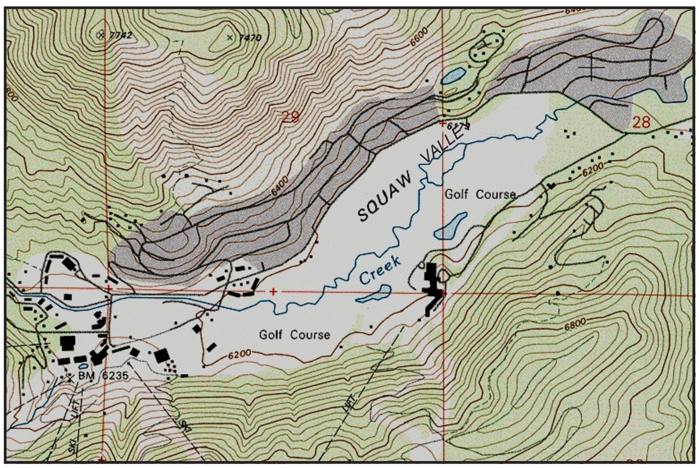
Squaw Valley
PUBLIC SERVICE DISTRICT



WEST YOST & ASSOCIATES
Consulting Engineers

August 14
2003

Squaw Valley Groundwater Development & Utilization Feasibility Study Update

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Prepared for
Squaw Valley Public Service District

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LIST OF ABBREVIATIONS

∑	The sum of
µg/L	Micrograms per Liter
AB	Assembly Bill
af	Acre-Feet
af/yr	Acre Feet per Year
ASCWD	Alpine Springs County Water District
AWWA	American Water Works Association
bgs	Below Ground Surface
BMP	Best Management Practices
CCR	Consumer Confidence Reports
CEQA	California Environmental Quality Act
CT	Disinfection Detention Time
DBP	Disinfection By-Products
District	Squaw Valley Public Service District
DOHS	State Department of Health Services
DU	Dwelling Unit
DWSAP	Drinking Water Source Water Assessment and Protection
EIR	Environmental Impact Report
EIS	Environmental Impact Statement
EPA	Environmental Protection Agency
Fe	Iron
ft	Feet
ft^2	Square Feet
GAC	Granular Activated Carbon
gal/yr	Gallons per year
$gal/yr/ft^2$	Gallons per Year per Square Foot
gpcd	Gallons per Capita per Day
gpd	Gallons per Day
gpd/unit	Gallons per Day per Unit
gpm	Gallons Per Minute
HVAC	Heating, ventilation and air conditioning
kwh	Kilowatt Hours

MAE	Mean Absolute Error
MCL	Maximum Concentration Limit
ME	Mean Error
MG	Million Gallons
mg/L	Milligrams per Liter
mgd	Million Gallons per Day
Mn	Manganese
MPA	Microscopic Particle Analysis
MTBE	Methyl Tertiary – Butyl Ether
Mutual	Squaw Valley Mutual Water Company
MW	Monitoring Well
ND	Non-detection
No.	Number
PAC	Powdered Activated Carbon
PAH	Polycyclic Aromatic Hydrocarbons
PCA	Potential Contaminating Activities
PLC	Programmed Logic Controller
RMSE	Root Mean Squared Error
RWQCB	Regional Water Quality Control Board
SCADA	Supervisory Control and Data Acquisition
SDWA	Safe Drinking Water Act
STD	Standard Deviation of Errors
SVMWC	Squaw Valley Mutual Water Company
SVPSD	Squaw Valley Public Service District
SWRCB	State Water Resources Control Board
TCPUD	Tahoe City Public Utility District
TDS	Total Dissolved Solids
TM	Technical Memorandum
TROA	Truckee River Operating Agreement
T-TSA	Tahoe-Truckee Sanitation Agency
ULFT	Ultra-Low Flow Toilet
UV	Ultraviolet

