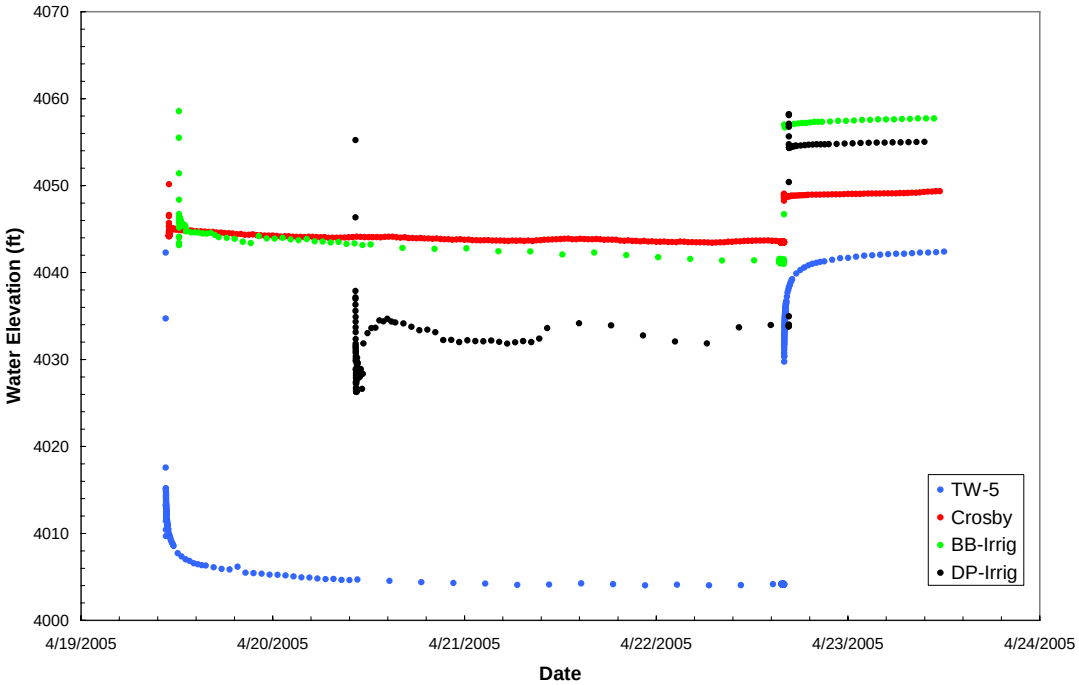



Figure 3. Water level as a function of time for the four pumping tests in the Wadsworth area.

Table 1. Pumping test analysis parameters for the Wadsworth area wells.

| Well Name | UTM – NAD 83 – Zone 11 Easting | Northing | Flow Rate ($m^3$/day) | Maximum Drawdown (m) | Specific Capacity ($m^3$/day/m | Hydraulic Conductivity (m/day) |
|---|---|---|---|---|---|---|
| BB-Irrigation | 304080 | 4388498 | 2,873 | 5.27 | 545 | 11 |
| Crosby | 304475 | 4388706 | 6,672 | 2.04 | 3,267 | 25 |
| DP-Irrigation | 302505 | 4388679 | 2,246 | 6.52 | 344 | 9 |
| TW-5 | 304691 | 4388489 | 2,055 | 11.61 | 177 | 3 |

A two-phase field campaign was undertaken by Stetson Engineers to evaluate the groundwater resources in the Wadsworth area. The objectives of the Phase I report (Stetson Engineers, 1999) were to:

- Characterize the local aquifer based on a review of existing data and well drilling
- Demonstrate the reliability of the Truckee River as a recharge source based on historical records
- Determine the potential yield of the aquifer
- Recommend locations, target zones, spacing and depth, and casing design criteria for potential future production wells

The objectives of the Phase II project (Stetson Engineers, 2006) were to:

- Install new monitoring wells, analyze the lithology, and perform pumping tests
- Perform a pumping test on the existing Crosby irrigation well

- Sample groundwater and analyze water quality
- Conduct multi-well stress test of the DP-Irrigation and Big Bend Ranch wells
- Update the evaluation of the potential water supply in the south Wadsworth area.

The Stetson Engineers (2006) report developed numerous conclusions and recommendations regarding the potential use of the Wadsworth area groundwater for future water supply, as summarized below.

- They estimated that approximately 10,000 afy could be produced from the north and south well fields in the south Wadsworth area. Of this amount, they predicted that approximately 2,000 to 3,000 afy could be extracted in the area of the DP-Irrigation well. The balance (6,000 to 10,000 afy) could be produced in the vicinity of the Big Bend Ranch.
- The multi-well aquifer testing that was performed in the Big Bend Ranch area refined their estimates of potential pumping rates, pumping lifts, well design, future well locations, well spacing, and water quality.
- The water quality analysis suggests that blending of water from the north side of the Truckee River with water pumped in the Big Bend Ranch area could achieve the proposed arsenic standard of 0.010 mg/l. Their analysis suggests that such blending would result in a composite concentration of 0.008 mg/l, but significant uncertainty still exists in the arsenic concentrations within the Truckee River.
- They also recommend that new production wells be phased in such that additional water quality sampling could be done to ensure that the water quality meets State health standards.

**Mapping Truckee Canal Standing Water**

A survey was performed in January 2006 to ascertain how much water was impounded in the canal after the closure of water supply from the Truckee River in December 2005. The following activities were performed on 32 sites along the Truckee Canal between Wadsworth and Hazen: location and elevation were measured using a global positioning system (GPS), depth and length of the stagnant water were measured with a rod and tape, and photographs of the impounded water were taken. The information from the survey was grouped into six categories:

- Dry (areas with no water)
- Unlined canal with water depth less than 0.3 m (1.0 ft)
- Unlined canal with water depth ranging between 0.3 and 0.6 m (1.0 and 2.0 ft)
- Unlined canal with water depth greater than 0.9 m (3.0 ft)
- Concrete-lined canal with water depth greater than 0.3 m (1.0 ft)
- Intermittently lined canal with water depth greater than 0.3 m (1.0 ft)

The depth data were plotted and superimposed on the Truckee Canal rates calculated by Mihevc *et al*. (2002). The resultant plot (Figure 4) suggests that there is more spatial variability in the seepage than indicated by the previous seepage analysis. In some sections of

the canal there is a general agreement between the estimated seepage rate and pond levels, but the pond levels vary significantly along the canal. It was expected that in areas of higher estimated seepage rates, there would be little to no standing water during the period when the canal was not in operation. The Truckee Canal is lined on all sides with concrete just before it enters the western edge of the Fernley irrigation district. The lined sections tended to have more ponded water than unlined sections. It is important to note that the canal slope would most likely control the ponding behavior, but the canal slope was not measured during the canal survey.

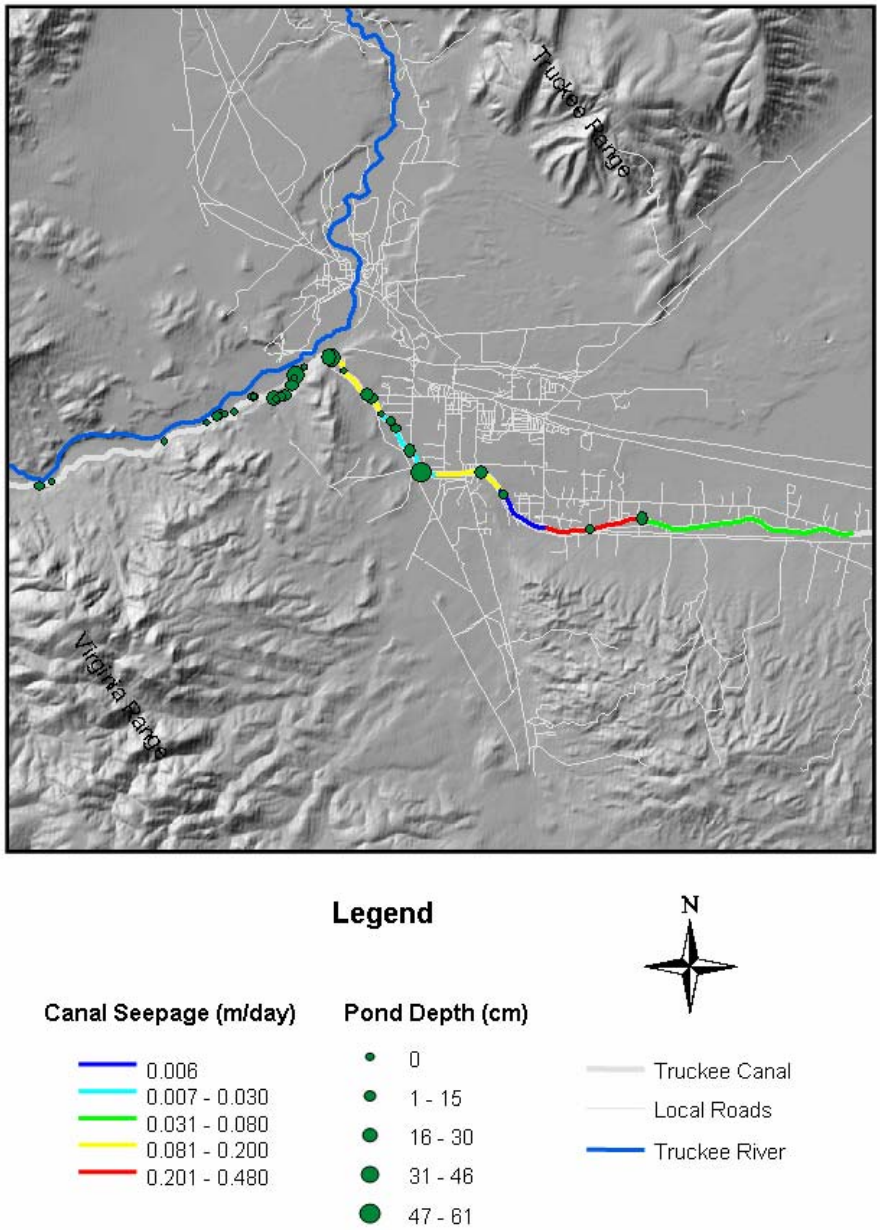



Figure 4. Depth of water along the Truckee Canal in January 2006 superimposed on the Truckee Canal seepage rates calculated by Mihevc *et al*. (2002).

## MODELING APPROACH

This section describes the methods used to construct the numerical groundwater flow model for the Fernley/Wadsworth hydrographic basins. The process follows that described in the American Society for Testing and Materials (ASTM) Standard Guides D 5447-93 (ASTM, 1993a) and D 5610-94 (ASTM, 1994b), and is also outlined in Anderson and Woessner (1992). The process includes the following items:

- Define purpose
- Construct conceptual model
- Choose mathematical model
- Choose code/program
- Verify code
- Design model
- Calibrate model
- Verify model
- Conduct sensitivity analysis
- Present results
- Conduct post-audit

The purpose was presented in the Objective section. The conceptual model is presented in the Conceptual Model section below. The mathematical model and code description are given in the Groundwater Flow Equations and Code Selection sections, respectively. The modeling program, MODFLOW (Harbaugh *et al.*, 2000), has been extensively tested and verified so further code verification was not done. The model design is discussed in the Boundary Condition section and calibration is subdivided into the Steady-state and Transient Calibration sections. Unfortunately, an independent dataset was not available to verify the Fernley/Wadsworth groundwater flow model, so this step is omitted from the modeling process. Likewise, a post-audit was not performed, but could be done in the future. Although an uncertainty analysis was not included in this report an uncertainty analysis was performed and is reported in Bansah (2006). The results of this study are presented in the Model Results section.

### Conceptual Model

The groundwater flow system in the Fernley/Wadsworth area is controlled by the following processes:

- Regional groundwater gradient
- Aquifer-river interactions
- Seepage from the Truckee Canal
- Seepage from canal laterals
- Natural recharge

- Irrigation recharge
- Groundwater pumping
- Evapotranspiration (ET)

The regional groundwater system in the Fernley/Wadsworth area can be subdivided into eastern and western components. Groundwater flows to the east from below Derby Dam and then runs generally parallel to the Truckee River as it moves north toward Pyramid Lake. In the southern portion of the study area, groundwater flows north through Fernley, and then northwest toward the Truckee River. There is a groundwater divide located in the eastern portion of Fernley that differentiates the west and east flow systems. Groundwater that originates on the eastern side of the divide flows toward the Fernley Sink or southeast toward Hazen.

The Truckee River acts as both a source and sink for groundwater flow. The magnitude of the river-aquifer interaction is uncertain, primarily because the amount of groundwater that flows into the Truckee River is small compared to the typical streamflow rates. During low-flow periods, when the impact of groundwater can be measured, the Truckee River appears to be a gaining stream. Two-low flow studies that have been conducted indicate that the net groundwater contribution between Wadsworth and Nixon range between 7,000 and 39,000 $m^3$/day (Bratberg, 1980; Pohll *et al*., 2001). Detailed studies upstream of Wadsworth have not been performed, but analysis of streamflow differences between Derby Dam and Wadsworth under low-flow conditions suggests that this reach is also gaining groundwater. Under higher-flow conditions, it is likely that the Truckee River acts as a source of groundwater.

The Truckee Canal is hydraulically disconnected from the regional groundwater (Mihevc *et al*., 2002). The water table is always below the bottom of the canal, which indicates that the canal always acts as a recharge source to the groundwater system. Several studies have estimated the magnitude of the seepage loss (Van Denburgh and Arteaga, 1985; Pohll *et al*., 2001; Mihevc *et al*., 2002). Mihevc *et al*. (2002) used detailed thermal measurements along the canal to estimate the spatial variability and total seepage loss within the Fernley area. Their study suggests that seepage varies considerably along the length of the canal and that the average seepage rate is 3,300 $m^3$/day/km (2.17 cfs/mi) within the Fernley area. Pohll *et al*. (2001) developed a range of canal seepage rates from 1,700 to 5,000 $m^3$/day/km (1.12 to 3.29 cfs/mi). The canal laterals located within Fernley also provide recharge to the groundwater system, but less is known about the seepage rates from these canals.

Natural recharge for the hydrographic areas covering the model domain was estimated by Van Denburgh *et al*. (1973), using the Maxey-Eakin method. Recharge in the Fernley area was estimated as approximately 2,000 $m^3$/day (0.528 mgd; 592 afy). Dodge Flat was estimated to receive 5,000 $m^3$/day (1.32 mgd; 1,480 afy). The Tracey Segment was estimated to receive 20,000 $m^3$/day (5.28 mgd; 5,922 afy). The model domain includes a small section of the Tracey Segment and almost all of the Dodge Flat and Fernley Area hydrographic basins.

Inefficiencies of flood irrigation within the agricultural areas in Fernley and Wadsworth provide another source of recharge. Pohll (2004) estimated that irrigation

recharge rates range between 30 and 90 cm/yr (0.98 and 2.95 ft/yr). The area covered by irrigation is constantly decreasing due to development.

Evapotranspiration from phreatophytes and open playa area removes a significant amount of groundwater from the system. Van Denburgh and Arteaga (1985) estimated an ET rate from bare playa and native vegetation to be approximately 26,000 $m^3$/day (6.87 mgd; 7,699 afy). Pohll *et al*. (2001) estimated that ET within the study area was 18,000 $m^3$/day (4.76 mgd; 5,330 afy) and 14,000 $m^3$/day (3.70 mgd; 4,145 afy) for water years 1993 and 1996, respectively. It is important to note that, although a majority of ET is derived from groundwater, the City of Fernley wastewater treatment plant diverts water to the playa located in the eastern portion of the study area. A large percentage of this treated water is lost to ET.

Groundwater pumping in the model domain occurs primarily in three areas: the extent of the City of Fernley municipal system, the Wadsworth area, and the Nevada Cement Company property. The City of Fernley average groundwater production over the period 2000 to 2006 is 28,235 $m^3$/day (7.46 mgd; 8,360 afy), yet the pumping rates are clearly trending upward (Vpoint Engineers, written communication, 2006). In the Wadsworth area, the total groundwater pumping is 1,100 $m^3$/day (0.29 mgd; 326 afy) (Stetson Engineers, 2003). The Nevada Cement Company production rates (6,800 $m^3$/day; 1.80 mgd; 2,013 afy) were estimated based on available water rights information from the Nevada State Engineer's Office.

All of the components described above contribute to the overall water balance of the Fernley and Dodge Flat hydrographic basins. Table 2 shows the water balance simulated by the steady-state version of the model, which is described in more detail below.

**Groundwater Flow Equations**

The groundwater flow equation for transient flow in a confined aquifer system can be developed by combining the equation for conservation of fluid mass and Darcy's Law

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right)+W = S_s \frac{\partial h}{\partial t} \tag{1}$$

where $K$ [L/T] is the hydraulic conductivity in the x, y, and z directions, $h$ [L] is the hydraulic head, W [1/T] is the volumetric flux per unit volume representing sources and sinks, $S_s$ [1/L] represents the specific storage, and t [T] is time. For unconfined conditions, specific storage is replaced by specific yield, and the entire equation is multiplied by the saturated thickness.

**Code Selection**

MODFLOW-2000 (Harbaugh *et al*., 2000) was used to solve Equation (1) for hydraulic head. The most recent version, which incorporates the SFR2 package (Niswonger and Prudic, 2005), was used to allow simulation of unsaturated zone flow beneath the Truckee Canal.

Multiple MODFLOW modules were used in the groundwater flow model. The Basic (BAS) module contains information on the initial conditions and IBOUND array, which defines active and inactive cells within the domain. The Layer Property Flow (LPF) module contains the aquifer type, and hydraulic parameters required to solve the finite-difference

equations. The Preconditioned Conjugate-Gradient (PCG) module uses modified incomplete Cholesky preconditioning to efficiently solve the matrix of finite difference equations (Hill, 1990). The Discretization (DIS) module contains information on the grid geometry and temporal discretization. The Well (WEL) module was used to simulate pumping wells, canal laterals, and mountain block recharge. The Evapotranspiration (ET) module was used to simulate ET within playa and phreatophyte areas. The Constant Head (CHD) package was used to simulate specified head boundary conditions, and the General Head Boundary (GHB) package was used to simulate groundwater flow to Pyramid Lake. The Stream Routing (SFR2) package was used to simulate the surface water-groundwater interactions along the Truckee River and Truckee Canal. The Recharge (RCH) module was used to simulate irrigation recharge.

Table 2. Fernley-Wadsworth steady-state groundwater flow model water budget.

| **INPUTS** | **($m^3$/day)** | **(acre-ft/yr)** |
|---|---|---|
| *Groundwater Flow* | | |
| Truckee River Canyon | 3,480 | 1,031 |
| Hazen | 0 | 0 |
| North | 0 | 0 |
| *Recharge* | | |
| Irrigation | 8,576 | 2,539 |
| Mountain Block | 17,273 | 5,115 |
| *Surface Water* | | |
| Truckee River | 1,936 | 573 |
| Truckee Canal | 47,790 | 14,151 |
| Lateral Canals | 12,103 | 3,584 |
| **Total Inputs:** | 91,158 | 26,993 |
| **OUTPUTS** | **($m^3$/day)** | **(acre-ft/yr)** |
| *Groundwater Flow* | | |
| Truckee River Canyon | 0 | 0 |
| Hazen | 13,482 | 3,992 |
| North | 11,779 | 3,488 |
| *Surface Water* | | 0 |
| Truckee River | 15,988 | 4,734 |
| *Groundwater Pumping* | | |
| Fernley Well 4 | 8,759 | 2,594 |
| Fernley Well 9 | 9,656 | 2,859 |
| Fernley Well 11 | 9,920 | 2,937 |
| Washoe County - Gregory St. | 437 | 129 |
| Washoe County - Stampmill 1 | 53 | 16 |
| Washoe County - Stampmill 2 | 48 | 14 |
| Nachez | 21 | 6 |
| Pyramid Lake Paiute Tribe - Industrial | 354 | 105 |
| Pyramid Lake Paiute Tribe - Well 2 | 43 | 13 |
| Pyramid Lake Paiute Tribe - Well 3 | 144 | 43 |
| Nevada Cement | 6,800 | 2,014 |
| *Evapotranspiration* | 13,675 | 4,049 |
| **Total Outputs:** | 91,158 | 26,992 |
| **Inputs - Outputs:** | 0 | 0 |
| **Percent Discrepancy:** | 0.00% | 0.00% |

## Model Domain

The model domain is subdivided into finite-difference cells. The horizontal cell size is a constant 200 m x 200 m (656 ft x 656 ft) throughout the model domain (Figure 5). The vertical grid spacing is variable, with grid size on the order of 25 m (82 ft) near the surface (layer 1), and increasing gradually to 200 m (656 ft) at the bottom of the model. The total number of finite-difference cells is 240,000 (150 x 160 x 10), although many cells are inactivated, as they are outside of the model area. The active area varies for each model layer depending on the location of the bedrock surface, but layer three has the largest active area of $3.2 \times 10^8$ $m^2$ (79,000 acres; 123 $miles^2$).

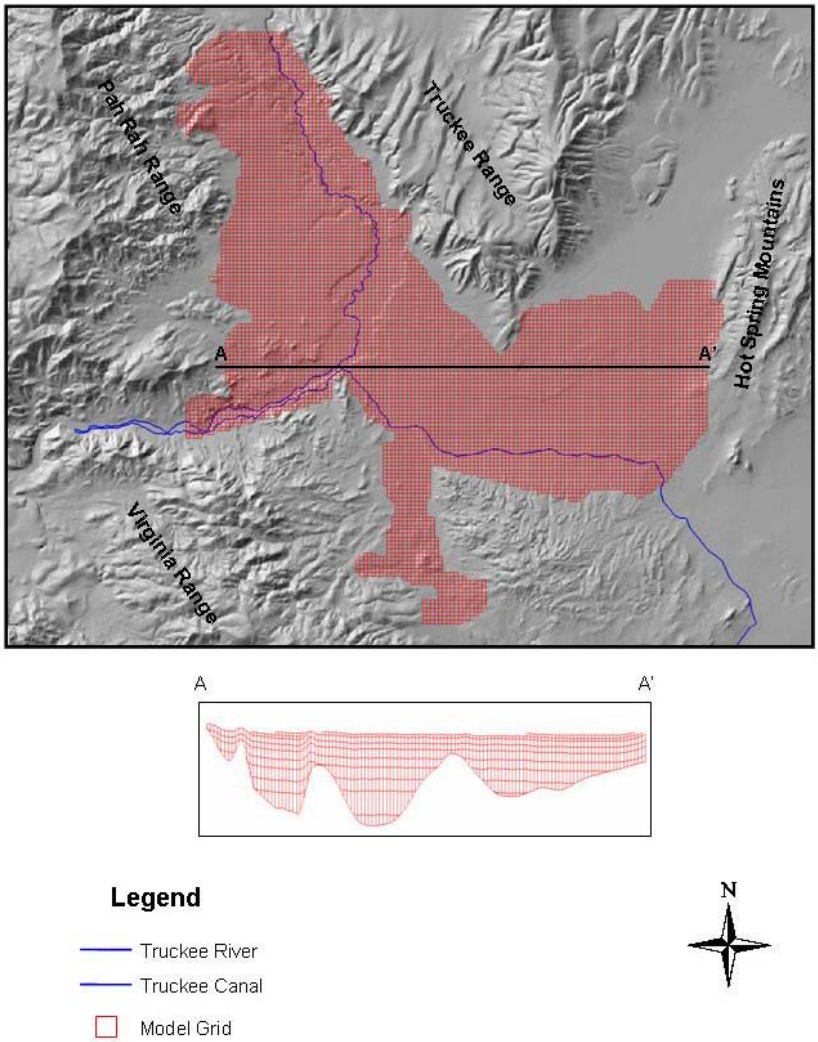



Figure 5. Groundwater model finite-difference grid overlain on the digital elevation model.

The model grid cells are specified as a combination of confined and convertible layer types. The confined cells are used for layers 4 through 10, where the water table is not likely to intersect the top of the layer. The upper three layers were simulated as convertible, and are allowed to dry or inactivate when the simulated head decreases below the bottom of the cell. The wetting package was not implemented in this model due to numerical instabilities.