EXECUTIVE SUMMARY

PROJECT BACKGROUND AND OBJECTIVES

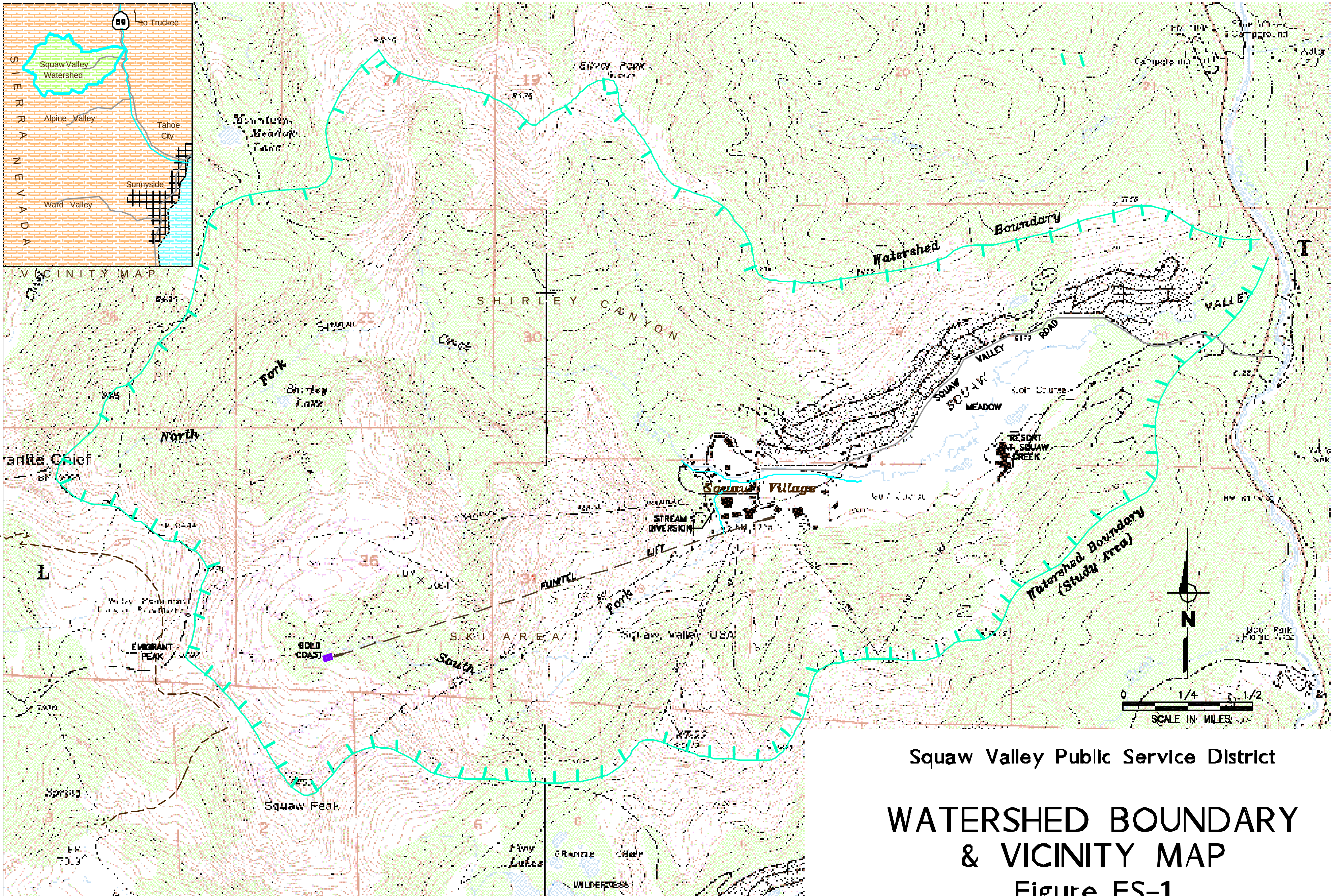
In the past ten years, there has been a significant development within Squaw Valley and as a result, water supply requirements have increased. The Squaw Valley Public Service District's (District) 1993 Water Master Plan, which has been used by the District to guide the orderly expansion of the water system, included recommendations for the collection of additional hydrogeologic data and the development of a groundwater management program for the valley. The scope of the recommended study also included the determination of the sustained yield of the basin to confirm adequate water supply will be available to meet the increased demands associated with the buildout of Squaw Valley's General Plan.

Over the past twenty years several studies have estimated the yield of the groundwater basin. These studies were based on very limited information and many simplifying assumptions to arrive at the estimated recoverable yield of the basin. The experience during the sustained drought period of 1987 through 1992 showed that the groundwater resource is capable of supplying current demands without exhibiting sustained overdrafting. However, there is concern about the capability of the groundwater basin to provide sufficient good quality water to supply increasing future demands. In addition, there is the need to replace the existing wells that are more than 40 years old and require increasing rehabilitation efforts to maintain their water production capabilities.

The groundwater basin west of the golf course provides good quality water that meets all primary and secondary drinking water standards. The watershed boundary and location map of Squaw Valley are shown on Figure ES-1. The groundwater in the rest of the basin is of lower quality due to the geology and highly mineralized geothermal springs. A number of wells have been drilled recently that have produced water with high iron and manganese concentrations that will require treatment prior to being distributed for consumption. The District is also concerned that the current supply meet future drinking water standards for radon and arsenic.

The District is proceeding in a diligent manner to identify the needed water supply and treatment facilities to meet increasing demands. The ability to continue to serve all the District's needs from the western end of the valley without treatment is limited. The District needs a plan to guide the responsible development of additional water supply sources and treatment facilities to meet the demands of their customers. The results of the Groundwater Development and Utilization Feasibility Study were used to identify the sustainable yield of the basin, develop a watershed management plan to protect the resource, site new well locations and prepare a recommended capital improvement program for the District to continue to meet the water supply needs of the valley.

Since the completion of the Squaw Valley Groundwater Development and Utilization Feasibility Study Report, dated October 2001, the District has undertaken an update of the groundwater model and reanalysis of the sustainable yield. New information had become available since the development of the original computer model and the model was updated using newly created mapping of the eastern two-thirds of the valley and the recently collected geologic and hydrologic data.



Squaw Valley Public Service District

WATERSHED BOUNDARY & VICINITY MAP

Figure ES-1

The updated model was then recalibrated to the original calibration period and data. The updated model was then used to develop new estimates of sustainable yield using just existing wells and then adding new wells to meet future water requirements. This updated report incorporates descriptions of the work to update the model and the results of the new sustainable yield analyses.

SCOPE OF STUDY

The scope of work for this feasibility study to develop supplemental water supplies included the thorough evaluation of the surface and groundwater resources in Squaw Valley including siting and drilling of new test holes, development of a water resources protection and management plan, development of a basin-wide groundwater model, evaluation of alternatives to meet the District's future water demands, estimation of the basin's sustainable yield, identification of new well sites that can supply water either without and with treatment, and development of capital improvement program recommendations including wells, piping and treatment facilities.

The following sections summarize the work accomplished as part of the feasibility study and present recommendations for the protection, development and use of the water resources in Squaw Valley.

REVIEW AND ANALYSIS OF EXISTING DATA

The initial task in evaluating the available water resources in Squaw Valley was to collect and review available data on the occurrence and use of surface and groundwater in the valley, the physical features and characteristics of the groundwater basin and the numerous petroleum hydrocarbon spills and naturally occurring elements in the valley. The data were used in subsequent tasks including the development of a source water protection plan and watershed sanitary survey, a groundwater hydrology model of the valley, an assessment of the available water resources to meet anticipated demands, and ultimately a recommended capital improvement program to develop additional water supply to meet District's future needs.

Hydrogeologists with Kleinfelder prepared three technical memoranda (TMs) associated with the collection, review, and compilation of background data on the hydrogeology, aquifer characteristics, well construction, and water quality of the groundwater basin in Squaw Valley. The information summarized in these TMs was the primary source of data used in the subsequent tasks performed as a part of this feasibility study. There have been numerous studies, reports, investigations, explorations, and wells constructed in the valley that provide a significant amount of data useful in the development of a computer model of the basin and in analyzing the valley's water resources. During the past forty years numerous wells and borings have been installed and countless water and soil samples collected and analyzed along with pumping tests. These sources provide a large body of data to define the characteristics and occurrence of water bearing strata in the groundwater basin. Review of the published and file material was completed and a synopsis of the pertinent information was summarized by Kleinfelder in a TM entitled "Squaw Valley Groundwater Background Data" dated February 15, 2000. A separate TM, entitled "Report on Field Activities" dated February 15, 2000 was prepared by Kleinfelder on the drilling and sampling of two test holes. This TM described the work performed and summarized the information obtained on the lithology, aquifer characteristics, and water quality. In March 2000, three additional test holes were drilled and sampled by Kleinfelder. Technical data and associated documentation relating to test holes 3, 4 and 5 installed at the Resort at Squaw Creek from March 7 to 11, 2000 were presented in a third technical memorandum dated June 6, 2000.

A separate investigation of known, man-made chemical and natural occurrences of specific elements in groundwater of Squaw Valley was undertaken by Kleinfelder. This investigation identified and evaluated the available information on known chemical contamination sites, primarily releases of petroleum hydrocarbons. The contamination sites were identified from searches and reviewing files from county and state regulatory agencies. Historical photos were also reviewed to assist in identifying the locations of past structures and activities that may have been associated with possible releases of contaminants to the groundwater. A total of 13 petroleum hydrocarbon release sites were identified through this research. The review of the record information on each of these sites was summarized by Kleinfelder in a fourth TM entitled "Known Man-Made Chemicals and Natural Occurrences in Groundwater" dated December 17, 1999.

WATERSHED INVESTIGATION, SOURCE WATER ASSESSMENTS AND GROUNDWATER PROTECTION PLAN

The watershed sanitary survey was conducted to obtain initial information on existing contaminant sources and to identify development and activities in the watershed that may contribute contaminants to surface water and subsequently to the groundwater resources of the valley. Identification of these sources is an important step toward the subsequent development of a watershed management plan to protect the water resources of the valley so that they may continue to serve the vital community needs. The Watershed Sanitary Survey report was submitted to the District for their use in June 2001.

The District and the Mutual Water Company completed Source Water Assessment that included the following elements:

- Delineation of the boundaries of the protection areas for wells providing source water for District customers
- Inventory of the sources of regulated and certain unregulated, contaminants of concern in the delineated areas/capture zones (to the extent practical)
- Determination of the vulnerability of the wells to contamination
- Public education and outreach

The Source Water Assessments Report, finalized in June 2001, also contained the delineation of capture zones for each of the production wells for the 1-, 2- and 5-year periods. The delineated areas or capture zones were determined using the groundwater model developed for the valley as part of this study. The delineated protection areas allow the District to focus protection, management strategies, and resources on areas providing the most benefit to the water resource.

The District invited stakeholders to form the Squaw Valley Groundwater Protection Advisory Group to help identify, develop and implement local measures that will advance the protection of the District's groundwater supply. A series of meetings were then held and a proposed Groundwater Protection Plan prepared.

A groundwater protection plan was developed through the stakeholder process. The plan provides direction and focus for groundwater protection efforts undertaken by the District and the community. The plan outlines management strategies that together will provide the key to a successful prevention program.

GROUNDWATER MODEL DEVELOPMENT

A groundwater flow model of the Squaw Valley Basin was developed as part of this study. Development of the model was discussed in the report titled, "Groundwater Model Report" prepared by Derrik Williams, Registered Geologist. The primary modeling objective was to develop a tool for the District's future water planning efforts that could evaluate future groundwater management alternatives. The model incorporates all known groundwater recharge and discharge mechanisms, as well as all available hydrogeologic data from the basin. The model successfully simulates water level fluctuations in both production wells and monitoring wells throughout the basin, and reasonably simulates flows in Squaw Creek. The combination of a solid technical model base and successful calibration has resulted in a valuable tool for future groundwater management studies.

The groundwater model is the best tool available for estimating effects of various pumping and recharge scenarios, and should be used for planning future groundwater management. Pumping rates from existing wells, placement of future wells, and effects of pumping on stream flows can all be studied with the existing model. The model will improve any future planning decisions, and can identify optimal groundwater management strategies.

As with all groundwater models, additional data will help validate the model, and direct modifications to uncertain model parameters. Data that may be particularly helpful includes measured stream flows entering and leaving Squaw Valley, and additional water level data from the western portion of the basin. Squaw Creek flow data will corroborate estimates of the amount of groundwater lost or gained by stream interaction. Additional stream data will furthermore allow accurate calibration of the impact on streamflow from groundwater pumping.