## Boundary Conditions

Boundary conditions are needed to solve the groundwater flow equation. There are three types of boundary conditions:

- Dirichlet – Specified head
- Neumann – Specified flux
- Cauchy – Head dependent

Dirichlet boundary conditions are used to pre-specify the hydraulic head at certain cells. The model does not solve for head at these locations. Neumann boundary conditions are applied as a positive or negative flux into the model domain. Cauchy boundaries are head-dependent: a flux across a boundary is calculated based on the difference in head between the boundary head and the first adjacent model cell, multiplied by the aquifer conductance (Harbaugh *et al*., 2000). The following boundary conditions were applied to the model:

- Bedrock – no-flow boundary designated for the bottom of the model, as the bedrock is assumed to be impermeable
- Truckee Canal – internal fluxes calculated via the SFR2 package
- Mountain block recharge – specified flux boundaries (WEL package) for the Virginia, Truckee, and Pah Rah ranges, and the Hot Spring Mountains
- Northern boundary – head-dependent boundary (GHB package) representing groundwater flow moving toward Pyramid Lake
- Eastern boundary – specified head boundaries (CHD package) representing groundwater flow moving toward Hazen
- Western boundary – specified head boundaries (CHD package) representing groundwater inflow along the Truckee River corridor
- Truckee River – internal general head boundary using the SFR2 package
- Truckee Canal Laterals – internal specified flux boundary using the WEL package
- Production wells – internal specified flux boundary using the WEL package
- Irrigation recharge – internal specified flux boundary using the RCH package
- Evapotranspiration – internal general head boundary using the ET package

<u>Bedrock</u>

A no-flow boundary was used along the base of the model. No appreciable amount of groundwater flow is known to flow into or out of the Fernley groundwater system from consolidated rocks, other than the portion associated with mountain block recharge (Van Denburgh and Arteaga, 1985).

The bedrock surface elevation was determined by Pohll (2004). Potential fields (gravity and magnetic field measurements) modeling was used to determine a reasonable depiction of the bedrock surface (Widmer, 2001). As additional information was collected within the basin, the original bedrock surface model was adjusted to reflect the information

collected in new boreholes. Specifically, the depth to bedrock was increased in the southern portion of the model because bedrock was not encountered in the shallow (< 60 m; 197 ft) borehole drilled adjacent to the Fernley airport. The modified bedrock surface is shown in Figure 6.

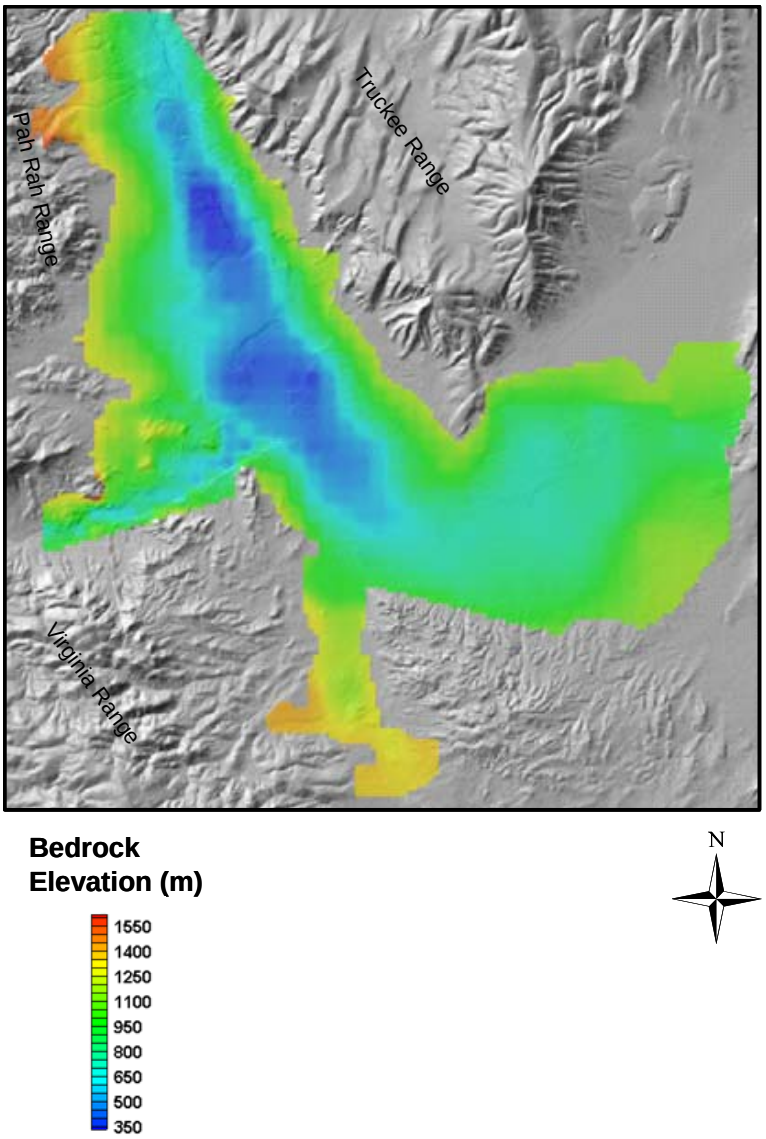



Figure 6. Elevation of the bedrock surface.

Natural Recharge

Natural recharge estimates were obtained by using the Precipitation Regressions on Independent Slopes Model (PRISM) (Daly *et al*., 1994), which provides average annual precipitation rates according to elevation. The PRISM model was developed utilizing precipitation data from 1961-1990 obtained from weather stations throughout Nevada (Daly *et al*., 1994). Geographical information system methods were used to delineate watershed subbasins, and regression equations (Maurer and Berger, 1997; Berger, 1998) that relate precipitation with subsurface flow rates, calculating natural recharge.

Berger (1998) and Maurer and Berger (1997) developed a set of regression equations that relate precipitation to recharge and surface runoff based on runoff measurements taken from five stations along the mountain front in Eagle Valley in western Nevada. Regression analysis of mean annual precipitation and mean annual runoff yielded a correlation coefficient of 0.89 using

$$RO = (2.28 \times 10^5)P^{3.96} \tag{2}$$

where *RO* (in/yr) is the estimated average annual runoff and *P* (in/yr) is the average annual precipitation in the mountain block.

Subsurface outflow from streams in the mountain block areas was estimated at eight stations using data from slug tests, chloride-mass-balance methods, and geophysical tools in Eagle Valley. The mean annual water yield was calculated by taking the sum of the mean annual runoff and subsurface flow. Regression analysis was also performed on the area-weighted precipitation, calculated from the PRISM map, and the mean annual water yield. This analysis produced the following relationship:

$$\mathrm{W} = (2.73 \times 10^3)P^{2.56} \tag{3}$$

where *W* (in/yr) is the mean annual water yield and *P* (in/yr) is area-weighted precipitation. The regression yielded a correlation coefficient of 0.86. The difference between the water yield and runoff is assumed to be the rate of subsurface recharge.

Watershed subbasins were delineated using GIS. Subtracting the runoff from the water yield and multiplying the result by the area of the subbasin produced the volumetric subsurface flow

$$\mathrm{S} = (\mathrm{W} - \mathrm{RO})A \tag{4}$$

where *S* ($L^3/T$) is the volumetric subsurface flow, *W* (L/T) is the water yield, and *A* ($L^2$) is the area of the subbasin. Figure 7 shows the average mountain block recharge for the subbasins used in the analysis. These values were applied as specified flux boundary conditions in the flow model.

The total mountain block recharge as simulated by the steady-state model is 17,200 $m^3$/day (4.54 mgd; 5,093 afy). The transient calibration and predictive simulations used a recharge-volume scale based on the annual precipitation as measured at the Reno, Nevada, airport. The recharge was scaled by simply multiplying the average recharge by the amount that precipitation deviated from the long-term average. For example, if for a given year, the annual precipitation as recorded in Reno, was 50 percent of the long-term average, the total recharge was decreased by 50 percent.

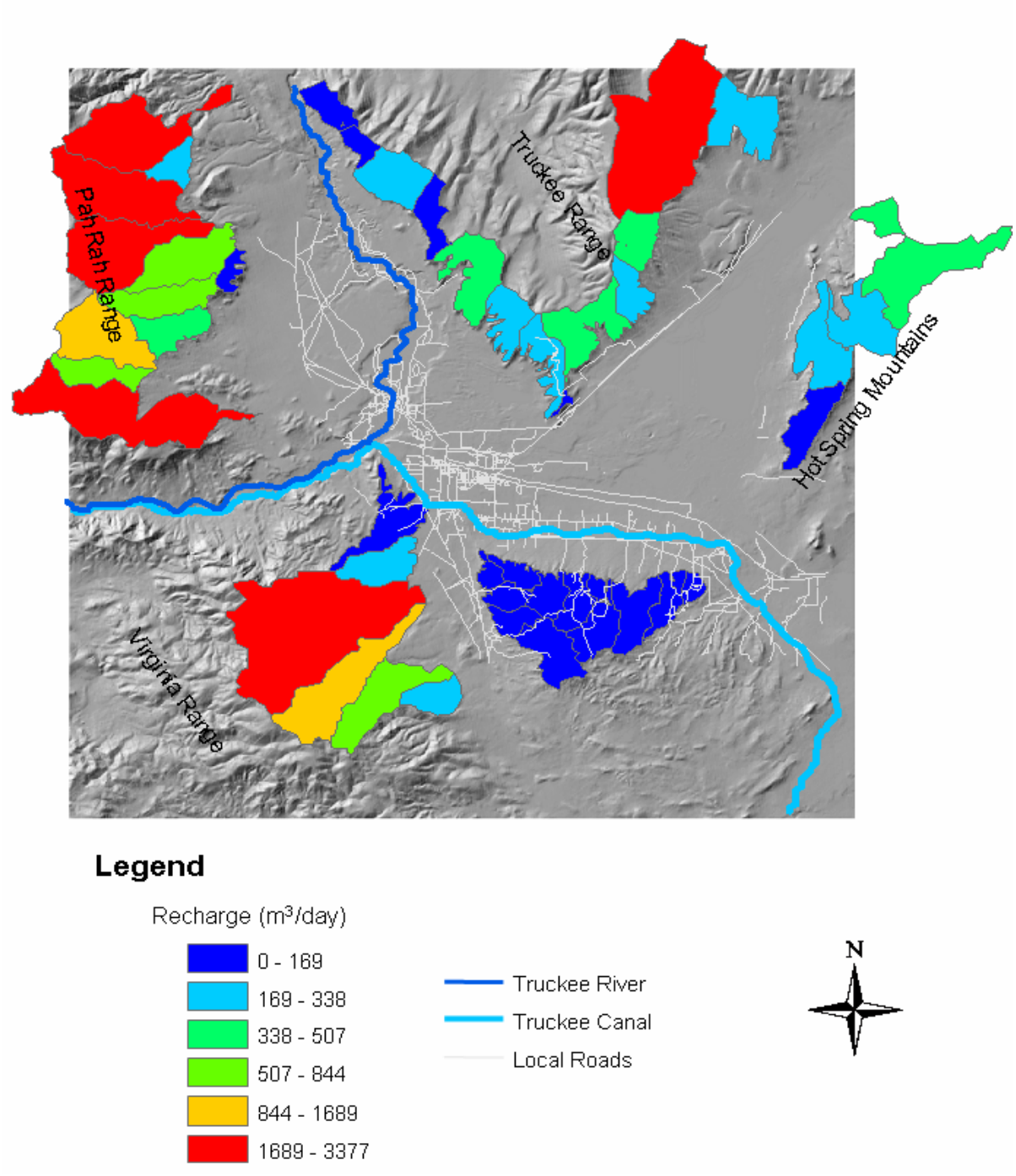



Figure 7. Mountain block recharge rates in the Fernley and Dodge Flat hydrographic basins.

Truckee River and Truckee Canal

In previous versions of the Fernley/Wadsworth groundwater model, the Truckee Canal was treated as an internal specified flux boundary. Seepage flux for the Truckee Canal was estimated using a numerical model that simulates water and energy flux in a two-dimensional, variably saturated domain, and a nonlinear parameter estimation code used to optimize horizontal hydraulic conductivity and anisotropy of the canal bed (Mihevc *et al*.,

2002) (Figure 4). Truckee Canal losses were approximately 38,000 $m^3$/day (10.0 mgd; 11,252 afy) within the study area (Mihevc *et al*., 2002). These estimates assume that the canal is always in operation. These estimates were then extrapolated to the entire canal section within the model domain to achieve a total rate of 42,000 $m^3$/day (11.1 mgd; 12,436 afy), which accounts for the concrete-lined sections located in the western portion of the model area.

In previous versions of the Fernley/Wadsworth groundwater model, the Truckee River was simulated as a general head boundary using the River (RIV) package. This study uses the SFR2 package to simulate the movement of water between the aquifer and the Truckee River and Truckee Canal.

The new MODFLOW package SFR2 (Niswonger and Prudic, 2005), allows simulation of unsaturated flow beneath surface-water bodies. The SFR2 package uses a kinematic-wave approximation to Richards' equation to simulate unsaturated flow beneath streams. Unsaturated flow is simulated independently of saturated flow within each model cell, when the water table is below the elevation of the surface-water body. This new option is important for simulation of delayed recharge response, as in the case of the Truckee Canal.

Since the SFR2 package directly simulates the seepage flux within MODFLOW, one cannot directly specify the flux rates. Instead, the hydraulic conductivities and unsaturated zone parameters (Mihevc *et al*., 2002) are used in the SFR2 package. Minor modifications of the hydraulic parameters were required such that the simulated leakage rates under flowing canal conditions were in agreement with their seepage rates. The bed hydraulic conductivities ranged between 0.2 and 0.4 m/day (0.66 to 1.31 ft/day) for the Truckee River and 0.1 to 0.5 m/day (0.33 to 1.64 ft/day) for the Truckee Canal. The model uses the Brooks and Corey (1966) equation to relate unsaturated zone hydraulic conductivity to water content. A value of 3.5 was used for the Brooks-Corey exponent for all reaches. The saturated water content was set to 0.30 for all reaches. The SFR2 package calculates residual water content based on the difference between saturated water content and specific yield. Because the equations used to calculated fluid movement in the unsaturated zone require that the residual water content is nonzero, the specific yield was set to 0.25, which produces a residual water content of 0.05.

The SFR2 package requires a number of parameters to calculate surface flow and seepage. Geometric parameters define the topology of the stream network and the streambed elevations for each reach. This information was generated using Groundwater Modeling Software (GMS), a MODFLOW preprocessor. Stream bed slope was calculated from a digital elevation model (DEM). Because the DEM did not have the appropriate resolution to capture the exact streambed elevation, minor modifications were made to the slope values to ensure that slope was always positive (i.e., in the direction of flow). A Manning's roughness coefficient was used to calculate a rating curve (flow versus depth) for each reach. The roughness coefficient was specified as 0.028 for the Truckee River and 0.025 for the Truckee Canal.

The calibration model input was U.S. Geological Survey (USGS) streamflow data from the Truckee River gage below Derby Dam and the Truckee Canal gage near Wadsworth. For the predictive simulations, Truckee River and Truckee Canal flow rates were based on simulations produced by Stetson Engineers, which used historic Truckee River streamflow data and OCAP requirements to move the specified amount of water

through the system, and divert the appropriate amount of water into the Truckee Canal (Stetson Engineers, written communication, 2006). This was done to predict reasonable flow rates based on how the Truckee River system will be managed under the current arrangement. Water years 1901 to 2000 were simulated. As discussed, 20 year periods of the record were used for each simulation. Figure 8 shows Truckee River and Truckee Canal reaches used as input for the SFR2 package.

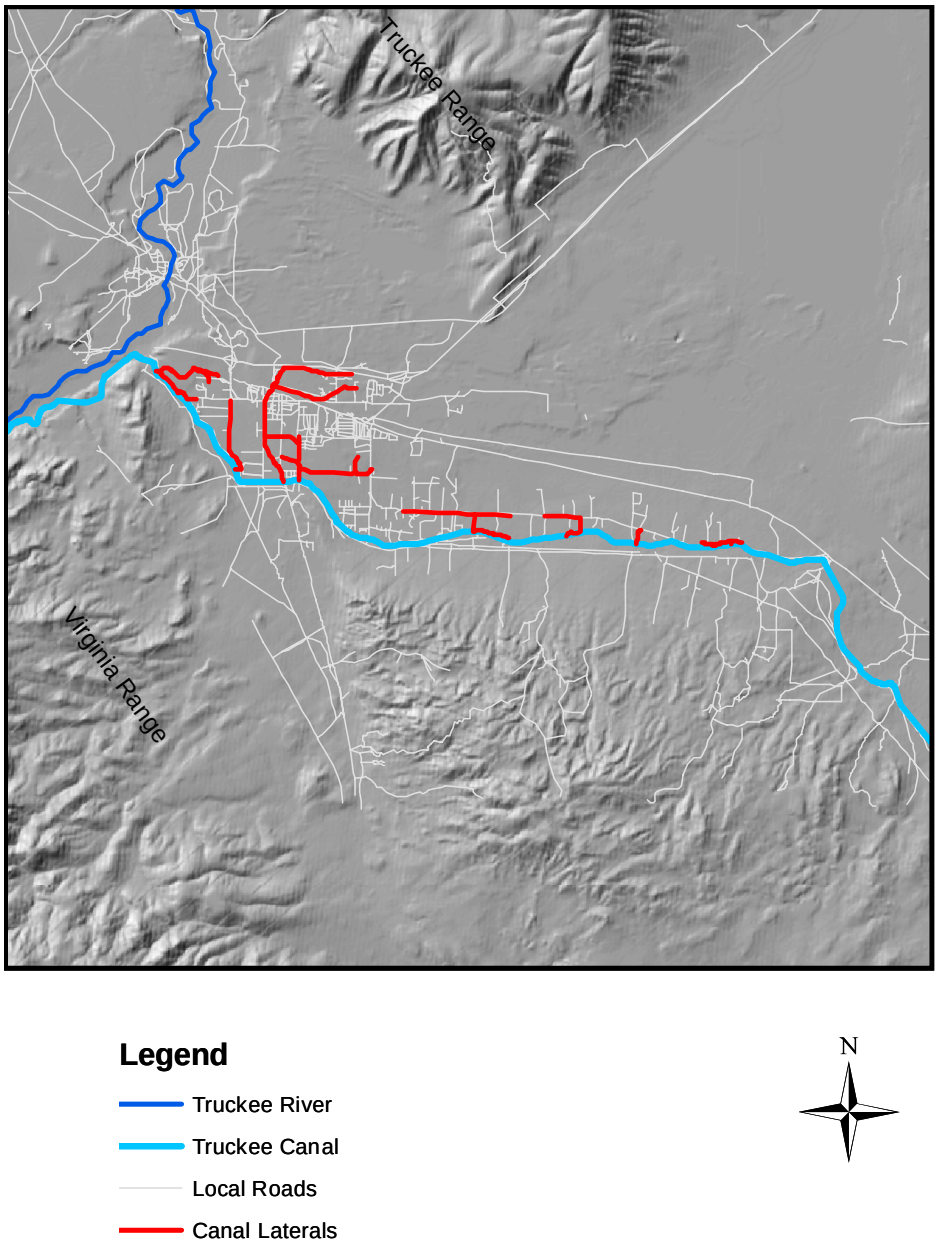



Figure 8. Location of Truckee Canal laterals.

Truckee Canal Laterals

Lateral seepage was calculated from literature values. In 1989, the amount of water diverted from the Truckee Canal to the Fernley area was reported as 43,200 $m^3$/day (11.4 mgd; 12,792 afy) (U.S. Bureau of Reclamation [USBR], 1994). The conveyance efficiency of the laterals is 68 to 75 percent (J. Lively, U.S. Bureau of Reclamation, personal communication, 2006). The high and low end of the conveyance efficiency, 28 to 32 percent, was used as the range for modeled lateral seepage. Based on the above calculations, the target range for the transient calibration was 12,100 to 13,800 $m^3$/day (3.20 to 3.65 mgd; 3,583 to 4,086 afy). These target values were used for the predictive simulations. When the Truckee Canal was not in operation, the laterals were not allowed to recharge the groundwater system. The locations of the lateral canals are shown in Figure 8. The lateral canals were simulated via injection wells (specified flux) rather than the SFR2 package as the individual canal diversion rates were not known.

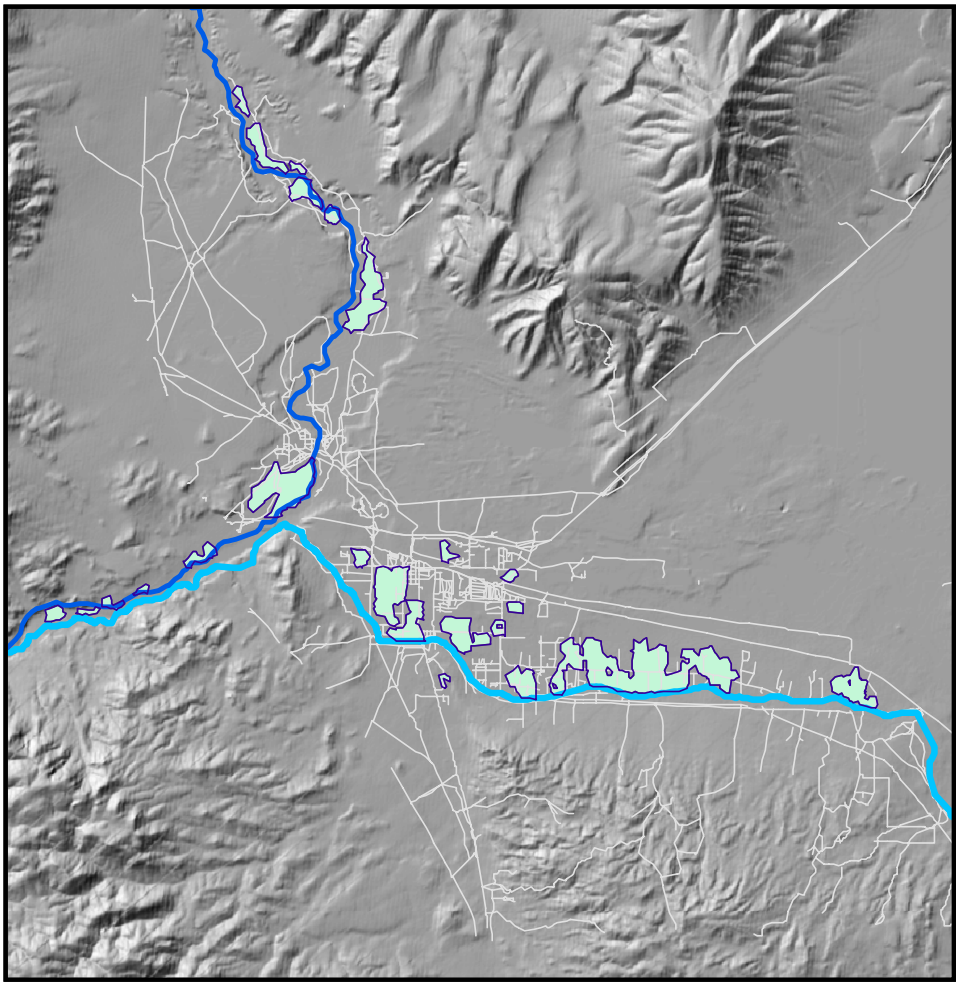
Irrigation Recharge

Irrigation recharge rates were taken from Pohll (2004) and modified to account for the decrease in irrigated area over the last several years. Pohll (2004) estimated that approximately 20.8 $km^2$ (8.03 $mi^2$) are irrigated, however, that area has decreased as a result of increasing development. An initial estimate of the irrigated area was determined by applying the Southwest Regional Landcover Data (USGS, 2004), which is representative of conditions in 2000. The irrigated areas which had been converted to developed areas between 2000 and 2004 were then eliminated using 4-foot-resolution digital orthophotography acquired in 2004-2005, which was provided by Washoe County Department of Water Resources. Using this approach, the irrigated acreage in 2004-2005 was found to be approximately 11.3 $km^2$ (4.36 $mi^2$). When the irrigation regions are mapped to the MODFLOW grid, the effective area of irrigation recharge is 21.4 $km^2$ (8.26 $mi^2$). Since recharge is input into MODFLOW as a rate (L/T) and is then internally multiplied by the cell area, the recharge rate input into MODFLOW had to be decreased by 52 percent.

Unsaturated zone modeling performed by Pohll *et al*. (2001) estimated that irrigation recharge rates ranged from 30 to 90 cm/yr (0.98 to 2.95 ft/yr). During the calibration of the steady-state flow model, it was found that recharge rates on the higher end of this range did not produce acceptable results. Therefore, it was assumed that the recharge rate was 30 cm/yr (0.98 ft/yr) for the agricultural areas. As noted above, this rate had to be decreased by 52 percent for input into the model to account for the inaccurate mapping of the recharge areas. The total annual recharge volume from agricultural irrigation is 8,500 $m^3$ (2.25 million gallons; 6.89 acre-feet) applied to the areas shown in Figure 9. Irrigation recharge remained constant in all of the transient simulations, regardless of Truckee Canal operations.

Groundwater Pumping

Groundwater withdrawals from the City of Fernley municipal system, Wadsworth area developments, and the Nevada Cement Company were included as internal specified flux boundaries using the WEL package. Average production rates (2000-2005) were used for the steady-state calibration simulation. Actual production records were used to apply water withdrawals to each well during each monthly stress period of the transient calibration. The predictive model is based on projections from the City of Fernley 2005 Water Master Plan (Vpoint, 2005). The locations of all pumping wells are shown in Figure 10.