Water level and hydrologic parameter data from the western end of the Squaw Valley Basin will assist in future water management planning. The western portion of the basin has generally better producing wells, and the groundwater in the western basin generally does not require treatment before it is served. Additional data on the production capability of the western basin, along with information about the impact of Squaw Creek on water levels in the western basin, is crucial to future water planning efforts.

As with all groundwater models, the results are only as accurate as the data on which the model is based. Assumptions about the basin dynamics are based on the best available data at the time of model development. As new data becomes available, new interpretations of the basin hydrogeology may require re-structuring of parts of the model.

Since completion of the Feasibility Study Report in 2001, new geologic and hydrologic information became available from a number of sources. This new information included a historical map and aerial photograph showing the location of Squaw Valley Creek in the western portion of the valley prior to construction of the parking lot and channelization of the creek. Also additional information was obtained on the groundwater level response to pumping during the construction dewatering for the Intrawest Squaw Valley Village project in 2002. Also, the District undertook the detailed aerial photogrametric mapping of the valley producing topographic maps of the eastern two-thirds of the valley at a scale of 1 inch = 100 feet with a contour interval of 1 foot. This information and mapping along with several other pieces of data were used to update the groundwater model. This update was not a complete reevaluation of the model, but rather an attempt to incorporate the new data and produce a more accurate representation of the groundwater basin. The new information and mapping and the work completed to update the model are described in a technical memorandum that is included in the Appendix of this report.

ESTIMATE OF ULTIMATE WATER PRODUCTION REQUIREMENT

A projection of the ultimate buildout water demands in Squaw Valley was prepared. Projections are included for the demands served by the District and Mutual, and the Resort at Squaw Creek for golf course irrigation and snow making. The buildout water demand is based on recent estimates by the District of future development that is limited to 80 percent of the development allowed by the 1983 Squaw Valley General Plan and Land Use Ordinance and current water use habits. Estimates of potential water savings from several water conservation measures were also provided. The projection of ultimate water production requirements and a discussion of the need for additional water supply facilities were included.

The buildout water production requirements in the valley with full implementation by the District and the Mutual of the recommended conservation program described above and pumping by the Resort at Squaw Creek for golf course irrigation and snow making is summarized in Table ES-1. The total annual production estimated at full build-out is 2,091 af per year, or 681 million gallons.

Table ES-1. Required Annual Water Production with Conservation in Squaw Valley at Buildout (af)

Supplier/Use	Required Production
Squaw Valley Public Service District	1,628
Squaw Valley Mutual Water Company	202
Resort at Squaw Creek	
Golf Course Irrigation	138
Snowmaking	123
Total	2,091

Also the average day and maximum day production requirements for the District and the Mutual have been estimated and are shown in Table ES-2. The recommended minimum water supply facilities production capability for municipal water purveyors is to be able to meet the maximum day production requirements with the largest supply source out of service. The maximum day production requirements will be used in future work to identify the needed number and size of wells to be in service at buildout.

Table ES-2. Average Day and Maximum Day Production Requirements for District and Mutual at Buildout

Purveyor	Average Day Production Requirement		Maximum Day Production Requirement[a]	
	gpm	mgd	gpm	mgd
Squaw Valley Public Service District	1,010	1.45	2,525	3.64
Squaw Valley Mutual Water Company	125	0.18	315	0.45
Total Municipal Production Requirement	1,135	1.63	2,840	4.09

(a) Maximum Day Production Requirement = 2.5 times the Average Day Production Requirement

WATER PRODUCTION REQUIREMENTS

The water supply and production for the District and the Mutual are identified in Tables ES-1 and ES-2. The annual supply should be available in all years, except in drought emergency years when demand management should be implemented to reduce demands to equal the supply available. The District has recently enacted a water conservation ordinance to assist in managing the demands and groundwater resource. Section 3.33 "Critical Water Supply Shortage, Emergency Water Conservation Restrictions" of the District's Water Code, sets forth requirements for all District customers to implement mandatory reduction in average base water consumption by 20 percent or more during a critical water supply shortage. A 20 percent reduction in the District's buildout demand shown on Table ES-1 would reduce the annual water supply requirement from 1,628 af to about 1,300 af.

The District's water supply production facilities should be capable of supplying the maximum day demand with the largest well out of service. The pumping capacities of the existing wells are shown in Table ES-3. The pumping capacities range from 120 gallons per minute (gpm) for Well 3 to 400 gpm for Well 5. The District's total pumping capacity is 1,250 gpm. In addition, the District has two horizontal wells that are capable of producing up to 40 gpm. Therefore, the total water supply capacity of all sources is 1,290 gpm. With the largest well out of service, the total capacity is 890 gpm.

Table ES-3. Existing District Water Supply Capacity, (gpm)

Existing Supply Facility	Pumping Capacity
Well 1	390
Well 2	340
Well 3	120
Well 5	400
Horizontal Wells	40
Total Supply Capacity	1,290

In average or wet years, the District's buildout demands were estimated to increase to the values shown in Table ES-2. The water production facilities must produce the maximum day demand with the largest production well out of service. The estimated maximum day demand is 2,525 gpm. Assuming the largest well is out of service, the water supply capacity should be increased by 1,635 gpm. To meet this requirement, the District will need to construct 4 to 6 new wells that produce in the range of 250 to 400 gpm each.

GROUNDWATER MODEL SIMULATIONS TO ESTIMATE SUSTAINABLE YIELD

To develop a reasonable estimate of the dependable water supply that can be developed for use within Squaw Valley, a series of groundwater model runs were completed. The updated groundwater model has been used to develop revised estimates of the basin's sustainable yield during drought years. The definition of sustainable yield was first developed and then a series of iterative model runs were undertaken to develop estimates of the maximum pumping that can be sustained during a critically dry year.

Definition Of Sustainable Yield

For this study, sustainable yield has been defined as the maximum amount of reasonable quality water that can be pumped from the groundwater basin during a critically dry year without significantly impacting the pumping water levels of existing wells. Additional water could be pumped from the valley east of the parking lot area; however, studies show that the water will probably be of marginal quality for drinking purposes and require substantially more treatment than would be economically justified. The sustainable yield analyses of the basin assumed the recharge in a critically dry year is represented by that experienced in 1994. Pumping of existing wells was increased and proposed new wells added in the basin to identify the maximum annual pumping amount that can be sustained without lowering the pumping water levels below the top of the existing wells' perforations. For the estimate using the updated model, the allowable level in the District's Well No. 2 was lowered by 15 feet to reflect a more reasonable pumping level in the well if it were reconstructed or sleeved in the future. This criterion is a conservative approach for defining sustainable yield. The District could and has operated its wells at lower water levels for short periods of time. Lowering water levels below the perforations can lead to operational problems including cascading water, entrained air and increased biofouling. A series of analyses were performed using the updated model to first identify maximum pumping using only the existing District and Mutual wells, and then using the existing wells and proposed new wells, to estimate the sustainable yield of the basin. The results of the sustainable yield analysis using the updated model are described in the technical memorandum included in the Appendix and are summarized below.

Sustainable Yield Analysis

Monthly pumping rates were determined for each simulation and tables created as input data. The model was then run. The resulting water levels in each of the municipal wells were compared to the minimum acceptable water levels. If the levels were found to be above the minimum acceptable elevations for all wells, the pumping rates were increased. Conversely, if the water levels were found to be below the acceptable elevations, the pumping rates were decreased. The sustainable yield was then determined as the maximum annual quantity of pumping that can occur under the critical hydrologic conditions.

Maximum Pumping of Existing Wells

A series of groundwater model simulations were performed to estimate the maximum amount of pumping that could be extracted from existing production wells during a drought year. The drought conditions chosen for the simulations duplicated the dry year conditions of 1994. Each model run simulated two consecutive years of drought.

The total pumping of existing wells of 706 af/year is a conservative estimate of the sustainable yield in that water levels can be lowered below the top of the perforation for short durations and District Wells 2 and 3 could be reconstructed with lower perforations thereby allowing more pumping and lower water levels. This is 167 af greater than the combined pumping by the District and the Mutual of 539 af during the year 2000.

Maximum Pumping of Existing Wells and New Wells

A series of groundwater model simulations were performed to estimate the basin's sustainable yield, defined as the maximum amount of pumping from the basin during a drought year that maintains acceptable water levels in existing wells. As with the simulations of maximum pumping of existing wells, the critically dry conditions of 1994 were used to represent drought conditions. Each model run simulated two consecutive years of drought.

Monthly pumping rates for the existing production wells were set to the same levels established from the analysis for Maximum Pumping of Existing Wells. Additional wells were then added to the model at locations thought to provide significant water supply, but may be of a quality that requires treatment. Pumping from the Resort at Squaw Creek Wells 18-2 and 18-3 was increased to take advantage of the idle capacity of these wells.

Additional wells that were considered included District Well 4RII, 4th Fairway Well, Test Hole No. 1, and two new wells in the western portion of the basin. No additional wells were added to the area around the existing cluster of wells in the east parking lot area. From previous analyses it appeared that during a drought, no additional water could be extracted from this area.

Analysis of simulations incorporating District Well 4RII showed that during drought years, pumping this well lowers the water level in District Well 2, sometimes significantly. During normal or wet years, this well is fed by recharge from Squaw Creek. Recharge from Squaw Creek is diminished during droughts, however, and this well effectively takes water from District Well 2. District Well 4RII was thus removed from further drought year analyses.

As detailed in the Appendix, an additional sustainable yield analysis with the updated model was conducted after relaxing two important assumptions contained in the previous analyses. This final analysis again looked at the sustainable yield available from both existing and new wells. The significant assumptions changed for this analysis included:

1. The perforations in District Well 2 were assumed to be lowered by 15 feet. This could be accomplished by sleeving the existing well or reconstructing the well since it is over 40 years old. This assumption will lower the acceptable simulated water level by 15 feet. The previously assumed minimum water level in District Well 2 was 6177 feet msl. The new assumption results in a minimum water level of 6162 feet msl for District Well 2.

2. It was assumed that wells could provide different percentages of water throughout the year. In the previous analyses, the pumping rates for all wells were increased and decreased by the same amount each month. A 20% increase in total pumping was attained by all wells increasing production by 20%. In the final analysis, all wells can be increased or decreased independently.

As with the second analysis described above, two consecutive critically dry years were simulated in each run. The 1994 hydrology was used to simulate the hydrology of a critically dry year.

The simulations suggest that it will be difficult for the District to meet the estimated buildout demands during critically dry years. The final pumping rates represent approximately 73 percent of the buildout demand identified in Table ES-1, which represents about 58 percent of the demand if full buildout of the 1983 General Plan and Land Use Ordinance were permitted. This is primarily because of the lack of summertime recharge from Squaw Creek during drought years. This is equivalent to an annual sustainable supply of 1,524 af per year. These results assume the lowering of the perforations in District Well 2 and a well efficiency of 70 percent; lower well efficiencies may result in less available supply. Of this total sustainable yield, pumping by each valley entity based on the assumptions in the simulations is shown in Table ES-4.