**Legend**

- Truckee River
- Truckee Canal
- Local Roads
- Agricultural Recharge

N

Figure 9. Location of agricultural recharge.

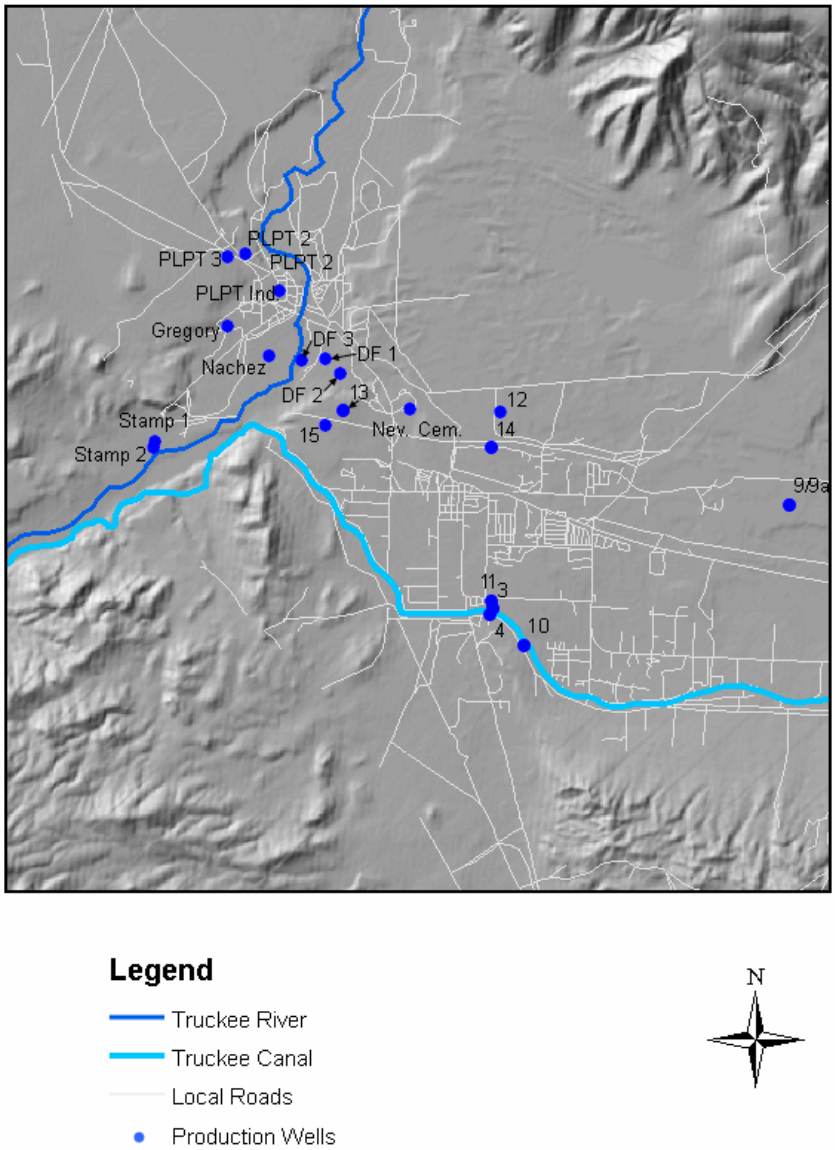



Figure 10. Pumping well locations.

The Fernley municipal system (wells 4, 9 and 11) was assumed to produce water at an average rate of 28,335 $m^3$/day (7.5 mgd; 8,390 afy) for the steady-state calibration (2000-2005). The Wadsworth wells were assumed to produce water at an annual rate at 1,100 $m^3$/day (0.29 mgd; 326 afy), based on weekly pumping records. An average annual rate of 6,800 $m^3$/day (1.8 mgd; 2,014 afy) was assumed for the pumping at Nevada Cement, based on permitted water right as determined by the State Engineer.

The monthly production well rates for the transient calibration simulation (2000-2005) are shown in Figure 11. Monthly pumping records were available for the Fernley wells and the wells located in the Wadsworth area that are controlled by Washoe

County. Detailed pumping records were not available for the Pyramid Lake Paiute Tribe wells (PLPT-Industrial, PLPT-2, PLPT-3, and Nachez), so the monthly rates were estimated such that the annual signal was in general agreement with the Washoe County wells and the total annual pumping from all of the Wadsworth wells was at 1,100 $m^3$/day (0.29 mgd; 326 afy). Monthly pumping rates were not available for the Nevada Cement well, so the full permitted value of 6,800 $m^3$/day was used throughout the transient calibration simulation.

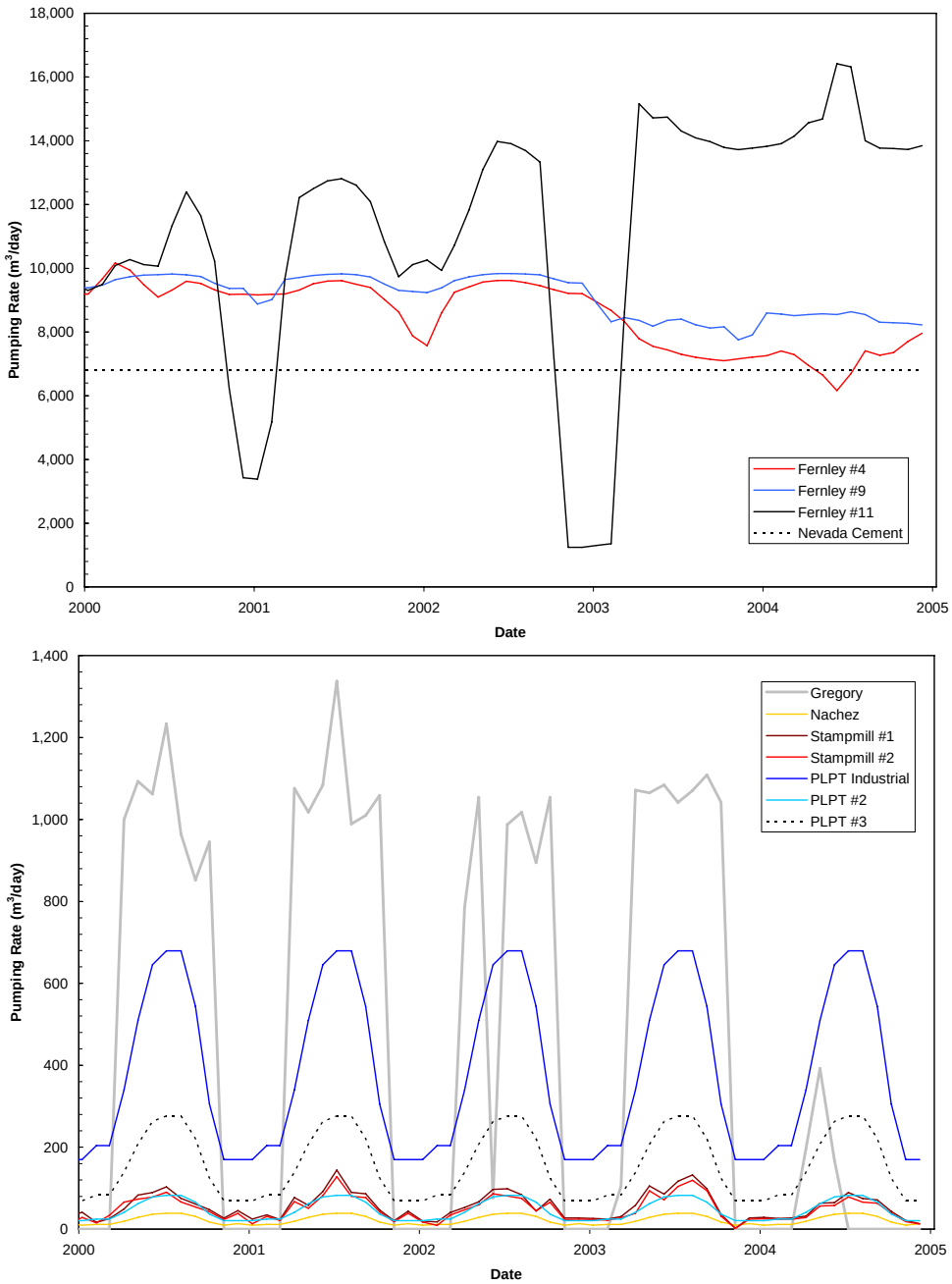



Figure 11. (a) Fernley municipal system and Nevada Cement well production rates for the transient calibration period, (b) Wadsworth area well production rates for the transient calibration period.

The transient prediction simulation required the projection of the pumping rates into the future. Projections were done for the Fernley and Wadsworth wells, but not for the Nevada Cement well since the full permitted value was assumed. The projected pumping rates are shown in Figure 12.

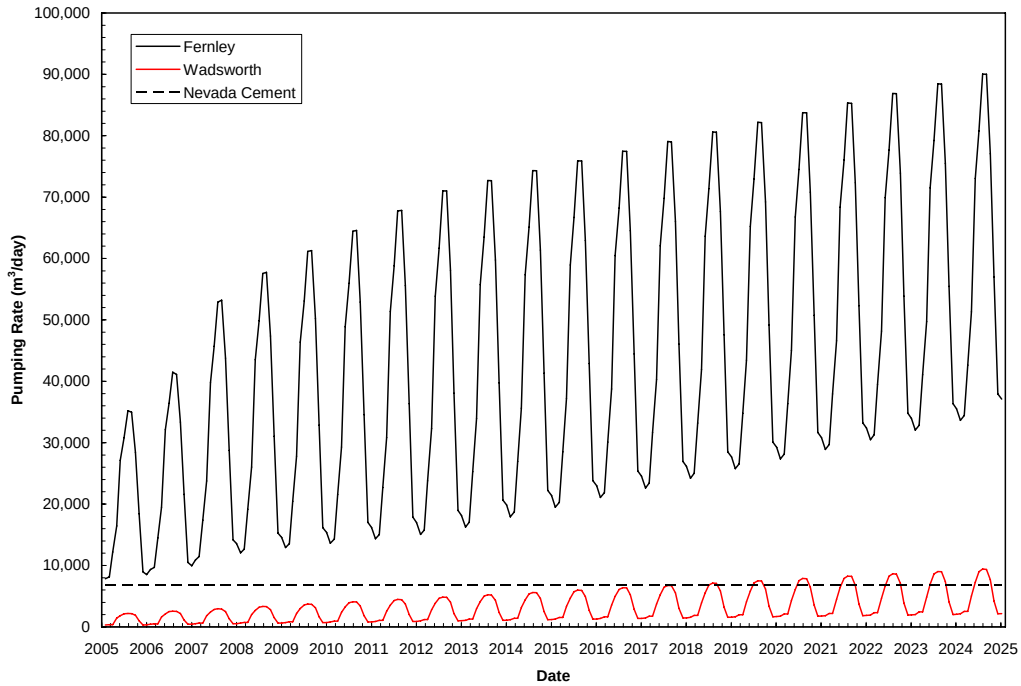



Figure 12. Total groundwater pumping for the Fernley, Wadsworth, and Nevada Cement wells.

Pumping rates in the Fernley area have been projected into the future as outlined in the Fernley Water Master Plan (Vpoint, 2005). Pumping was increased in existing wells (3, 4, 9/9a, 11, and 13) and then new wells (12, 14, and 15) were turned on as the plan suggested. Fernley currently uses groundwater as the exclusive source for its municipal system. According to the City of Fernley 2005 Water Master Plan (Vpoint, 2005), the State Engineer has limited the City's annual municipal groundwater pumping right to approximately 10.98 million $m^3$ (2,900 million gallons; 8,901 acre-feet) (Vpoint, 2005). Part of Fernley's plan is dedicated to water development through the year 2025. The plan indicates that Fernley will be looking to Truckee Canal Irrigation District water as an additional source to meet the growing demands of its municipal system. This possibility is not simulated in this study. When average annual water demand exceeds the permitted and deeded municipal supply, the balance will be withdrawn from the Wadsworth area via a hypothetical well field near where the 2005 pumping test occurred. This is estimated to occur around 2013. In an effort to simulate the seasonal fluctuations within the City of Fernley production wells for the predictive simulations, the monthly rates at individual wells were multiplied by the typical monthly deviation from the yearly average. This has the effect of producing higher rates in the summer months and lower rates in the winter months. The multiplication factors for each month are given in Table 3.

Table 3. Multiplication factors used to simulate seasonal fluctuations in the City of Fernley municipal wells.

| Month | Multiplication Factor |
|---|---|
| 1/1/2006 | 0.39 |
| 2/1/2006 | 0.40 |
| 3/1/2006 | 0.61 |
| 4/1/2006 | 0.82 |
| 5/1/2006 | 1.36 |
| 6/1/2006 | 1.55 |
| 7/1/2006 | 1.78 |
| 8/1/2006 | 1.78 |
| 9/1/2006 | 1.45 |
| 10/1/2006 | 0.95 |
| 11/1/2006 | 0.46 |
| 12/1/2006 | 0.44 |

In the Wadsworth area, the current total pumping rate of 1,100 $m^3$/day (0.29 mgd; 326 afy) was used for the steady-state and transient calibration simulations. Wadsworth municipal pumping has been projected into the future based on Wadsworth's master plan total area rates (Stetson Engineers, 2003). This was done by linearly interpolating from the current amount to the projected amount for the year 2025, approximately 5,300 $m^3$/day (1.40 mgd; 1,569 afy). Lacking individual well production records for the Wadsworth area, total pumping was divided among the known production wells.

Again, the Nevada Cement Company production rates were estimated based on water rights. A constant production rate of 6,800 $m^3$/day (1.80 mgd; 2,013 afy) was used for the Nevada Cement Company wells for the transient predictive simulations.

Evapotranspiration Rates

The Southwest Regional Landcover Data set (USGS, 2004) was applied to estimate different ET zones for greasewood, cottonwood, and playa areas (Figure 13). MODFLOW simulates ET based on a specified maximum potential ET rate, extinction depth, and land surface elevation. The ET rate is calculated based on a linear relationship between the simulated depth to water and the maximum potential ET rate. As the simulated depth to water increases, the simulated ET rate decreases. When the simulated depth to water increases beyond the extinction depth, ET ceases at that location in the model.

Literature values for groundwater ET rates were used to parameterize the ET package for each ET zone (White, 1932; Robinson, 1958). The maximum groundwater ET rates were $5.0 \times 10^{-4}$, $5.0 \times 10^{-3}$, and $5.0 \times 10^{-3}$ m/day ($1.6 \times 10^{-3}$, $1.6 \times 10^{-2}$, $1.6 \times 10^{-2}$ ft/day) for greasewood, playa, and cottonwood areas, respectively. These are maximum groundwater ET rates, which, in the model, are reduced based on the extinction depth. The extinction depths were specified as 7 m, 1 m, and 5 m (23, 3.3, and 16 ft) for greasewood, playa, and cottonwood areas, respectively. The land surface elevation was obtained from a 30-m (98-ft) DEM. Because the mapping of the irregular ET zones to the MODFLOW grid results in an artificially increased area as seen by MODFLOW, the rates had to be decreased for input into MODFLOW in a manner similar to the irrigation recharge as discussed above.

The steady-state model predicts approximately 13,800 $m^3$/day (3.65 mgd; 4,086 afy) of ET throughout the study area. Although this value is slightly smaller than the range of previous estimates (14,000 to 26,000 $m^3$/day; 3.70 to 6.87 mgd; 4,145 to 7,699 afy), the current model does not encompass the entire playa area located east of Fernley. Since the current model covers approximately 67 percent of the playa area, and approximately 75 percent of the simulated ET is derived from the open playa, it is likely that groundwater actually supplies 15,700 $m^3$/day (4.15 mgd; 4,649 afy) of ET in this area. Assuming that 100 percent of the Fernley treatment plant discharge (6,700 $m^3$/day; 1.77 mgd; 1,983 afy) contributes to playa ET, then the total ET rate could be as large as 22,500 $m^3$/day (5.94 mgd; 6,662 afy), which is within the range of previous estimates.

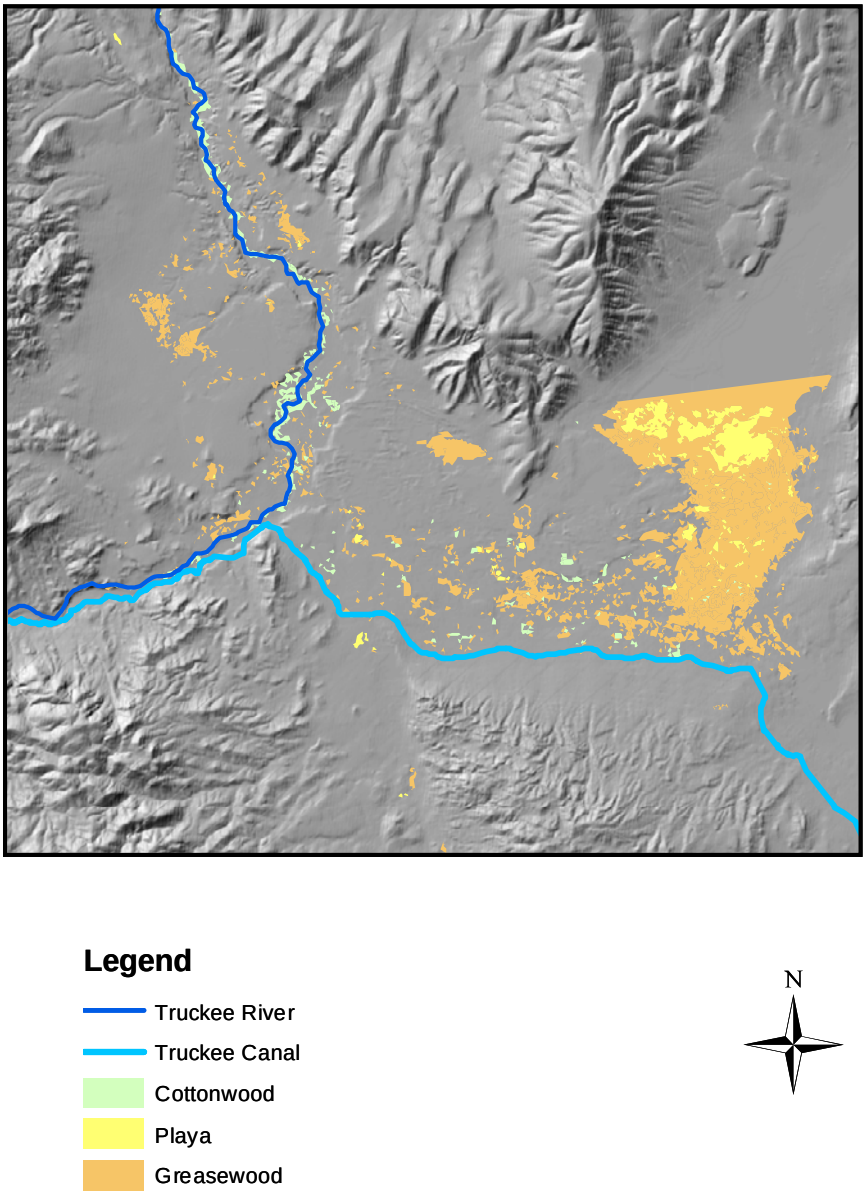



Figure 13. Location of evapotranspiration areas.

## Hydraulic Head Data

The spatial coverage of hydraulic head data within the model domain is limited. Likewise, there is a lack of deep wells to fully characterize the hydraulic head distribution within the deeper sediments. Static water levels from 62 wells were used to calibrate the steady-state model (Figure 14). The majority of these data were collected during the period 2000-2005. Only a subset of these data was useful for the transient model calibration, as temporal head observations were limited. Forty-two wells were used for the transient calibration, but for many of these wells only a few temporal measurements were available. Most of the water level measurements are limited to the upper 60 m (197 ft). Because of the limited number of temporal water level measurements, the transient model relies heavily on the steady-state calibration to parameterize the hydraulic properties of the local aquifer system.

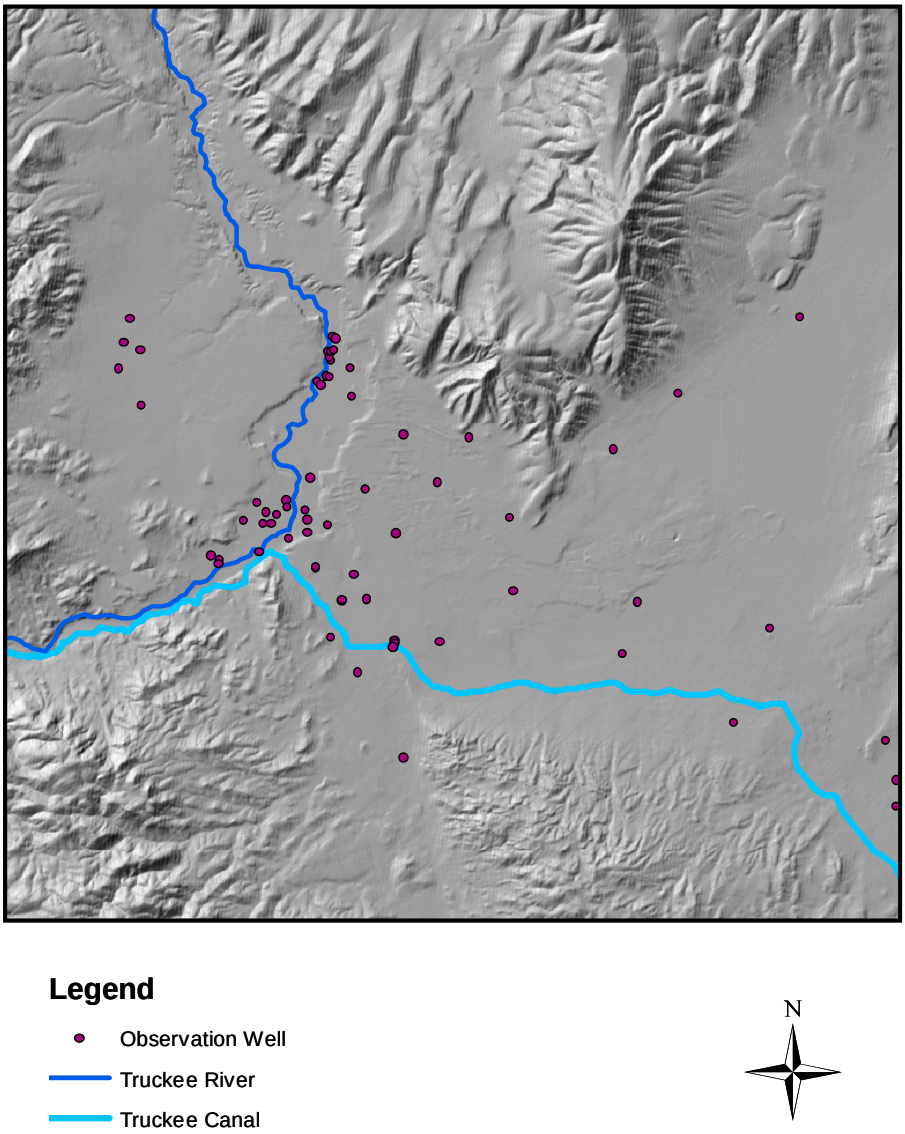



Figure 14. Location of hydraulic head observation wells.

## Hydraulic Conductivity

There are a total of 25 hydraulic conductivity measurements within the study area (Pohll *et al*., 2001). Hydraulic testing included pumping, recovery, and packer testing and was performed from 1997 to 2006. Most of the measurements are limited to the upper 60 m (197 ft) of the aquifer, with the exception being the TOF 2 well, where packer tests were performed at discrete intervals to a depth of 293 m (691 ft). The hydraulic measurements are summarized in Table 4. The arithmetic mean of the 25 measurements is 21 m/day (69 ft/day) and the geometric mean is 6 m/day (20 ft/day).