Table ES-4. Sustainable Yield Analysis – Assumed Pumping Rates, (acre feet)

Pumping Entity	Annual Pumping Amount
Squaw Valley Public Service District	1,091
Squaw Valley Mutual Water Company	172
Resort at Squaw Creek	261
Total Sustainable Yield	1,524

ALTERNATIVE WATER SUPPLIES EVALUATION

The District's estimated annual demand at buildout is 1,628 af. The sustainable yield of the groundwater basin was estimated to be about 1,524 af per year. The District's portion of the sustainable yield has been assumed to be about 1,100 af as was shown in Table ES-4. The District's horizontal wells in an average year produce up to 45 af. They will probably produce about half that amount during a drought year. Therefore, the total supply available to the District is about 1,120 af. A supply of 1,120 af will provide about 69 percent of the District's buildout demand during a critically dry year. The remainder must be supplied from other sources, or the demand during a critically dry year be reduced by about 31 percent through conservation or by limiting development. The supplemental production capacity required to meet District maximum day demands at buildout is about 1,600 gallons per minute.

The District has recently passed a water conservation ordinance that includes the curtailment of demands during a critically dry year. The ordinance requires that normal demands be reduced by at least 20 percent during a drought emergency. Increased demand management needs to be part of the final solution for meeting future demands in the valley.

Facilities must be identified to supply both the projected annual and maximum day demands. The alternative water supplies using sources inside the valley and outside the valley identified and evaluated as part of this study were:

- Additional Squaw Valley Wells
- Springs East of the Truckee River
- Truckee River Wells
- Alpine Springs County Water District

Each of these alternative supplies have been investigated and evaluated in terms of their capability to meet some if not all of the increased water supply needs.

Additional Squaw Valley Wells

Four to six wells located within Squaw Valley are required to supplement the production from existing wells to meet projected maximum day demands at buildout. It is assumed that each well will have a production capacity of between 100 to 400 gpm.

The pumping capacity from the wells that will continue to be considered as elements of the Squaw Valley Groundwater Alternative are summarized in Table ES-5. Their total pumping capacity is 1,600 gpm, which equals the amount of additional supply required at buildout. It has been shown in the sustainable yield analysis that these wells in combination with the existing District wells can produce the sustainable yield of the basin without adversely impacting water levels.

Table ES-5. Wells Included in Squaw Valley Groundwater Supply Alternative

Well Name	Assumed Pumping Capacity, gpm	Treatment Required
Well 4R II	400	No
Condo	300	Yes
4th Fairway Well	100	Yes
New Well 1	300	Yes
New Well 2	300	Yes
Wells 18-2 and 18-3	200	Yes
Total Pumping Capacity	1600	

Wells to be included in the Squaw Valley Groundwater Supply Alternative are shown on Figure ES-3. The water from all the new wells, except Well 4R II, must be treated to remove iron and manganese. It is assumed that supply from the horizontal wells located near the 4th Fairway well will be incorporated into pipe systems to bring the supply to the treatment plant. Pipelines to deliver water to the proposed treatment plant are also shown on Figure ES-3. An iron and manganese removal treatment plant has been evaluated and the preferred site for the plant based on a study performed by the District is at the existing District office site, just behind the existing office building. Section 8 provides the details of the evaluation and recommendations on the proposed treatment plant. The treatment plant would have a nominal capacity of 1,400 gpm or 2 mgd.

Springs East of The Truckee River

A local supply could be developed using the springs located about 4,500 feet southeast of the intersection of Squaw Valley Road and Highway 89. It was found that there is limited recharge in this area because of the impaired vertical permeability within the overlying volcanic rock. In addition, TCPUD used the area above the springs for many years to dispose of their primary treated wastewater effluent. DOHS has reviewed the use of this supply and has serious concerns about the water quality from these springs. District staff have recently visited the old spring collection boxes and found them to be in disrepair and producing a very low flow. It did not appear that these springs would provide a significant supply of water to the District, and their ability to be permitted by the DOHS without treatment is unlikely. This supply alternative has been dropped from further consideration for these reasons.

Truckee River Wells

The District may be able to divert surface water from the Truckee River. The Truckee-Carson-Pyramid Lake Water Rights Settlement Act of 1990, Public Law 101-618, includes the settlement of water rights claims on the Truckee River between the State of California, the State of Nevada and the Fallon Paiute-Shoshone Indian Tribe. However, in drought conditions spanning several years, releases from Lake Tahoe could be eliminated in the summer when the lake level drops below the natural rim elevation. Hydrologic studies for the Truckee River Operating Agreement (TROA) show that surface water could be available for diversion by the District during over 90 percent of the time. Diversion of water during droughts that are sustained for several years, such as the 1988 to 1992 drought, would probably be curtailed. Use of this source of supply would have to be coordinated with other water rights holders during extended dry periods to provide any supply to Squaw Valley. Therefore, consideration of this alternative as a firm water supply to sustain additional development was eliminated.

Alpine Springs County Water District

Alpine Springs County Water District (ASCWD) is located about 1.5 miles south of Squaw Valley. ASCWD's water supply facilities include four springs, two wells, and two snow production wells within the Bear Creek Valley. The combined capacity of the springs and wells is estimated to be about 1,100 gallons per minute according to a 1998 Water Audit prepared by ECO:LOGIC Engineering with all facilities operating. The actual capacity varies slightly between summer and winter conditions. The maximum day summer water demand of ASCWD is estimated to be about 400 gallons per minute, leaving as much as 700 gallons per minute of possible idle supply capacity.