Table 4. Hydraulic conductivity values measured within the Fernley-Wadsworth area.

| | UTM – NAD 83 – Zone 11 | | | |
|---|---|---|---|---|
| Name | Easting | Northing | Type | K (m/day) |
| 1 | 305968 | 4384033 | Pumping | 2.55E+02 |
| 4 | 304718 | 4387235 | Pumping | 9.91E+00 |
| 5 | 305910 | 4387029 | Pumping | 6.09E+00 |
| 6 | 305542 | 4386207 | Recovery | 7.00E+01 |
| 7 | 306308 | 4386242 | Pumping | 2.53E+01 |
| 8 | 308558 | 4384937 | Pumping | 6.10E+01 |
| 9 | 310718 | 4388779 | Recovery | 2.72E+01 |
| 10 | 308466 | 4389880 | Pumping | 2.38E+01 |
| 11 | 307201 | 4388270 | Pumping | 3.76E+00 |
| 12 | 306258 | 4389669 | Recovery | 8.12E+00 |
| 14 | 307433 | 4391354 | Pumping | 1.37E+01 |
| 15 | 305845 | 4392531 | Pumping | 5.46E+00 |
| 17 | 303876 | 4388133 | Pumping | 1.10E+01 |
| BB Irrigation | 304080 | 4388498 | Pumping | 1.12E+01 |
| Crosby | 304475 | 4388706 | Pumping | 2.50E+01 |
| DP Irrigation | 302505 | 4388679 | Pumping | 9.00E+00 |
| TOF 1 | 307434 | 4381325 | Pumping | 1.60E+01 |
| TOF 2 | 307208 | 4388290 | Packer | 1.73E+00 |
| TOF 2 | 307208 | 4388290 | Packer | 9.50E+00 |
| TOF 2 | 307208 | 4388290 | Packer | 1.73E+01 |
| TOF 2 | 307208 | 4388290 | Packer | 4.32E+01 |
| TOF 2 | 307208 | 4388290 | Packer | 6.74E+00 |
| TOF 2 | 307208 | 4388290 | Packer | 8.60E+01 |
| TOF 3 | 309457 | 4391263 | Pumping | 2.68E+01 |
| TW 5 | 304691 | 4388489 | Pumping | 2.70E+00 |

Hydraulic conductivities in the area span nearly four orders of magnitude (0.11 to 254 m/day; 0.36 to 833 ft/day). The spatial distribution of hydraulic conductivity is highly heterogeneous and there is no discernible pattern across the basin (Figure 15). The highest hydraulic conductivity (254 m/day; 833 ft/day) was measured at well 1, but it may be biased high due to its proximity to the Truckee Canal. Interestingly, the lowest measured hydraulic conductivity is at well 17, which is located adjacent to the Truckee River, where other pumping tests indicate higher conductivity values.

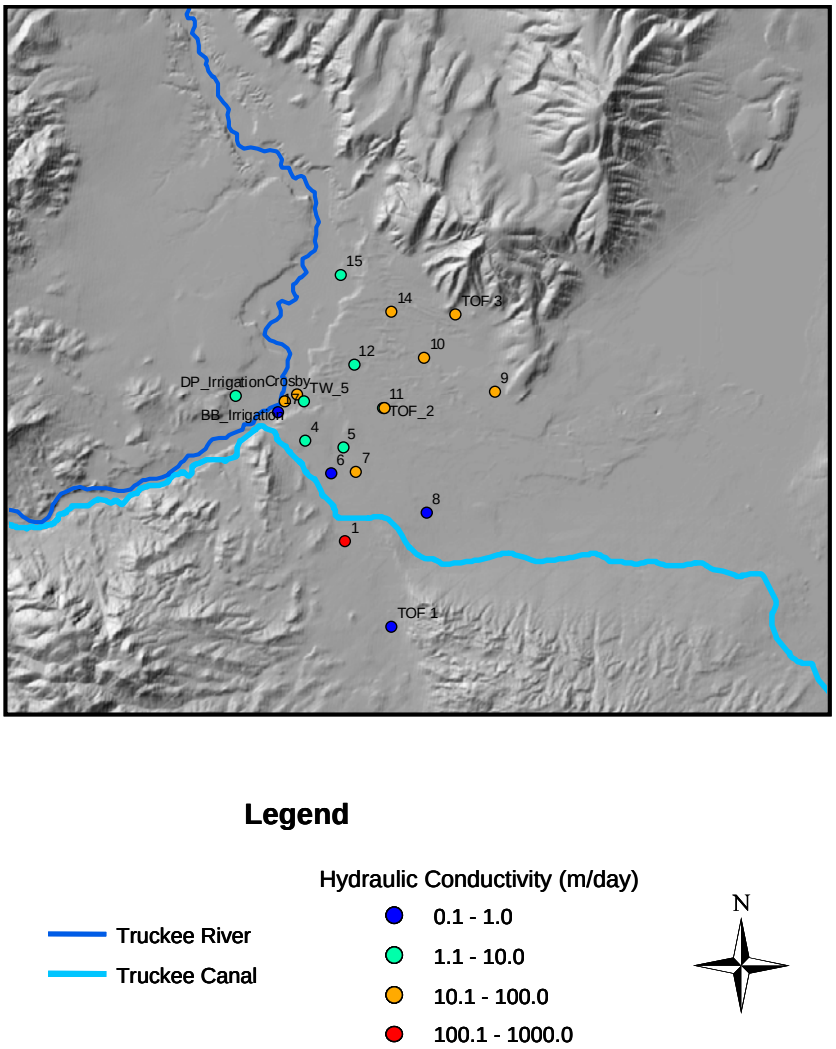



Figure 15. Hydraulic testing in the Fernley/Wadsworth area.

## Scope of Simulations

One of the primary purposes of this study is to determine the impact of increased pumping on both the Truckee River and the regional aquifer. During the January 13, 2006, meeting of the technical advisory team, two transient simulations were proposed to address the objectives of this study: a six-year (2000 to 2005) calibration simulation, and a 20-year (2006 to 2025) predictive simulation that includes projected Fernley and Wadsworth groundwater pumping. The two sections below discuss the simulations in detail.

### Calibration Simulation

The calibration simulation covers a six-year period from January 1, 2000 to December 31, 2005. The simulation was run using:

- 72, one-month stress periods
- Actual pumping rates for the City of Fernley municipal wells and major production wells in the Wadsworth area

- Industrial pumping (i.e., Nevada Cement Company) at permitted values
- Actual Truckee River and Truckee Canal flows
- Irrigation canals (i.e., laterals), based on the Truckee Division diversion as stated in the USBR report to Congress (USBR, 1994) and 68 percent to 75 percent delivery efficiency (J. Lively, U.S. Bureau of Reclamation, personal communication, 2006)
- Infiltration from irrigation, using a fixed irrigated area: the effective rate was reduced over the six-year period based on changes in irrigated acreage determined by aerial photography from 2000 and 2005
- Natural recharge, scaled linearly according to annual precipitation at the Reno airport because recent Fernley precipitation records were not available
- Evaporation estimates, taken from VanDenburgh and Artega (1985)
- Transpiration, based on literature values of predominant plant types; areas are from a geographic information system (GIS) plant cover classification
- The simulation results were compared to all available hydraulic head data. Only minor adjustments were made to the aquifer storage parameters (specific storage and specific yield) to achieve an agreement between simulated and measured values. A specific yield of 0.25 was used for layers 1 through 3 (convertible), which is consistent with alluvial aquifer systems. A specific storage of $10^{-6}$ ($m^{-1}$) was used for all layers to represent the confined aquifer storage.

Predictive Simulation

The 20-year predictive simulation begins on January 1, 2006, and ends on December 31, 2025. The items included in this simulation are:

- 240, one-month stress periods, with four time steps per stress period
- Pumping rates in the Fernley area, projected into the future as outlined in the 2005 City of Fernley Water Supply Master Plan (Vpoint, 2004). Once the projected groundwater pumping reached the permitted value, the excess was withdrawn from the Wadsworth area via a hypothetical well field near where the 2005 pumping test occurred.
- Wadsworth domestic pumping, projected into the future based on the 2003 Draft Wadsworth Water System Master Plan (Stetson, 2003)
- Truckee River and Truckee Canal flows, based on historical flows, and adjusted according to OCAP guidelines as provided by Stetson Engineers (Peter Pyle, Stetson Engineers, written communication, 2006)
- Individual 20-year flows were extracted from the period of record. A total of five predictive simulations have been produced to reflect the following historical periods:
    - Simulation 1: January 1, 1901, to December 31, 1920
    - Simulation 2: January 1, 1921, to December 31, 1940
    - Simulation 3: January 1, 1941, to December 31, 1960
    - Simulation 4: January 1, 1961, to December 31, 1980

- Simulation 5: January 1, 1981, to December 31, 2000

- Irrigation canals (i.e., laterals): deliveries were based on Truckee Division diversions stated in the USBR report to Congress (USBR, 1994) and 68 percent to 75 percent delivery efficiency (J. Lively, U.S. Bureau of Reclamation, personal communication, 2006). Deliveries will not occur if water is not available in the Truckee Canal for a given model time step.
- Infiltration from irrigation, simulated by using a fixed irrigation area and rate as determined by 2005 aerial photos
- The average annual mountain front recharge, determined by using the USGS method developed by Maurer and Berger (1997) and expanded by Berger (2000), was specified at the mountain block-piedmont interface.

The simulations have been used to predict the following for each monthly time step:

- Groundwater levels and drawdown
- Flow direction
- Infiltration from the Truckee Canal
- Inflows and outflows along the Truckee River and associated flows at the Nixon gage
- Infiltration beneath agricultural fields
- ET, primarily from phreatophytes and open water evaporation

## MODEL CALIBRATION

### Pilot Point Method

The pilot point methodology (Doherty, 2003) was used to calibrate the hydraulic conductivity field. The basis of the pilot point method is to calibrate a reduced set of hydraulic conductivity points rather than at each grid element. Doing so significantly reduces the number of unknowns in the calibration process, while maintaining the appropriate level of spatial variability.

The pilot point methodology was only used to calibrate the hydraulic conductivity field for layers 1 through 6 in the model. The lower layers (7 through 10) were assigned a constant hydraulic conductivity of 0.01 m/day (0.033 ft/day), as there were not enough deep head data to calibrate a heterogeneous conductivity field. Therefore, layers 1 through 6 are heterogeneous and layers 7 through 10 are homogeneous.

The pilot point methodology can be outlined as follows:

1. Compile the locations and values for measured values of hydraulic conductivity.
2. Determine the locations of the pilot points, such that areas without hydraulic conductivity measurements will be included in the calibration process.
3. Assign initial guesses to the pilot points.
4. Use PEST (Doherty, 2002) to automatically adjust the hydraulic conductivity values at each of the pilot points such that a minimum difference between the simulated and measured hydraulic heads is obtained.

5. Interpolate the measured and pilot point hydraulic conductivity values using a logarithmic-based inverse distance weighting to populate all of the finite-difference grid cells in the model domain.
6. Continue the automated calibration process until there is a minimum error between the simulated and measured hydraulic head data.

The three-dimensional nature of the groundwater flow model requires that both measured and pilot point locations be given a vertical position. Since the hydraulic conductivity measurements are not conducted at a single point in space, but rather over a distance defined by the well's screened interval, each measurement was classified as being either shallow or deep. In the context of the flow model, shallow is defined as layers 1 through 3 (approximately 0 to 100 m; 0 to 328 ft) and deep as layers 4 through 6 (approximately 100 to 300 m; 328 to 984 ft). There are no measurements or pilot points for the lower layers (7 through 10) as there are no head data to constrain the calibration. Therefore, layers 7 through 10 in the model were assigned a constant hydraulic conductivity of 0.01 m/day (0.033 ft/day).

A total of 42 measured and pilot points were used in layers 1 through 3 and 18 in layers 4 through 6. Therefore, a total of 60 pilot points were used in the calibration, with the 25 measured values held constant during the calibration process (Figure 16). It is important to note that of the 25 measured hydraulic conductivity values, only one well (TOF-2) was located in the layer 4 through 6 region. The 62 hydraulic head measurements (locations shown in Figure 13) were used to calculate the error between the simulated and observed head values. Fifty-seven of these wells were located in layers 1 through 3, and 5 in layers 4 through 6.

**Steady-state Calibration**

The steady-state model calibration yielded an acceptable agreement between the simulated and observed hydraulic head values. The absolute model error, which is taken as the sum of the absolute difference between simulated and observed head values divided by the total number of head measurements was 4.2 m (14 ft). Another calibration metric, the normalized absolute error, utilizes the absolute error divided by the total head change. The normalized absolute error was 3.6 percent. Generally, models are considered acceptable if the normalized absolute error is below 5 percent.

Model bias can be further investigated by plotting the simulated versus observed head values (Figure 17). The calibrated model does exhibit some bias as the simulated versus observed values plot below the one-to-one line. The model tends to underpredict the hydraulic head in areas of higher observed hydraulic head.

The spatial pattern of the calibration residuals (observed minus simulated head) is shown in Figure 18. Warmer colors represent model underprediction and cooler colors represent overprediction. In general, there is no spatial pattern in the calibration residuals, which indicates there is no particular region where the model performs poorly. The large underprediction located just north of the Truckee Canal in the central part of the valley, is most likely related to a known fault that passes through the area, but was not simulated in the model. Another model underprediction is found in the southern portion of Dodge Flat. In this case, there are three wells in close proximity that have drastically different head values. This could be due in part to localized pumping or erroneous water level measurement. The model

is both underpredicting and overpredicting at two wells located north of the Truckee Canal in the eastern portion of the valley. The model error in this region could be related to large heterogeneities that are not properly modeled.

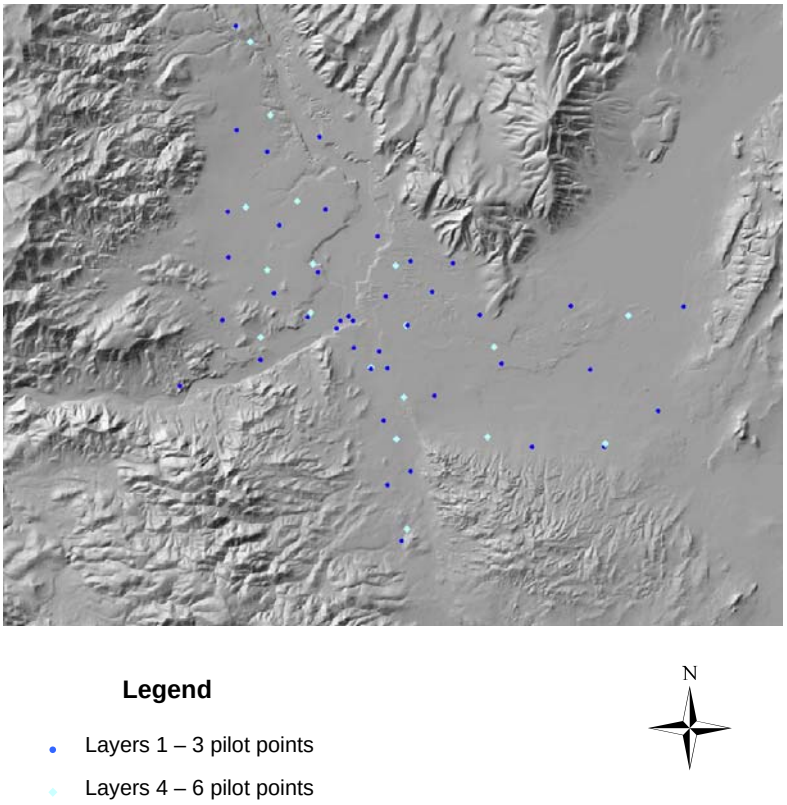



Figure 16. Pilot point locations used in the steady-state model.

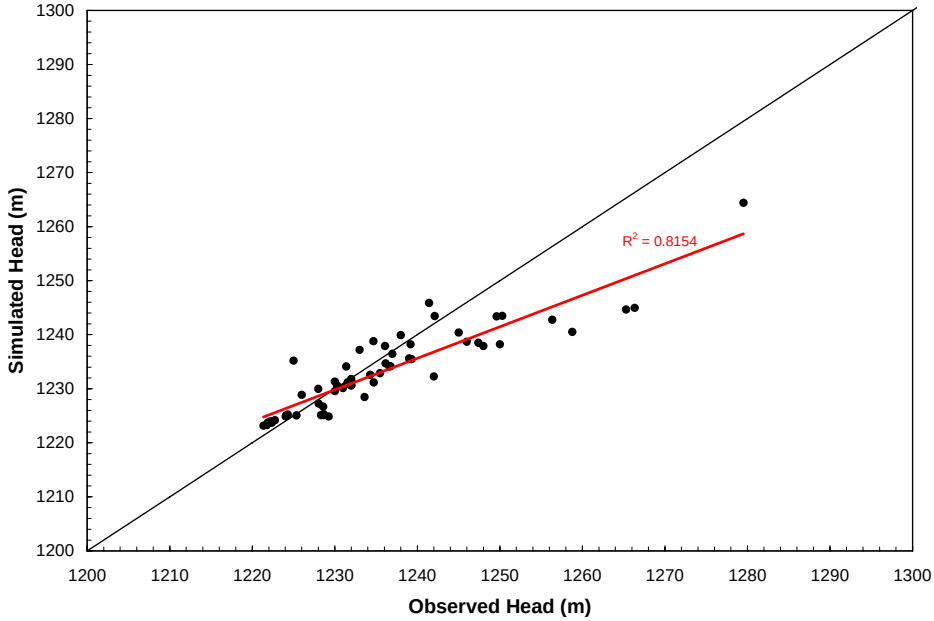



Figure 17. Simulated versus observed hydraulic head values for the steady-state calibration.

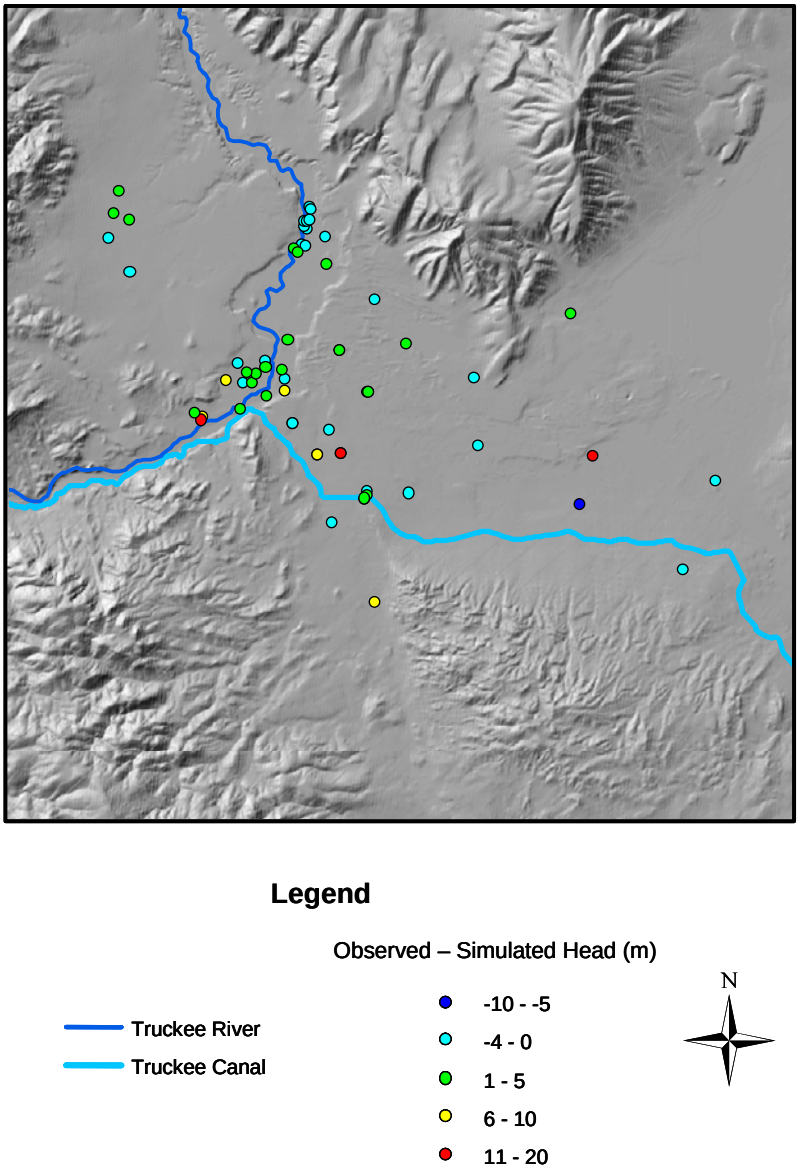



Figure 18. Steady-state calibration residuals, observed minus simulated hydraulic head.

The calibrated hydraulic conductivity fields are shown in Figures 19 and 20 for layers 1 through 3 and layers 4 through 6, respectively. The arithmetic mean is 10 m/day (33 ft/day) for layers 1 through 3 and 4 m/day (13 ft/day) for layers 4 through 6. In the shallower layers, the hydraulic conductivity is generally higher around the Truckee River, whereas in the lower layers the hydraulic conductivity is smaller adjacent to the Truckee River. Elsewhere in the model domain, the spatial distribution of hydraulic conductivity is highly heterogeneous. It should also be noted that in areas of limited hydraulic head measurements, the calibrated hydraulic conductivity may not be representative of actual conditions within the aquifer.

## Transient Calibration

The purpose of the transient calibration was to adjust the storage parameters (specific yield and specific storage) until a reasonable agreement was achieved between the simulated

and observed hydraulic heads during the January 1, 2000, to December 31, 2005, time period. This period was selected because it contains the largest amount of hydraulic head data. As noted above, there is a limited amount of head data available to calibrate the transient model. Therefore, a simple, manual adjustment of the storage parameters was used to yield similar trends between the simulated and observed heads. It is important to note that the hydraulic conductivity field determined in the steady-state calibration (i.e., pilot point methodology) was not adjusted in the transient calibration.

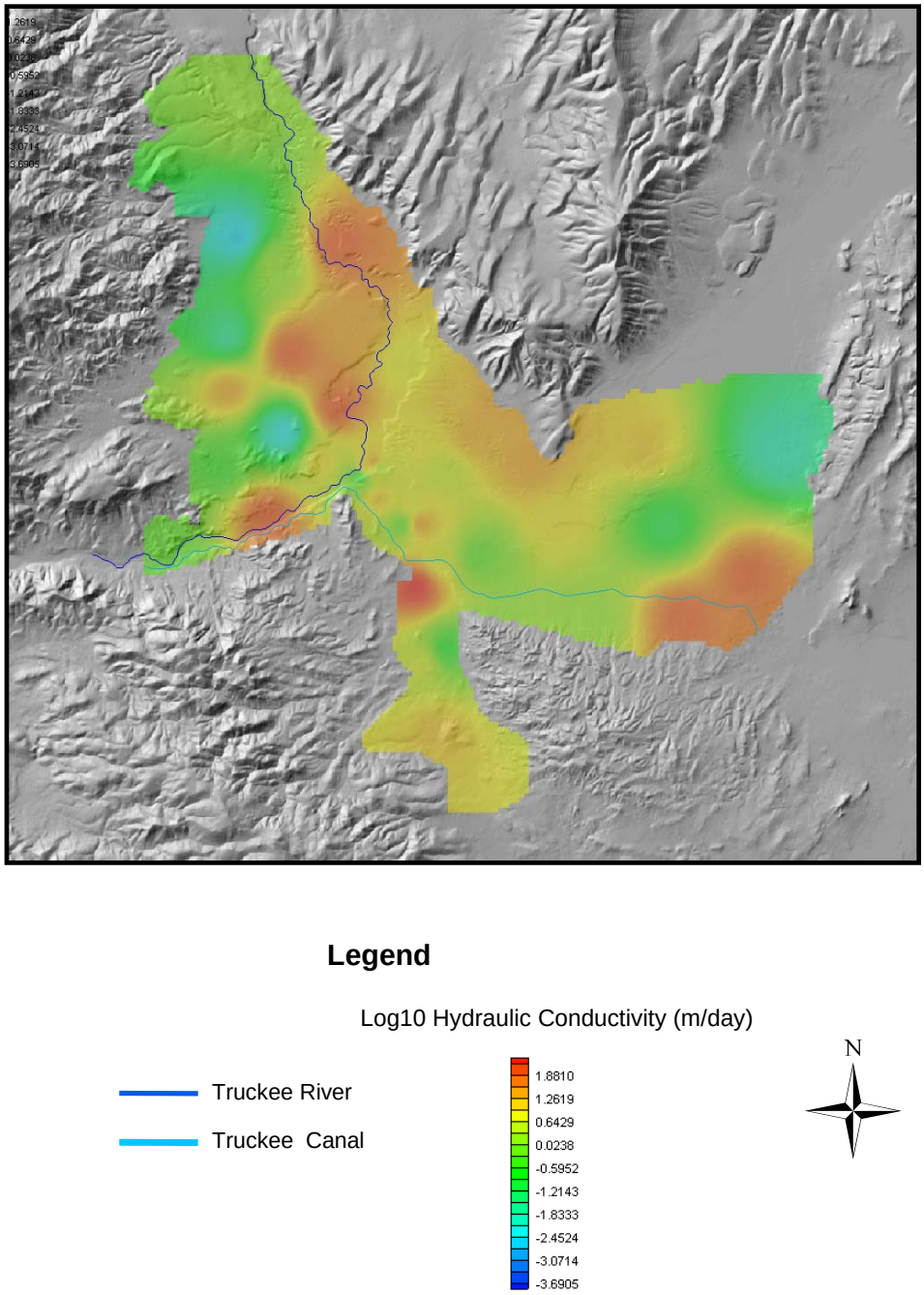



Figure 19. $Log_{10}$ hydraulic conductivity for layers 1 through 3.

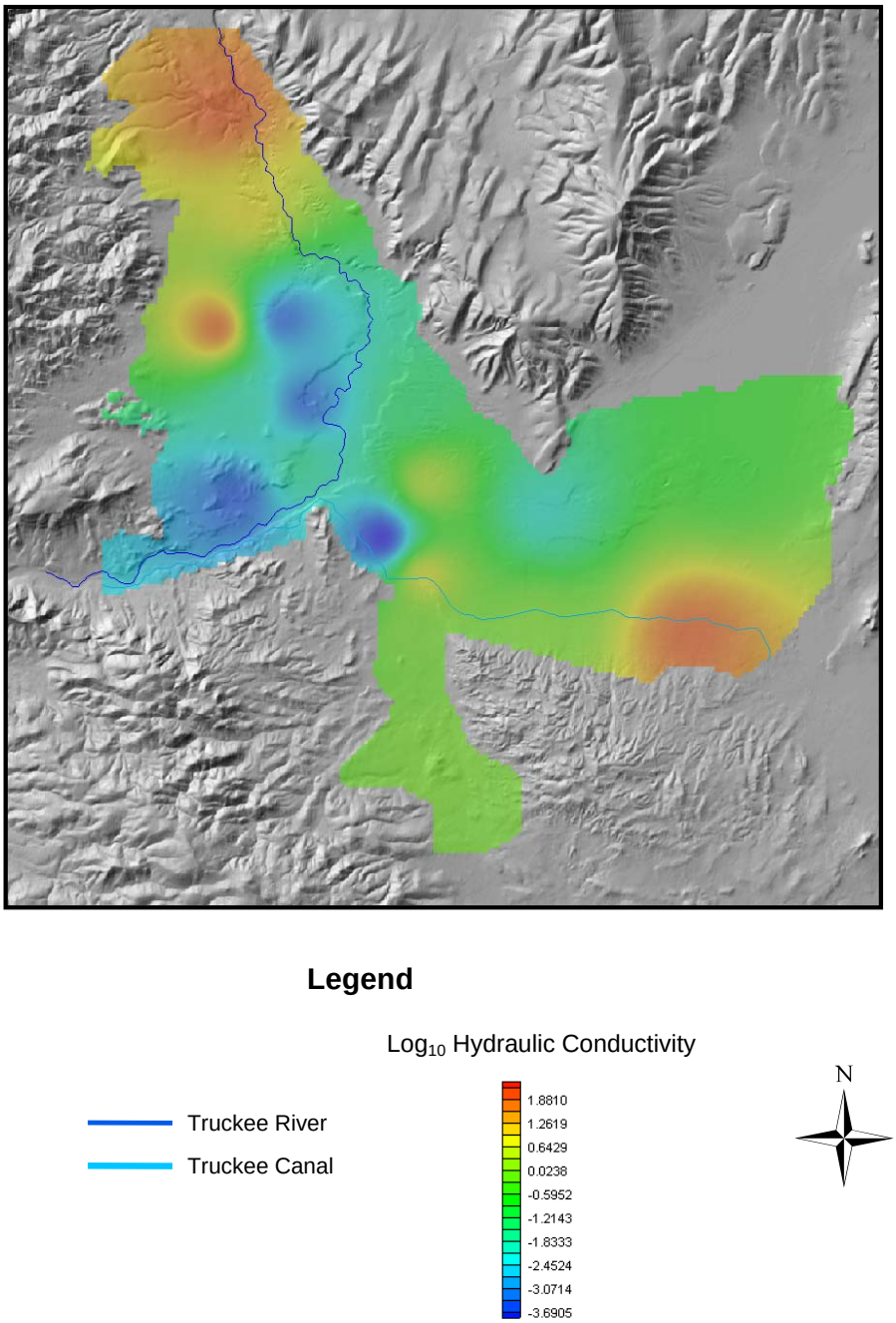



Figure 20. $Log_{10}$ hydraulic conductivity for layers 4 through 6.

A specific yield of 0.25 and specific storage of $10^{-6}$ $m^{-1}$ ($3.28 \times 10^{-6}$ $ft^{-1}$) were found to produce an acceptable agreement for the simulated and observed temporal variation in hydraulic head. These values are consistent with those reported in the literature for alluvial sediments (Freeze and Cherry, 1979).

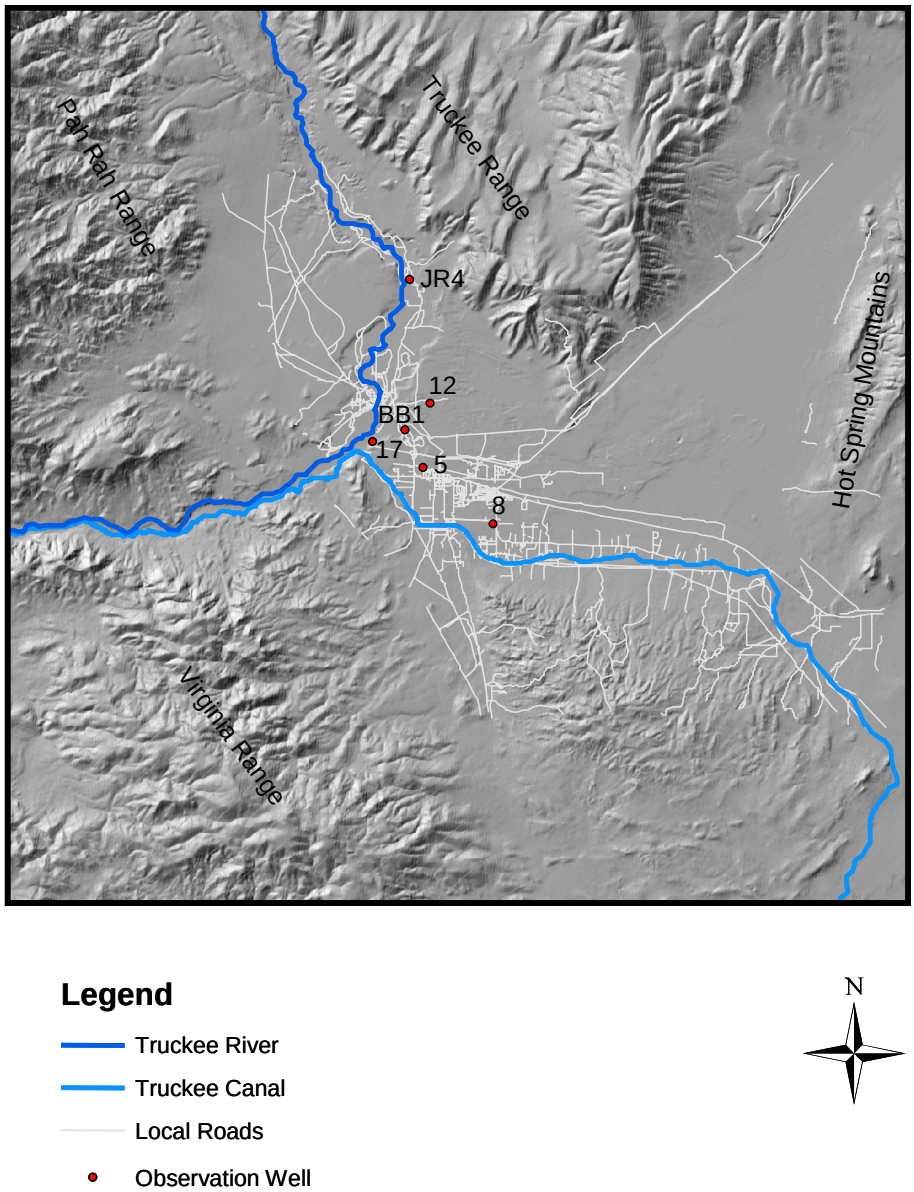
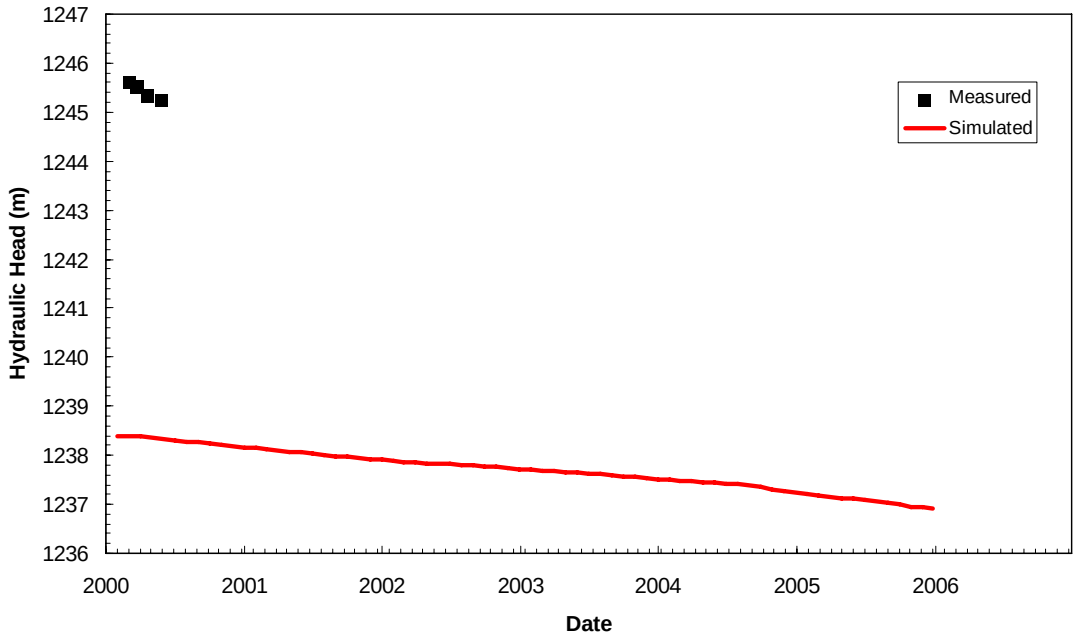



Figure 21. Location of observations wells used to compare simulated versus hydraulic head for the transient calibration.

Although 42 wells were used in the transient calibration, results from four representative wells are presented here (Figure 21). Plots of simulated versus observed hydraulic head for these four wells are shown in Figures 22 through 25 (wells 5, 12, 17, and BB1, respectively). Generally, there is a reasonable agreement in the trends between the simulated and observed hydraulic heads. As was noted in the steady-state calibration section, the overall magnitude between the model and measured heads can be in error, but if the trends are simulated correctly, then the transient nature of model can be deemed acceptable. The offset between the absolute magnitude between simulated and observed head is due to the errors inherent in the steady-state calibration. The general trend in well 5 is downward over the period of the simulation, which is reflected in the transient simulation. Both the

simulated and observed water levels at well 12 indicate water levels are decreasing moderately during this period. The model performs poorly at well 17, in which the data show a decreasing trend in 2000 and then increasing from mid-2000 to mid-2002. The model, however, simulates a decreasing trend until early in 2005, and a modest increase thereafter. Although the transient data are limited at BB1, the model predicts a downward trend in water levels, while the data suggest a slight increase in 2000.



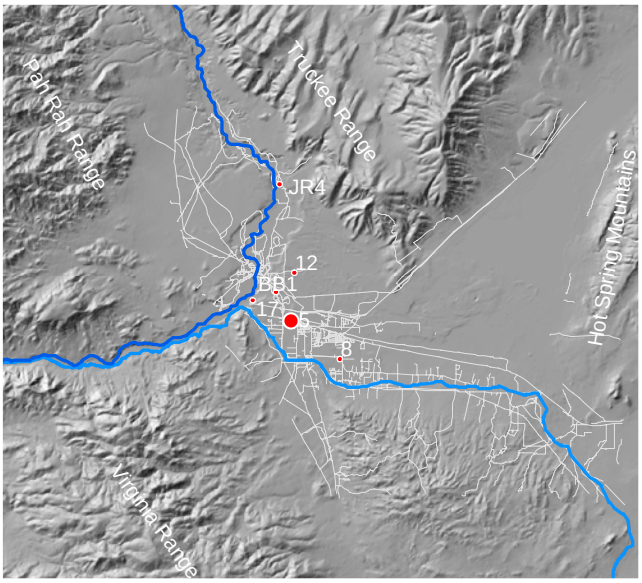
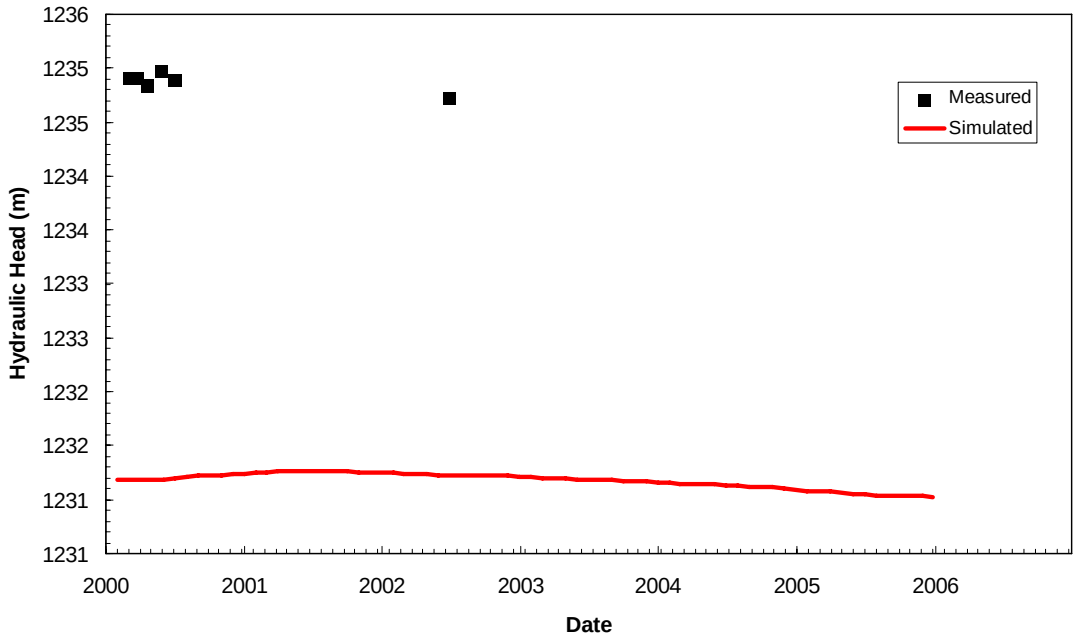




Figure 22. Simulated and measured hydraulic head at well 5 for the transient calibration. The image shows the location of the observation well as a large red circle.



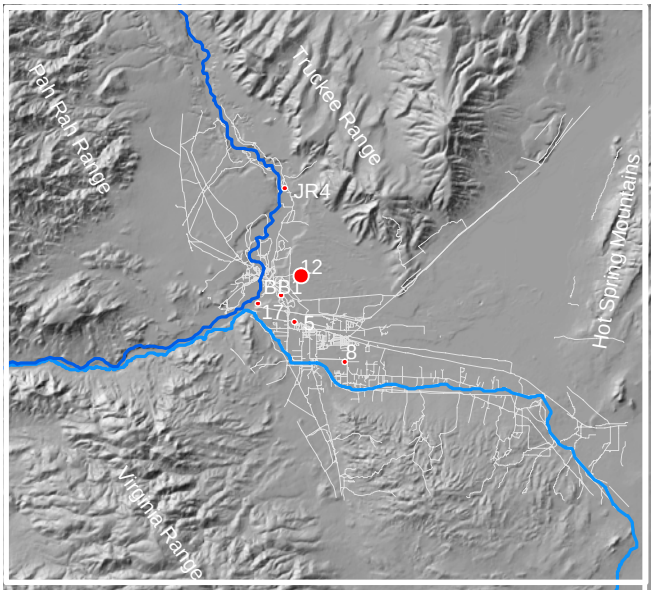




Figure 23. Simulated and measured hydraulic head at well 12 for the transient calibration. The image shows the location of the observation well as a large red circle.

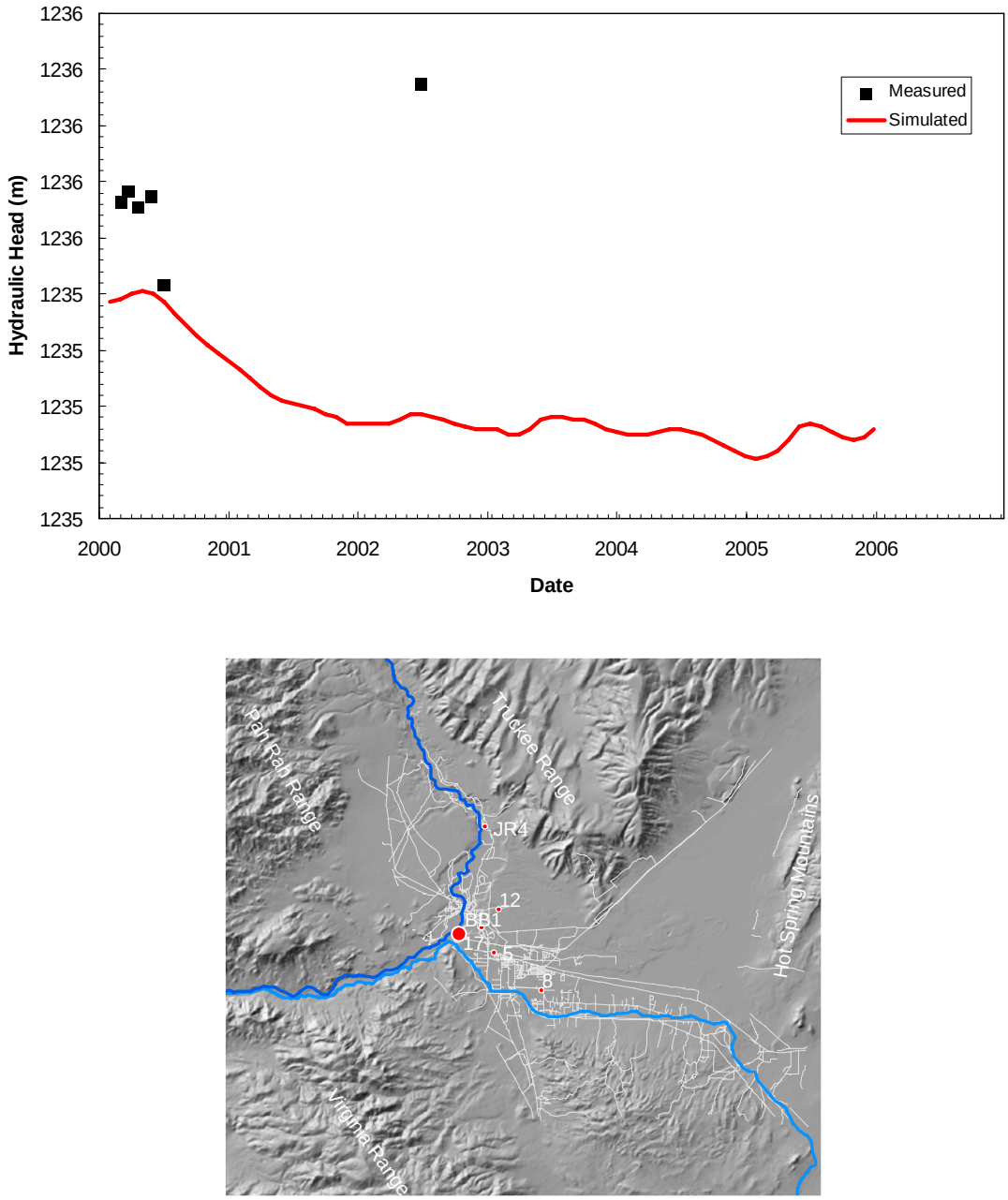
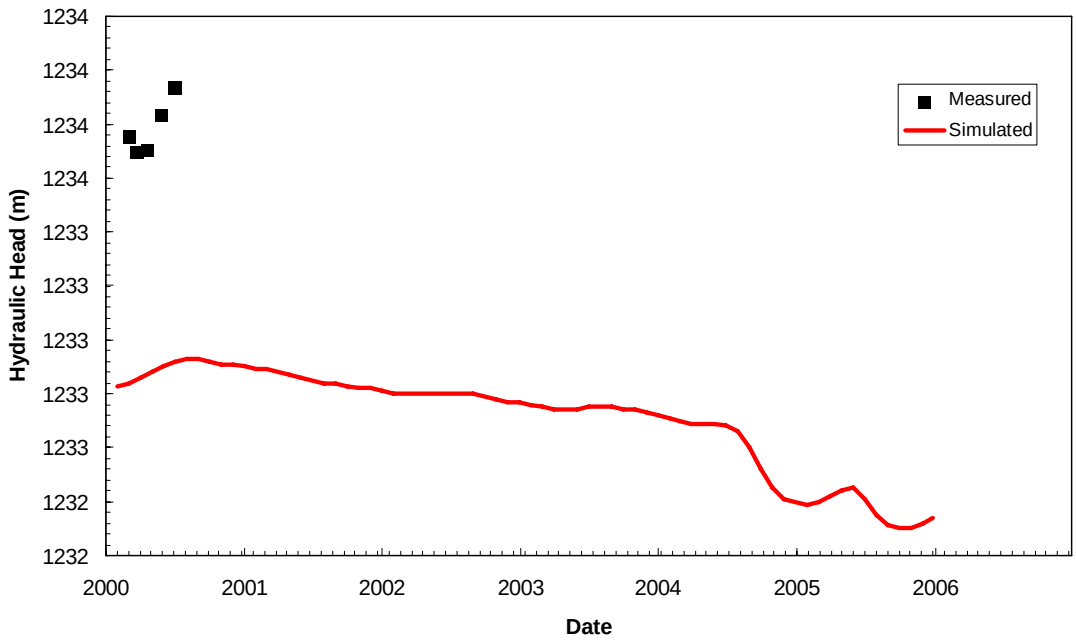



Figure 24. Simulated and measured hydraulic head at well 17 for the transient calibration. The image shows the location of the observation well as a large red circle.



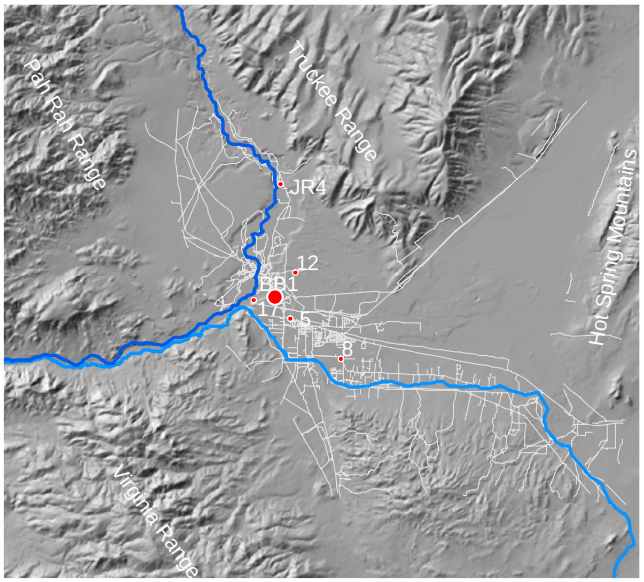




Figure 25. Simulated and measured hydraulic head at well BB1 for the transient calibration. The image shows the location of the observation well as a large red circle.

## RESULTS

### Steady-state Flow Model

The hydraulic heads as simulated from the steady-state model are shown in Figure 26. Generally, higher heads are simulated in the south, with lower heads in the north and eastern portions of the model domain. Lower heads in the eastern part of the valley are associated

with ET, which causes water to stagnate. Some water exits the domain in the southeastern portion of the domain as water flows toward Hazen. In the west, along the Truckee River corridor, higher heads are associated with mountain block recharge in Dodge Flat and groundwater inflow below Derby Dam. A groundwater mound is simulated in the central Fernley area due to higher seepage rates from the Truckee Canal. The simulated groundwater mound is 25 m (80 ft) above the surrounding water levels. The groundwater mound supports a groundwater divide known to exist just north of Fernley. Hydraulic heads along the western edge of Dodge Flat are higher than those near the Truckee River, causing northwesterly flow in this region.