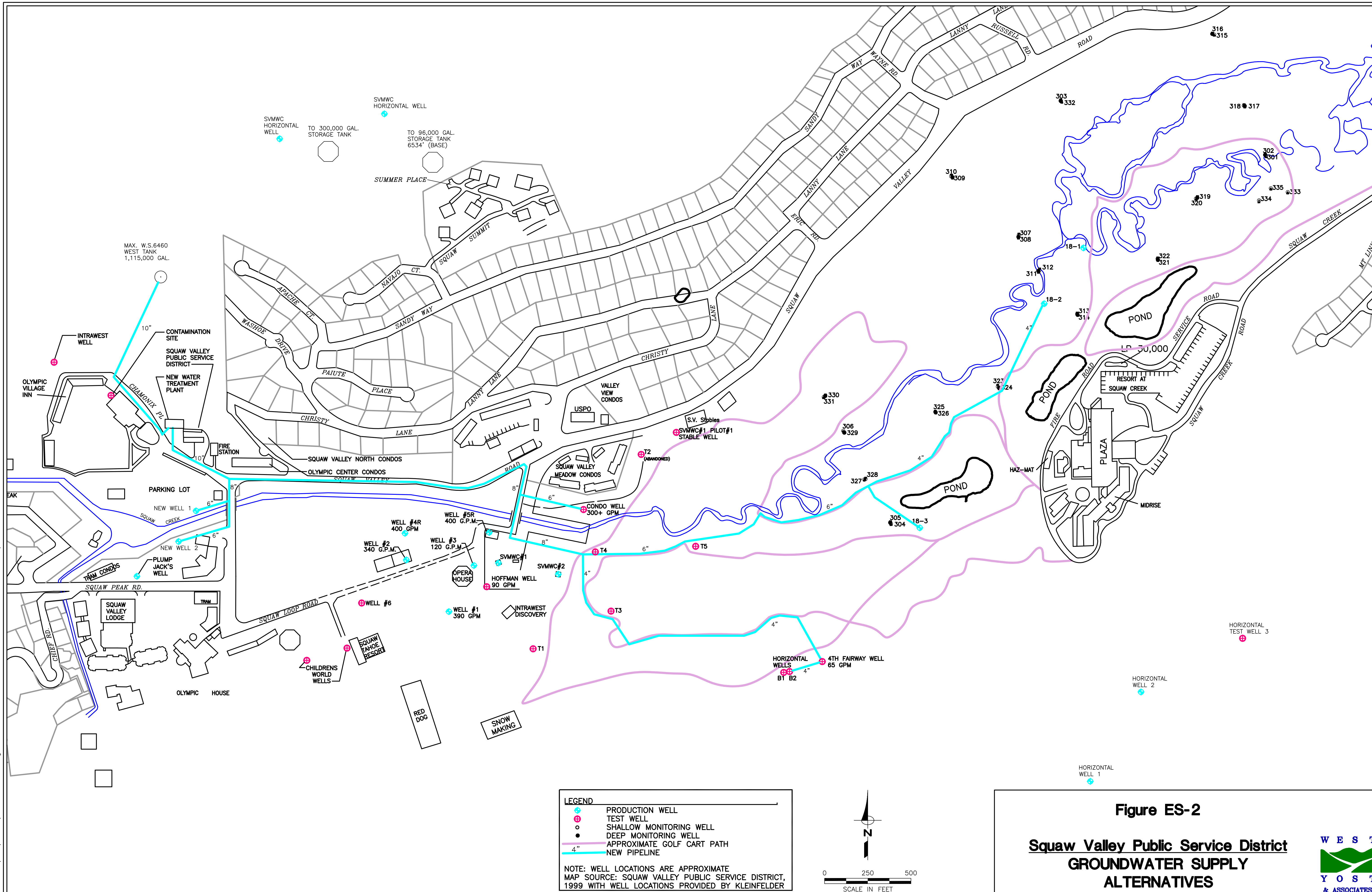


Figure ES-2

Squaw Valley Public Service District
GROUNDWATER SUPPLY ALTERNATIVES

However, it is not known at this time how much water would be available from ASCWD during any given year. The snow making wells are not used in the summer months, but the long-term sustainable yield from the wells, constructed in fractured rock has not been proven. Therefore, consideration of this alternative as a firm water supply to sustain additional development was eliminated.

The only alternative that provides the needed maximum day demand production and has a significant sustainable annual yield is the addition of new wells and an iron and manganese treatment plant in Squaw Valley. The Squaw Valley Wells alternative cannot provide sufficient supply to meet full buildout demands, without implementation of demand reduction measures. However, this alternative, coupled with a 20 percent reduction in demands, could meet the District's supply needs in a critically dry year.

WATER TREATMENT PLANT ALTERNATIVES

The groundwater quality data obtained from sampling production and test wells in Squaw Valley was reviewed to determine the general water quality characteristics of various groundwater sources, and establish specific treatment requirements to remove contaminants of concern. This evaluation was a preparatory step to identify the alternative treatment process that meets the goals of the study. The goal is to identify those treatment processes that can provide water that meets or exceeds current drinking water standards, provides flexibility for expansion and future treatment needs, and is cost effective. The recommended processes were further defined and conceptual treatment plant layouts and cost estimates prepared and the recommended treatment alternative was identified.