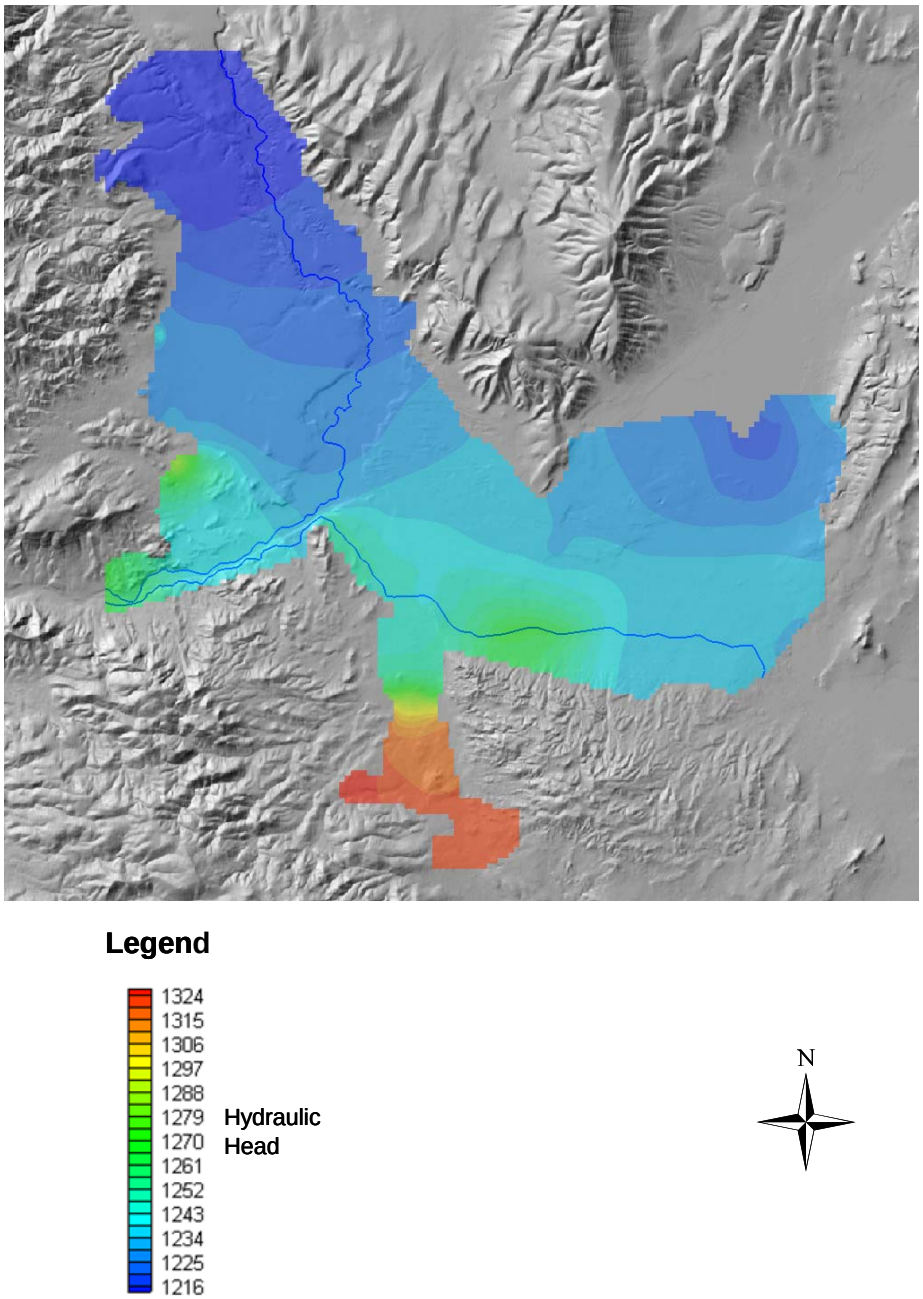



Figure 26. Simulated hydraulic head distribution from the steady-state flow model.

Advective particle paths were calculated to visualize groundwater flow directions using the steady-state model as shown in Figure 27. Groundwater flow from the south moves north, with a majority of the water captured by Fernley production wells 3, 4, and 11. Fernley municipal wells 3, 4, and 11 also capture Truckee Canal seepage that occurs to the east of this well field. Along the eastern portion of the valley, Truckee Canal seepage moves northward to the Fernley Sink, with a smaller component moving southeast toward Hazen. Along the western portion of Fernley, Truckee Canal seepage moves northward toward the Truckee River. Mountain block recharge in Dodge Flat moves northward and exits via the Truckee River or as groundwater flow toward Pyramid Lake.

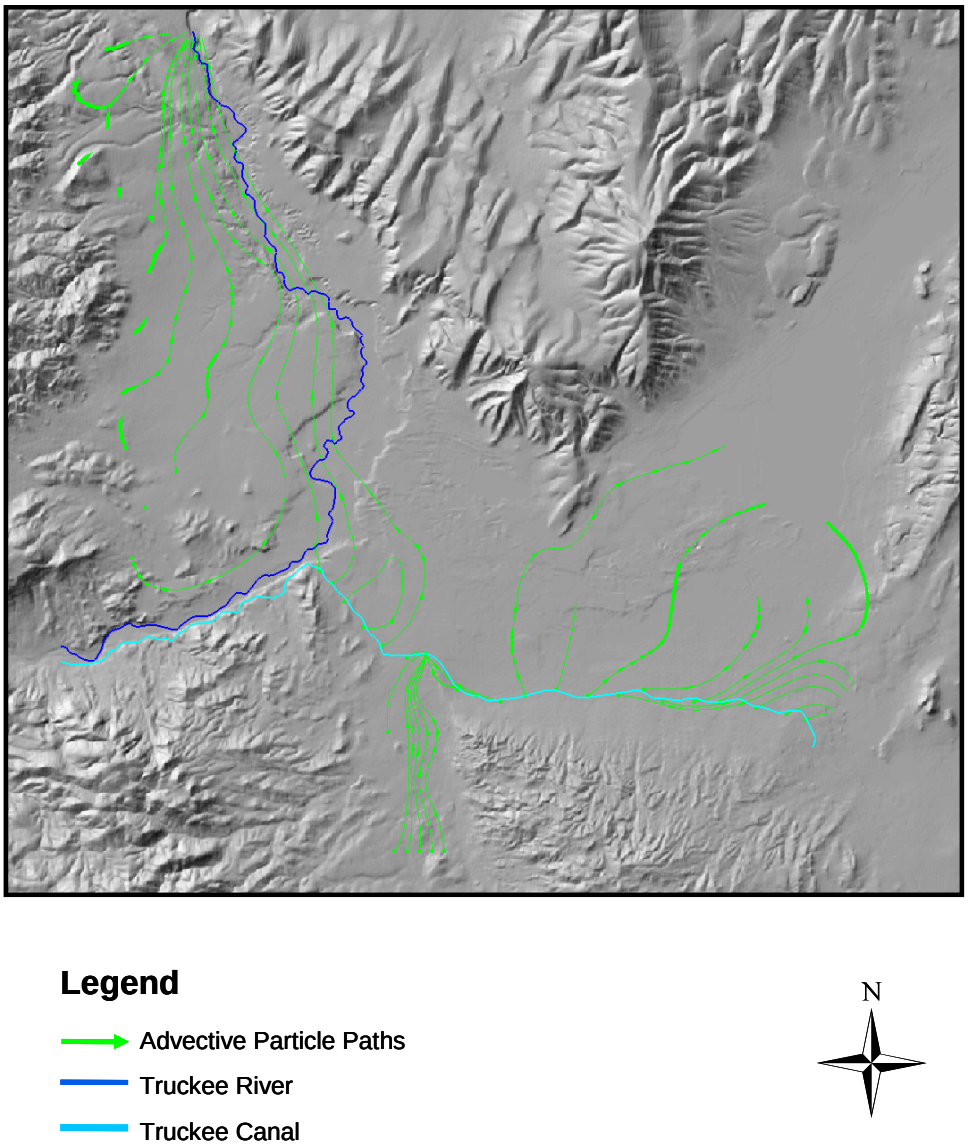



Figure 27. Advective particle paths as simulated from the steady-state model.

Evapotranspiration was primarily a water balance factor in the playa region east of Fernley and in the cottonwood zones adjacent to the Truckee River (Figure 28). The average ET rate in the steady-state model was $3.33 \times 10^{-4}$ m/day ($1.09 \times 10^{-3}$ ft/day) over an area of 41,040,000 $m^2$ (approximately 10,141 acres).

## Transient Calibration Model

The hydraulic heads simulated by the transient calibration model at the final time step (December 2005) are shown in Figure 29. The spatial pattern of hydraulic heads is nearly identical to the steady-state simulation. A closer examination indicates that there are small differences in hydraulic head along the Truckee Canal and Truckee River. These differences can be attributed to the use of the new SFR2 package to simulate the Truckee River and Truckee Canal. The transient simulation includes actual flow rates for these surface water bodies that could cause fluctuations in the hydraulic head levels adjacent to these features.

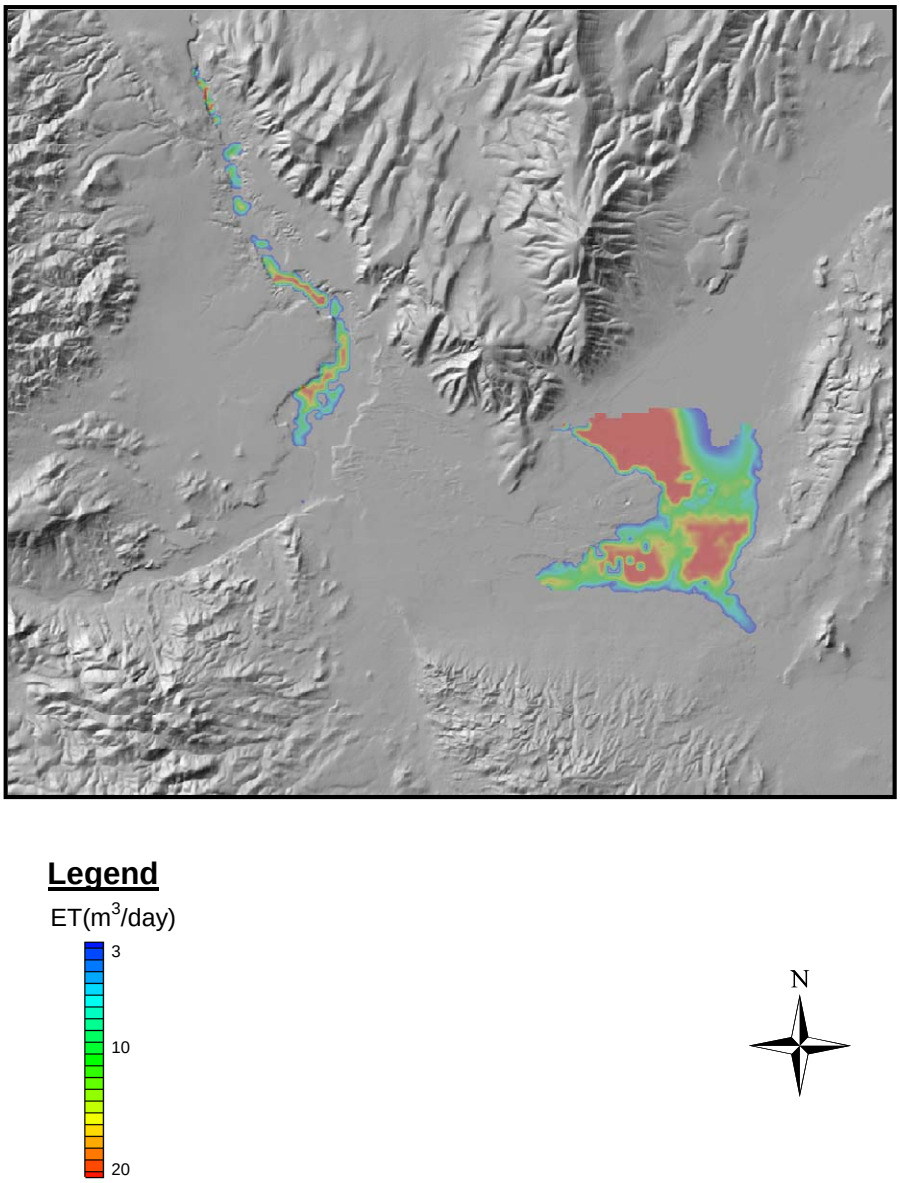



Figure 28. Simulated evapotranspiration for the steady-state flow model.

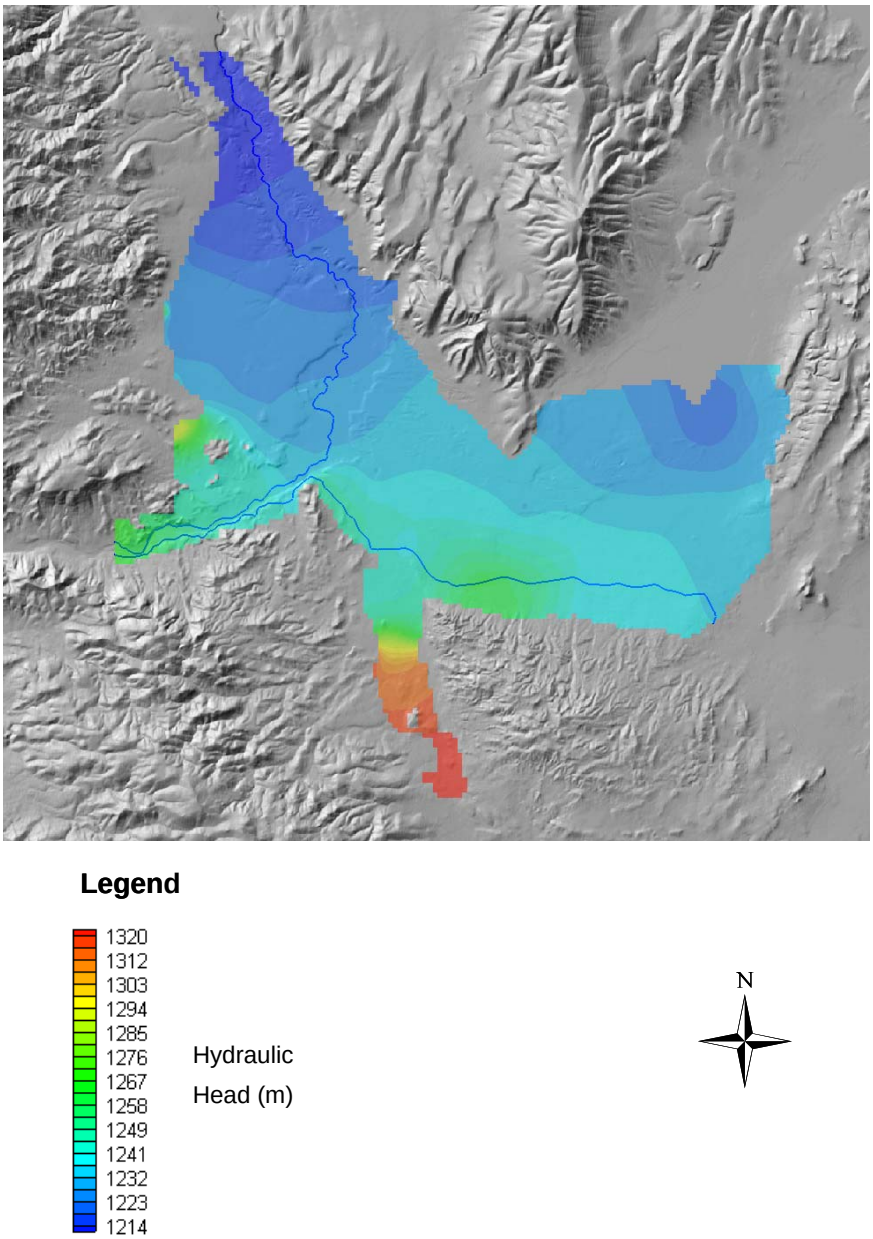



Figure 29. Simulated hydraulic head distribution from layer 3 of the transient calibration flow model at the final time step (December 2005).

Under current pumping conditions and Truckee Canal and Truckee River operations, the water levels do not change significantly over time. The primary reason for this is that flows have been maintained in the Truckee Canal throughout the simulation period. This suggests that if Truckee Canal flows are kept consistently above the estimated seepage rate of approximately 40,000 $m^3$/day (10.6 mgd; 11,800 afy), then the groundwater system will not change dramatically if the pumping rates remain relatively constant.

## Transient Predictive Models

Results from the five simulations were generally similar to one another. Natural recharge, Truckee River flow rates, and Truckee Canal flow rates were the three inputs that varied among simulations. As previously discussed, these input parameters were scaled according to past precipitation at the Reno airport, and simulations developed by Stetson Engineers (2006) using past Truckee River flows and Truckee Canal flows based on OCAP requirements. The variations in these water budget components did not cause a significant difference in the simulation outcomes.

The predictive simulations exhibit behaviors similar to previous models of the Fernley/Wadsworth area, with the exception of more drawdown simulated in the future in this model. In general, the simulations begin with a groundwater divide that separates the Dodge Flat and Fernley area hydrographic basins (Figure 30). The divide is located in the southeast portion of the Fernley irrigation areas and is associated with large seepage rates from the Truckee Canal. Groundwater travels from the Tracey Segment into Dodge Flat and from the southern portion of the model domain north toward Fernley. The water mounded under Fernley, due to Truckee Canal seepage and agricultural recharge, moves either northwest towards Wadsworth or northeast to the playa discharge area. Over the simulated period, as groundwater pumping increases in the Truckee River floodplain aquifer, a cone of depression develops and deepens over the 20-year simulation. This cone of depression is apparent in the area called Big Bend, just east of where the Truckee River turns north, diverging from the Truckee Canal. Drawdown occurs on the order of 20 to 30 m (66 to 98 ft) in this area. The cone of depression shifts the groundwater divide that separates the Dodge Flat and Fernley area hydrographic basins (Figure 31). Figure 32 visually demonstrates the progression of the drawdown cone throughout simulation 1.

The Truckee River gains or loses water in different reaches depending on several factors: the ease of communication with the groundwater system, the bed slope, and the river flow rate. Figure 33 shows the simulated Truckee River gains/losses and associated streamflow for simulation 1 of the transient prediction model. During high-flow periods (e.g., March 2009), the river is predominantly losing, with the exception of a small reach located southwest of Wadsworth and at the northern portion of the model domain. As the flow rates decrease, the river is predominantly gaining, except for a small reach in the Wadsworth area and another reach located in the southwestern portion of the model domain. Figures 34 through 38 show the net gain or loss along the entire section of Truckee River included in the model domain for simulations 1 through 5, respectively. These figures show a similar seasonal pattern in the timing of the gains and losses along the Truckee River.

The simulated Truckee Canal seepage exhibits a seasonal pattern, with most of the seepage occurring during the summer months. Figures 39 through 43 show the pattern of seepage for simulations 1 through 5, respectively. The average seepage rate for all five simulations is 33,400 $m^3$/day (8.82 mgd; 9,890 afy), which is smaller than the value (47,790 $m^3$/day; 12.6 mgd; 14,151 afy) used in the steady-state model. The steady-state model assumes that the Truckee Canal flows are constant, and does not account for cessation periods.

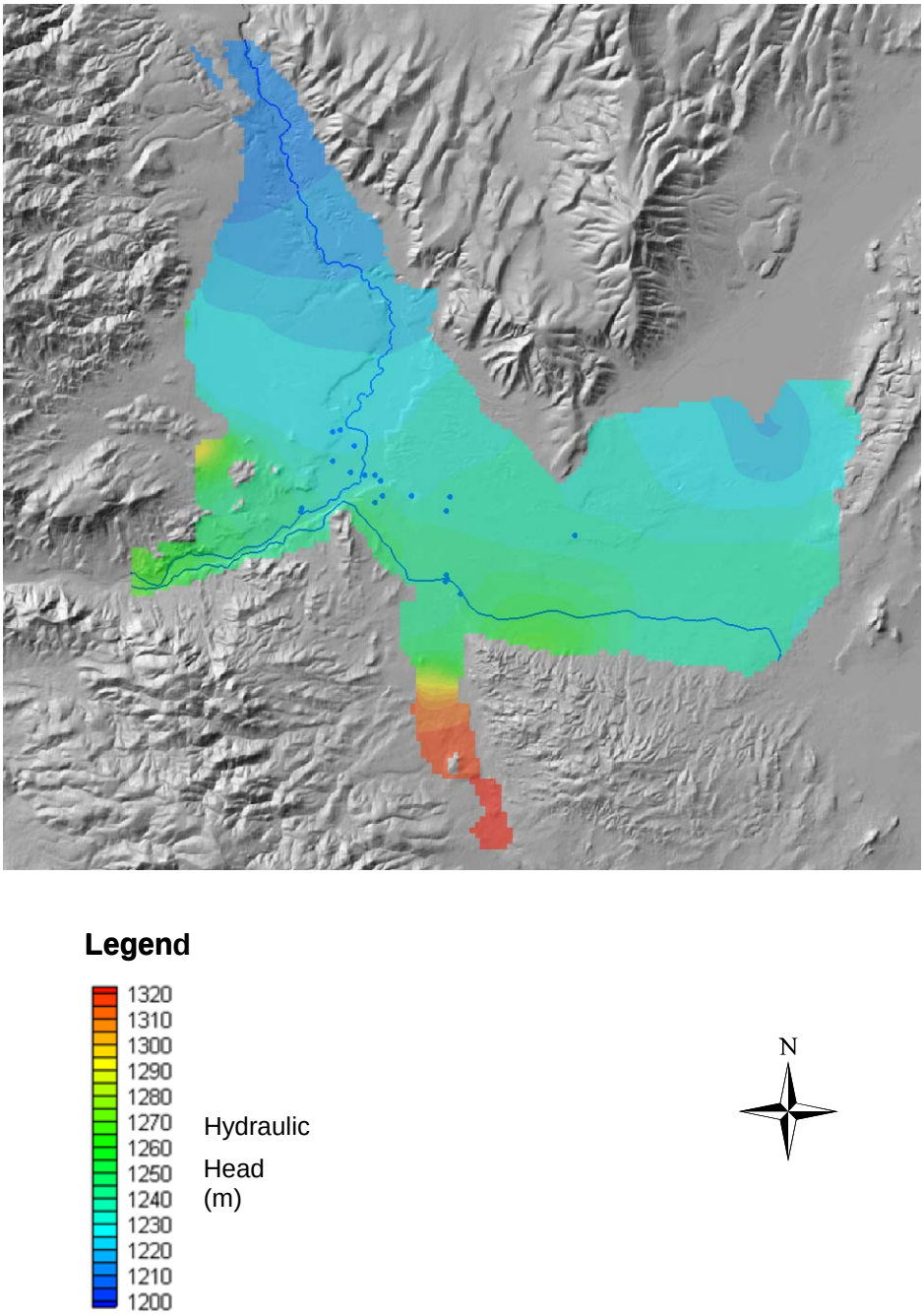



Figure 30. Simulated hydraulic head from layer 3 of the transient simulation 1 at then end of 2006.

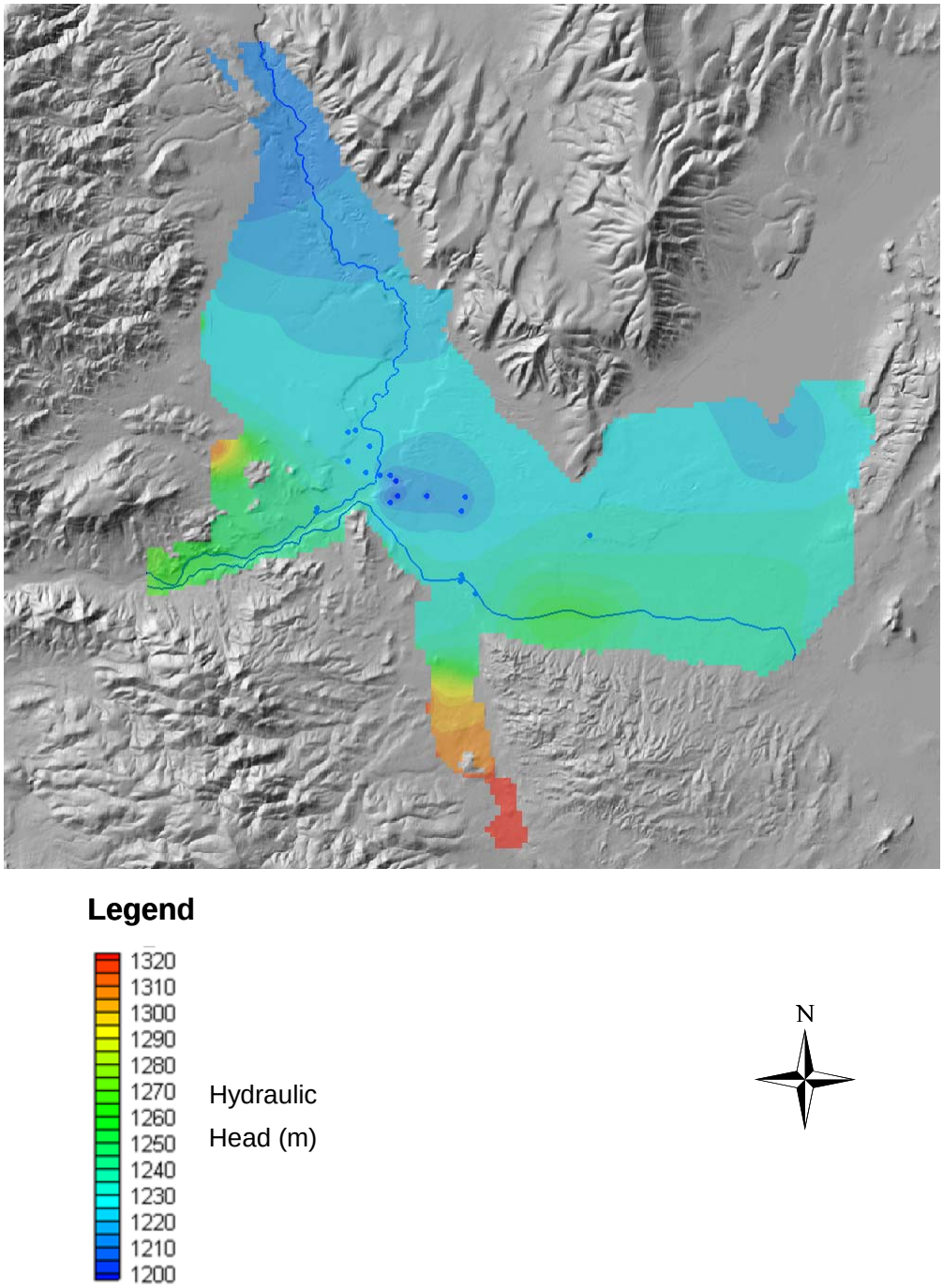



Figure 31. Simulated hydraulic head from layer 3 of the transient simulation 1 at then end of 2026.

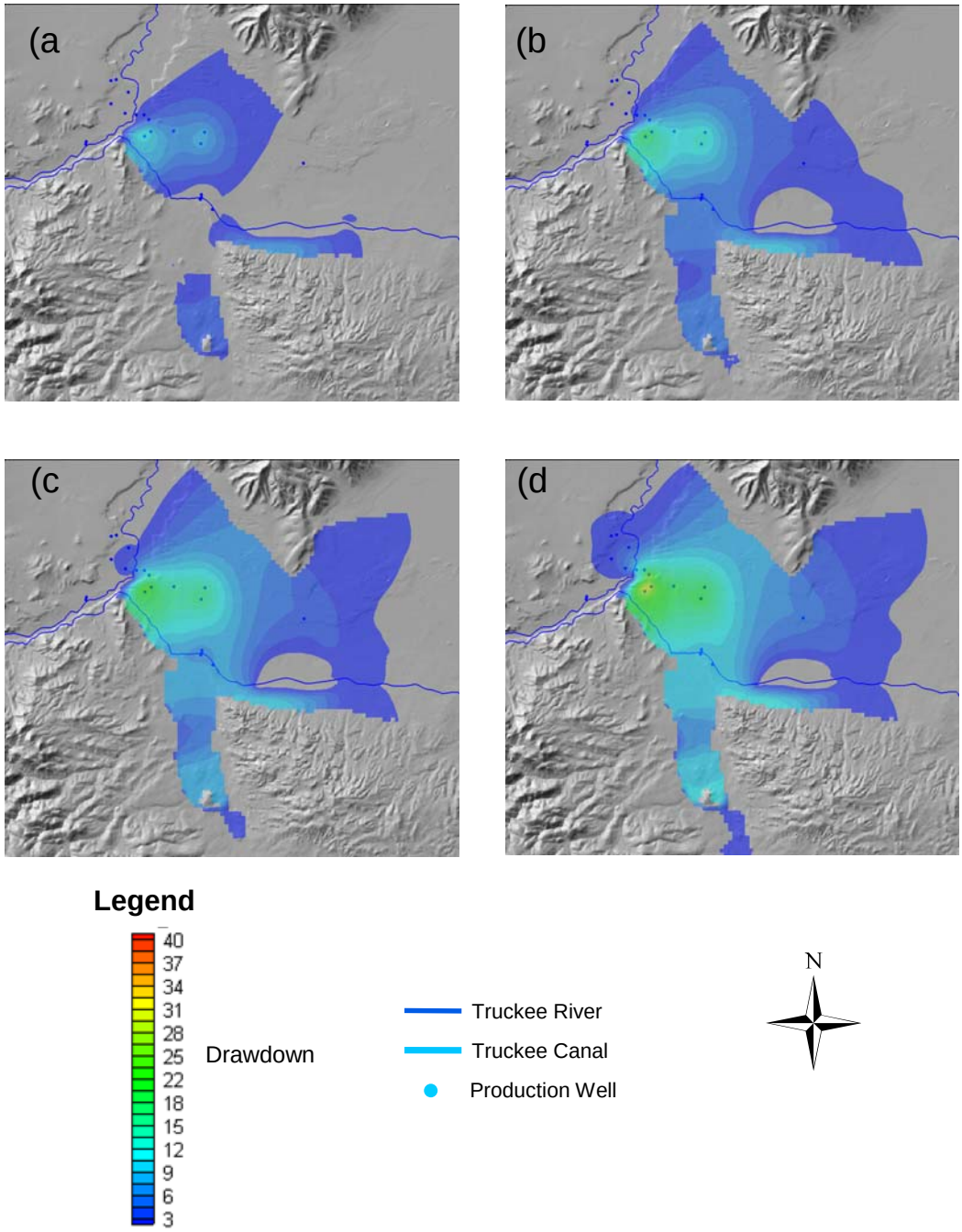



Figure 32. Simulated drawdown for simulation 1 at (a) 2011, (b) 2016, (c) 2021, and (d) 2026.

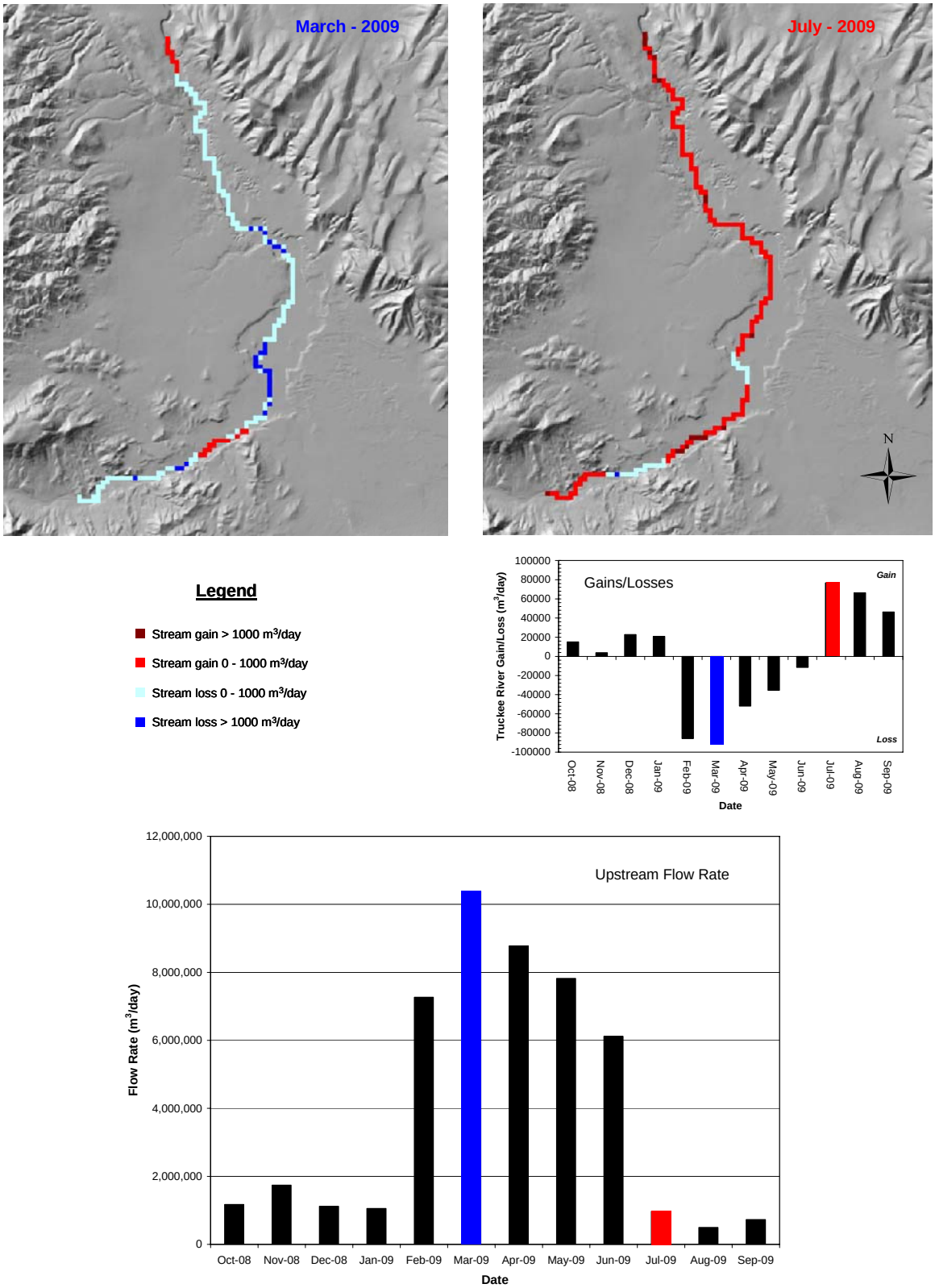



Figure 33. Simulated Truckee River gains/losses and associated streamflow for simulation 1 of the transient prediction model.

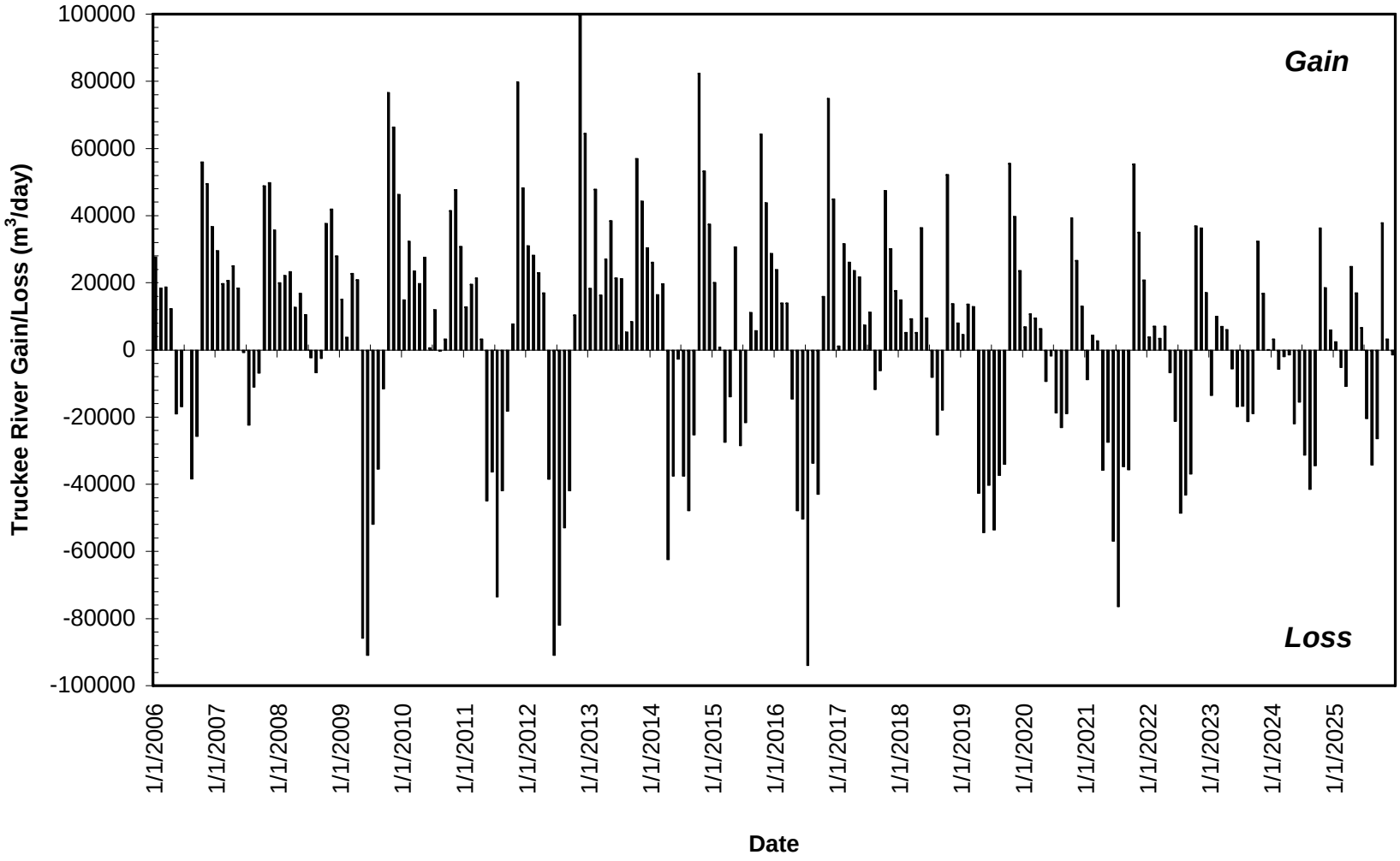



Figure 34. Truckee River gain and loss for simulation 1.

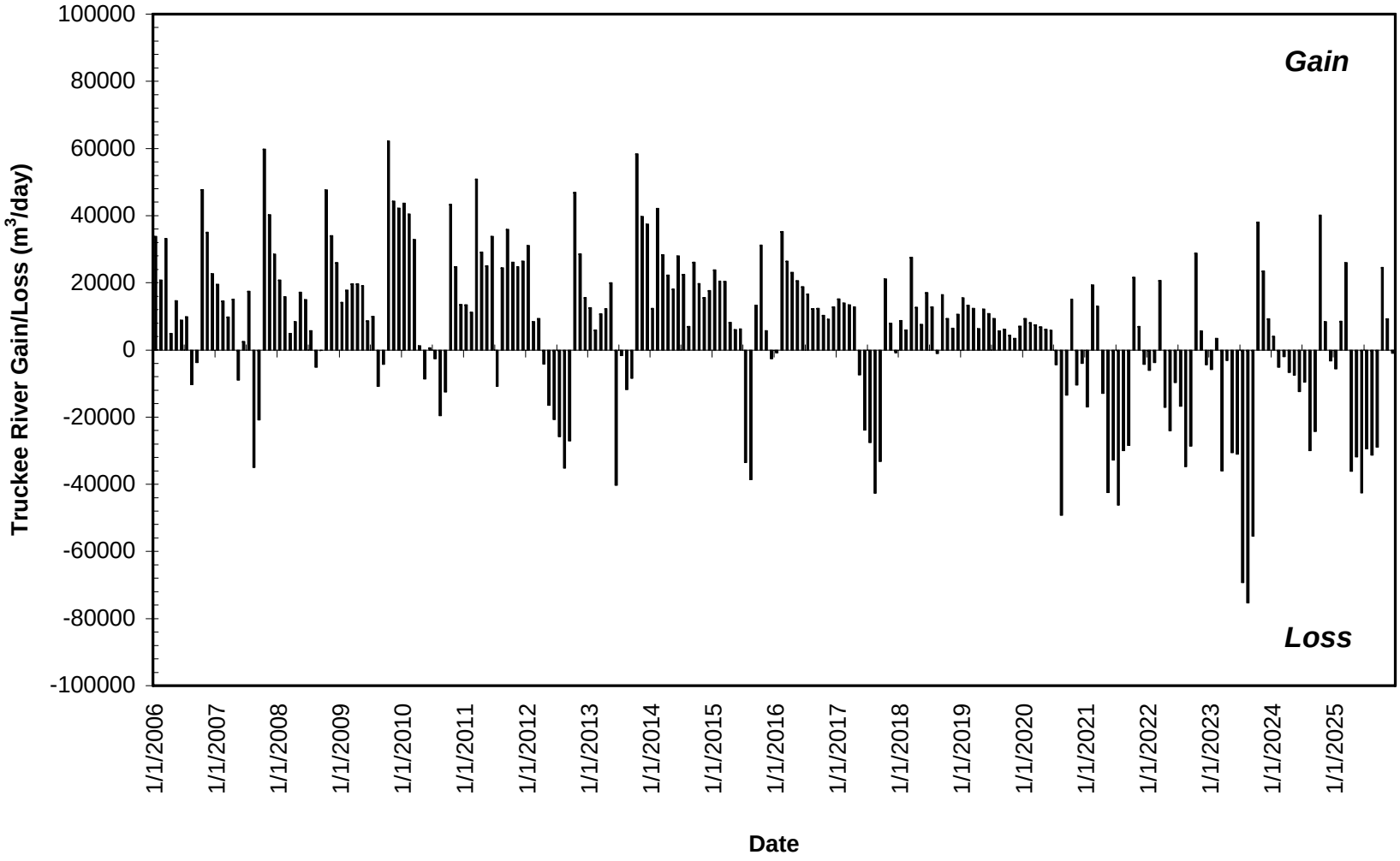



Figure 35. Truckee River gain and loss for simulation 2.

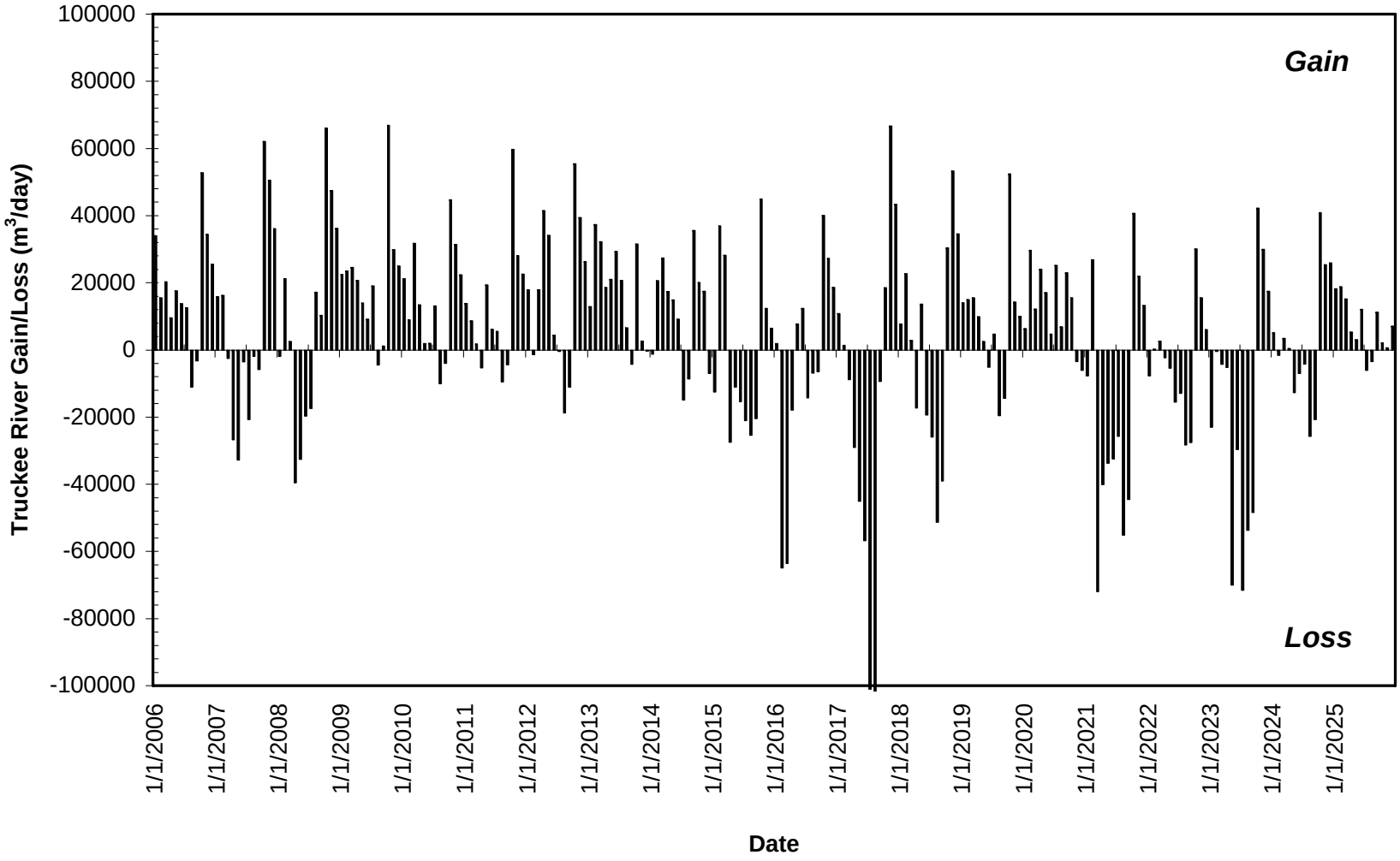



Figure 36. Truckee River gain and loss for simulation 3.

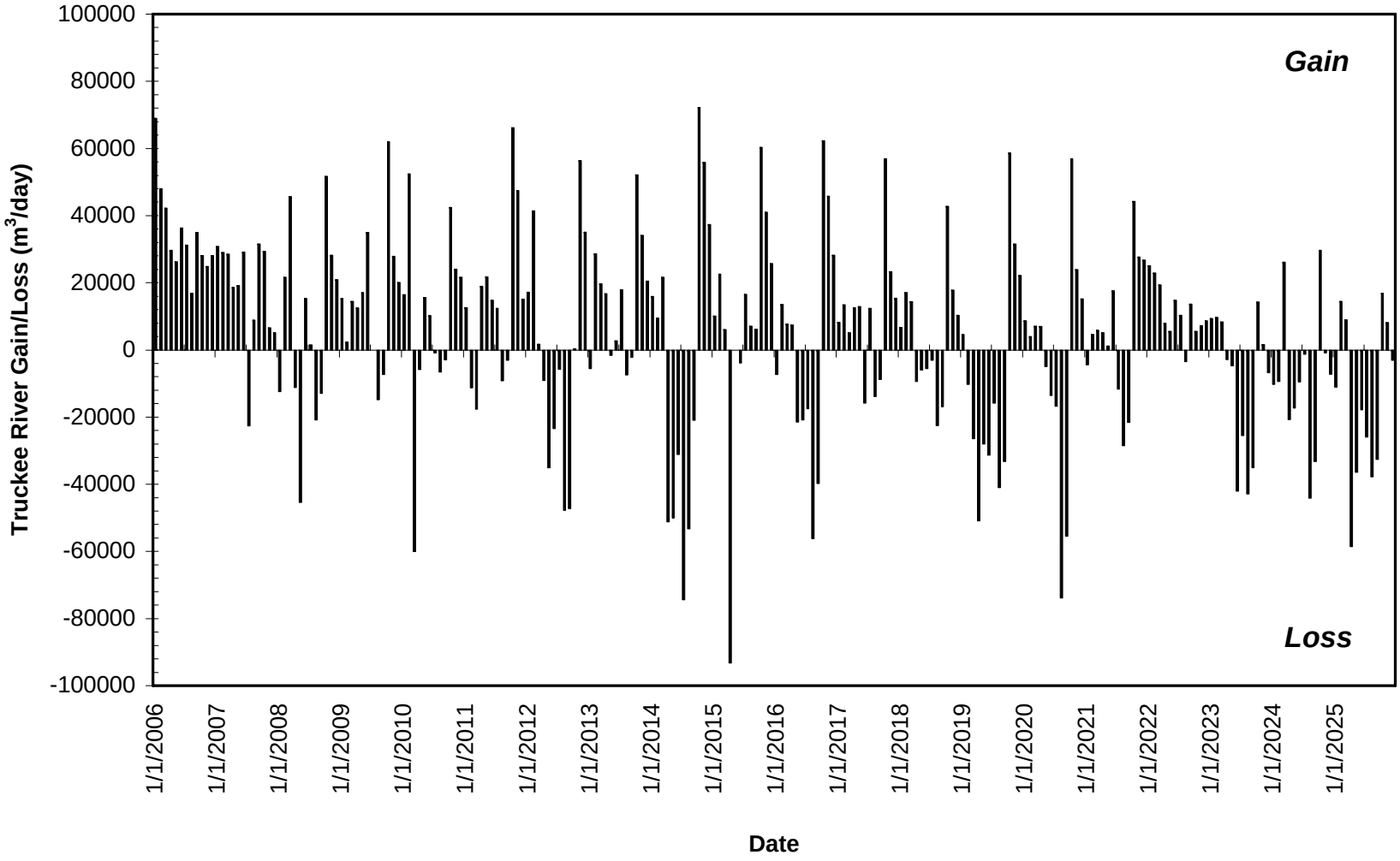



Figure 37. Truckee River gain and loss for simulation 4.

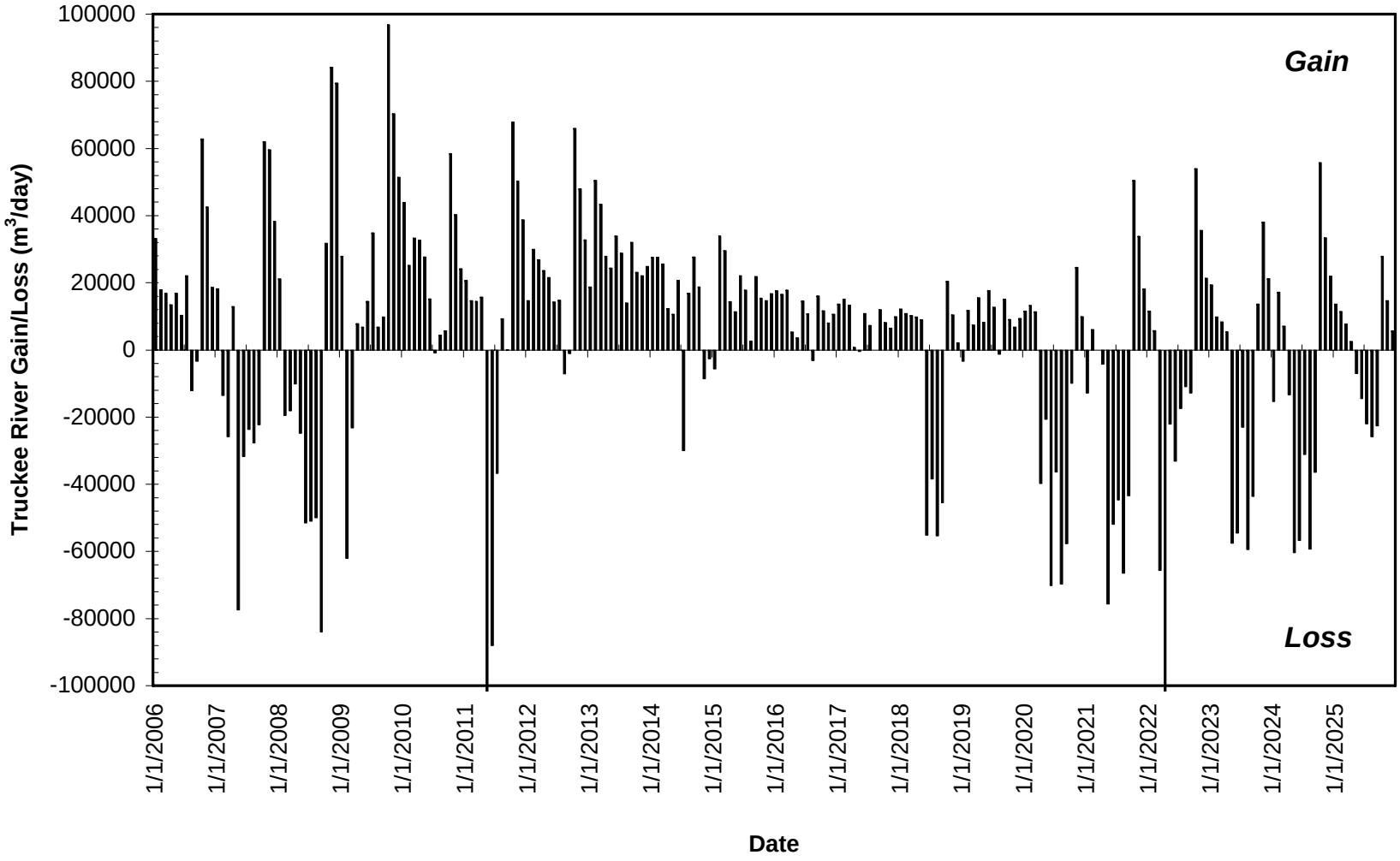



Figure 38. Truckee River gain and loss for simulation 5.

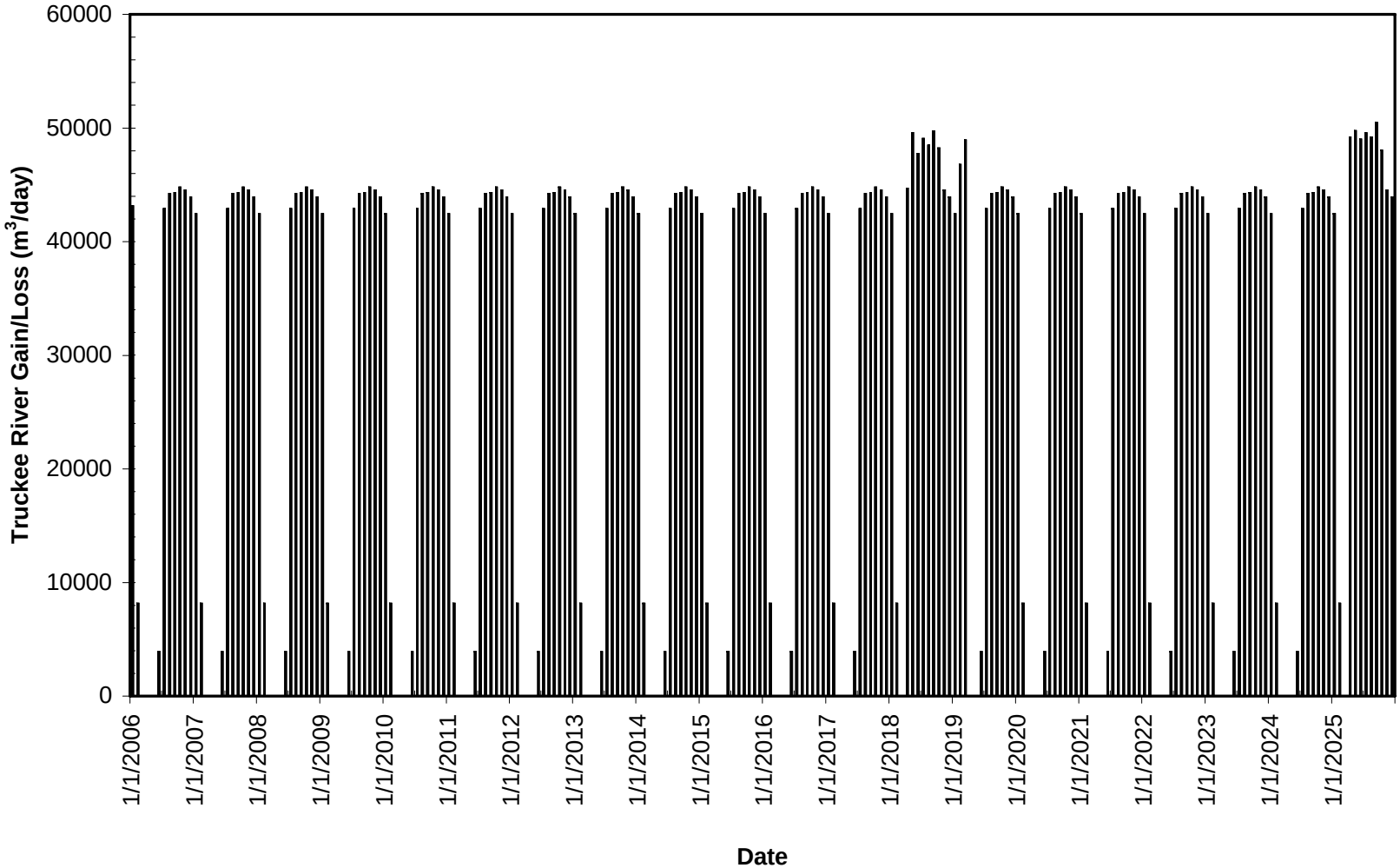



Figure 39. Truckee Canal seepage rates for simulation 1.

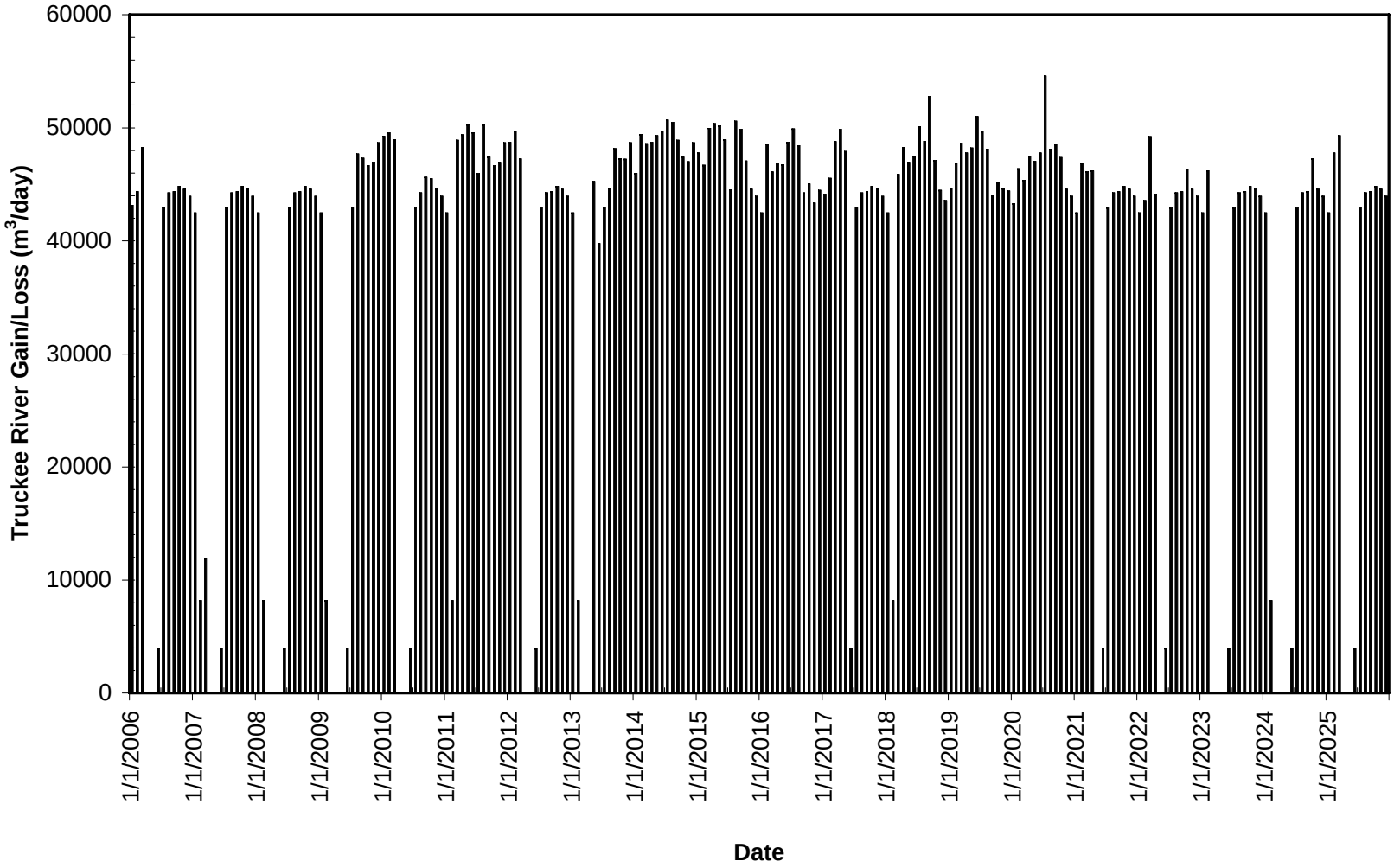



Figure 40. Truckee Canal seepage rates for simulation 2.

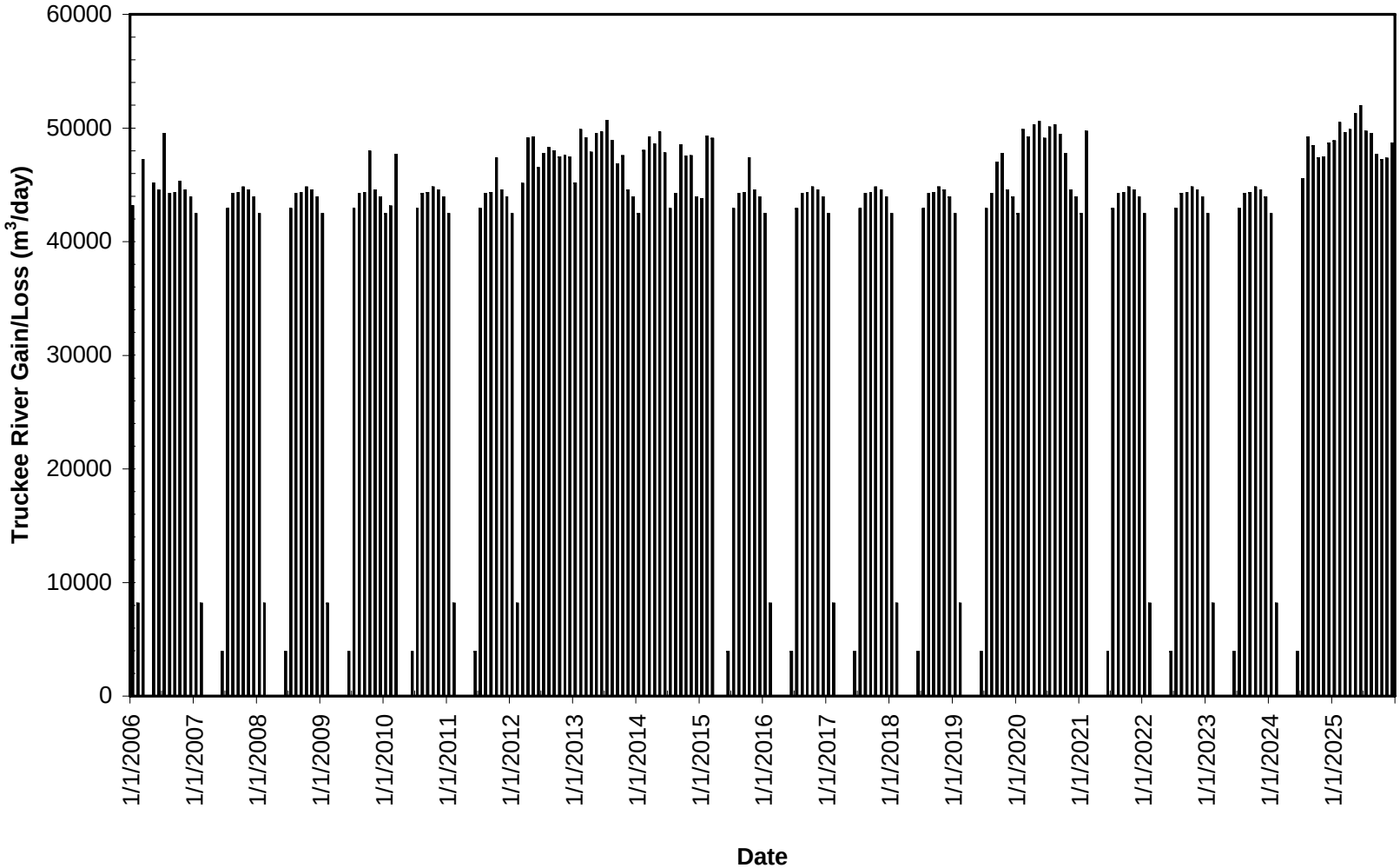



Figure 41. Truckee Canal seepage rates for simulation 3.

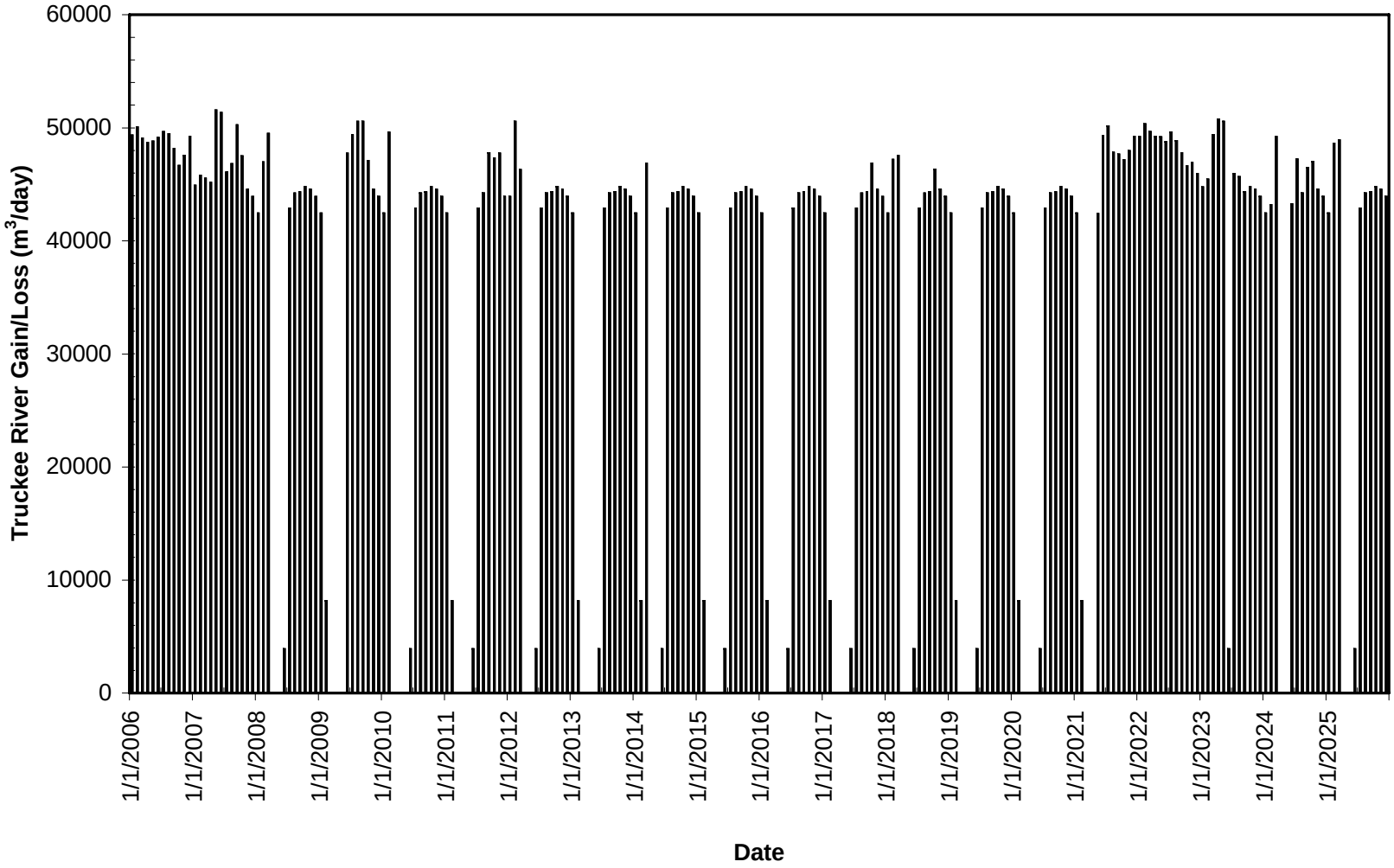



Figure 42. Truckee Canal seepage rates for simulation 4.

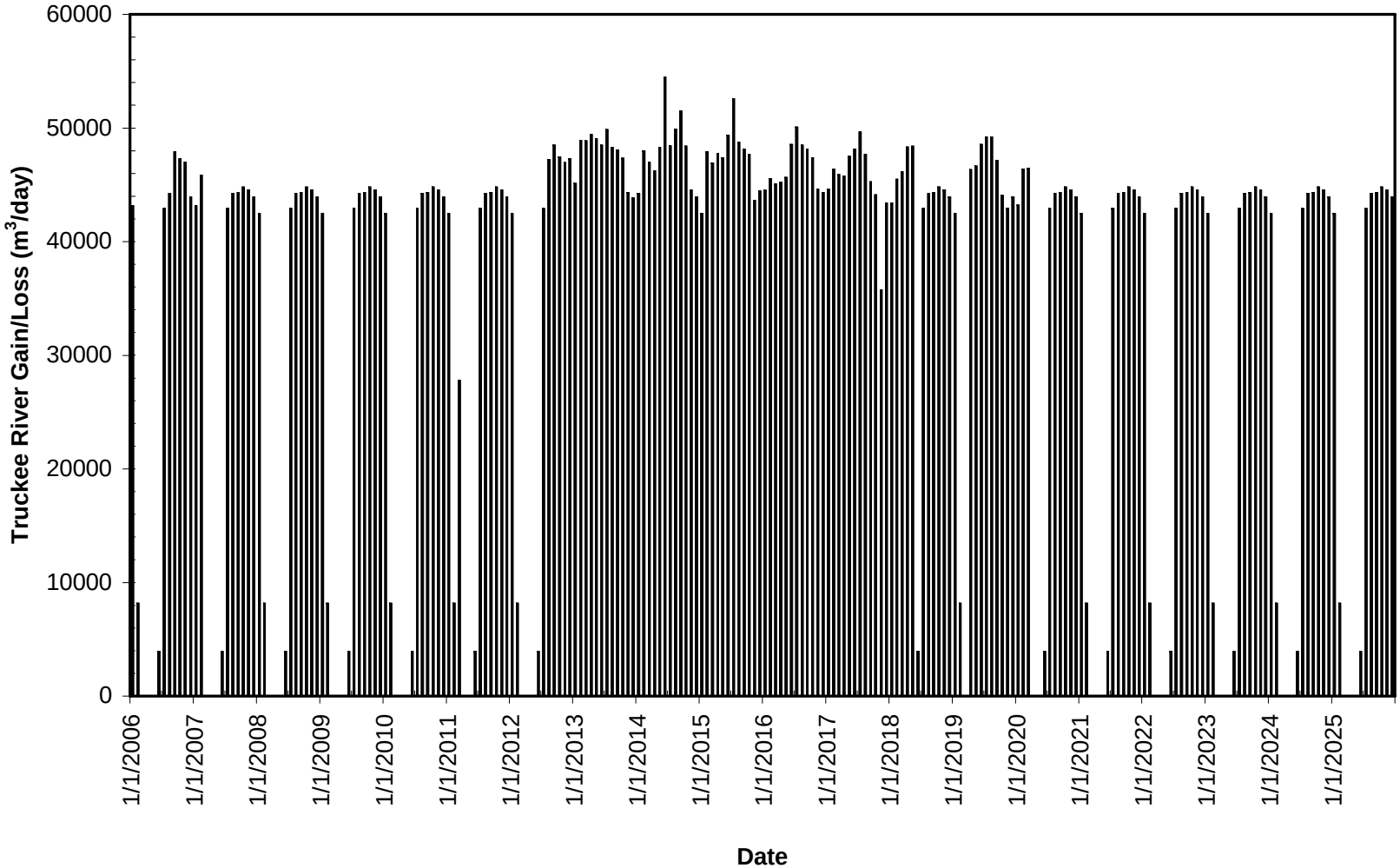



Figure 43. Truckee Canal seepage rates for simulation 5.

## CONCLUSIONS

The purpose of this study was to develop a predictive groundwater model to aid in decision making regarding placement and design of additional groundwater wells. Both a steady-state and a transient groundwater flow model were constructed. The steady-state model provides a relatively accurate tool to quickly assess long-term changes to the groundwater system. The transient model provides a more detailed look at the aquifer system by including the temporal variation in groundwater pumping, recharge, and, more importantly, Truckee River and Truckee Canal flows. These management tools can now be used by the City of Fernley, the Pyramid Lake Paiute Tribe, and USBR to further assess the impacts of present and future water management decisions.

The major conclusions that can be drawn from this study include:

- Under current pumping conditions, and Truckee River and Truckee Canal flows, the groundwater system should not change dramatically.
- Under OCAP Truckee Canal flows would become more seasonal, which will decrease the total amount of seepage from the Truckee Canal.
- Under OCAP and increased pumping, the groundwater levels adjacent to the new pumping centers will decrease on the order of 20 to 30 m (66 to 98 ft).
- Under OCAP and increased pumping, the groundwater levels adjacent to the older pumping centers will decrease on the order of 5 to 10 m (16 to 33 ft).
- Under OCAP and increased pumping, the area of significant water level decline is focused to the east of the Truckee River, which may cause poor quality groundwater to encroach upon the new production wells.

Although the area of significant drawdown is predicted to extend to the Truckee River, it appears the variations in river flow dominate the groundwater-surface water interactions.

## LIMITATIONS

The modeling tools used in this analysis provide the best available estimates of the groundwater system's behavior to further understand the groundwater system. All models are uncertain and should be used with some caution. This model cannot precisely portray the groundwater system, as is the case with any model. The predictions rely on a number of assumptions that may not be met in the actual system. For example, historical flows were used to estimate future conditions, yet future flow in the Truckee River and Truckee Canal may deviate from historical conditions.

The best way to view these results is to consider the information used in constructing the model and then consider the model's prediction of future trends. As is the case with most groundwater modeling predictions, the trends in hydraulic head tend to be more accurate than the simulated head values. In this study, the model predicted water level declines over the next 20 years for all situations. The prediction that water levels will decrease is more reliable than the predicted magnitude of drawdown at specific locations.

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