Treatment Process Evaluation

The favored treatment processes for removing iron, manganese and perhaps arsenic from the groundwater have been reviewed. A preliminary screening of potential processes was completed prior to identifying the favored processes. The capability of meeting treated water quality objectives applicable to water quality conditions in the valley were the major factor leading to the selection of viable treatment processes. The three process alternatives considered were:

1. Pressure greensand filtration
2. Ozone oxidation/gravity filtration
3. Membrane filtration

Pressure Greensand Filtration

The conventional method for removal of iron and manganese from groundwater involves oxidation, generally with chlorine, chlorine dioxide, potassium permanganate or perhaps ozone, followed by filtration. Pressure filters are generally used in iron and manganese filtration applications. Often where both iron and manganese are present at concentrations above the MCL, the manganese greensand filtration process has certain advantages that make it an attractive process. Consequently, in the ensuing cost analyses, the conceptual design was based upon the use of the manganese greensand filtration process. Costs received from a vendor of iron and manganese filtration treatment systems were used for the greensand pressure filtration alternative.

The greensand filtration process, although generally used where iron and manganese is the principal concern, also has the ability to perform effectively as a filtration media for arsenic removal. The pressurized greensand filtration process could be used for arsenic removal wherein oxidation of the reduced forms of arsenic generally found in groundwater would be accomplished with chlorine and potassium permanganate or perhaps ozone, and then removed through filtration. To improve arsenic removal, a small amount of a primary coagulant such as aluminum sulphate (alum) or ferric chloride could be added to remove the arsenate precipitate.

The inline filtration process (pressure greensand filters) has some limitations when used for removal of microbial contaminants of surface water origin. For example, the DOHS discourages the use of pressure filters using the inline filtration process for surface water sources because of concern for turbidity breakthrough caused by the generally higher pressure used with pressure filters. Where the inline process is used, however, DOHS restricts the pressure filter rates to no more than three gpm per square foot of filter area. Recognizing that there is a possibility that new wells may possibly fall under the influence of surface water contamination, a filtration rate of 3 gpm per square foot was selected to size the pressure filters for this principally iron and manganese removal application. If a surface water source becomes available in the future, a filtration rate of 3 gpm per square foot would also comply with the current design standard for the use of pressure filters for surface water treatment. It is likely, however, that for a direct surface water treatment application, or in a situation where groundwater wells could become contaminated with surface water inflow, an additional treatment barrier would be needed to comply with DOHS standards.

A process such as ultraviolet (UV) sterilization could be applied to the filtered water to meet possible cryptosporidia removal standards requiring a higher level of disinfection than could be provided solely by chlorination. Ultraviolet light sterilization has been found to be very effective for inactivation of cryptosporidia oocysts. UV treatment would probably also be the most effective and least expensive addition to the pressure filtration process to meet drinking water standards. Consequently, space should be provided in the treatment facilities for the addition of an ultraviolet disinfection process should it become necessary in the future.

Ozone Oxidation/Gravity Filtration

A treatment process alternative using ozone and gravity filtration was considered because this process would have the capability of oxidizing and removing iron and manganese, and inactivating and removing any microbial contaminant of surface water origin. Further, this complete treatment process, supplemented with ozone, would also be able to effectively treat any quality surface water supply while meeting all current and anticipated future drinking water standards. The ozone/gravity filtration process is significantly more complex than the pressure filtration alternative, but would have substantially greater treatment capability. The process could very adequately treat all groundwater sources in the basin, and effectively remove iron, manganese, and arsenic. Consequently, a process alternative based upon the ozone/gravity filtration alternative furnished in a factory-built package plant by U.S. Filter (Trizone process) was considered in the evaluation.

Membrane Filtration

A preliminary assessment of the feasibility of using membrane filtration for this application was also completed. Discussions with membrane suppliers indicated that iron and manganese would have to be first oxidized with a combination of chlorine and potassium permanganate prior to membrane filtration. The membrane process, thus offers no advantages with respect to possible elimination of chemical treatment requirements.

Following this preliminary screening, the first two process alternatives were retained, and the membrane process was eliminated from further analysis. Conceptual designs were then developed for the pressure greensand and the ozone oxidation/gravity filtration process. The principal components of each system alternative were identified and preliminary design criteria developed for the processes. These criteria and the conceptual design information were then used to prepare projected construction costs for a treatment facility designed around one of these two treatment processes.

Treatment Process Recommendation

Evaluation of the two most appropriate treatment process alternatives indicates that a treatment facility designed around the pressure greensand filtration process for iron and manganese removal would be the preferred alternative. It appears that the treatment requirement is primarily for iron and manganese removal and pressure filtration is substantially less costly than the other alternatives. This treatment process can also remove arsenic should levels in the groundwater rise above the MCL. Only if a surface source becomes available would the ozone/gravity filtration modular treatment alternative process be more suitable than the recommended process. However, the pressure greensand filtration system can be upgraded with UV treatment of the filtered water permitting DOHS to approve the use of the process for treating a surface source or a groundwater under the influence of a surface source. The estimated cost to add UV sterilization to the pressure greensand filtration treatment system would probably be about $350,000. Space has been provided in the conceptual facility layout to accommodate future UV disinfection equipment. Operation and maintenance costs favor this alternative over the ozone/gravity filtration alternative by a wide margin.

RECOMMENDED WATER SUPPLY AND TREATMENT FACILITIES

As stated previously, the District is expected to need an additional 1,600 gpm in water production capacity to meet buildout water demands. The District's annual groundwater supply requirement at buildout is estimated to be 1,605 af. This amount assumes 23 af are supplied by the District's horizontal wells. The Squaw Valley Groundwater Supply Alternative provides the needed production capacity to supplement the existing wells to result in sufficient supply capacity to meet the maximum day demand at buildout with the largest producer out of service. The groundwater basin has been shown to have a sustainable yield of about 1,524 af with the District's shared amount of it being 1,100 af. The groundwater supply alternative develops the full sustainable yield of the basin. To meet build out demands, additional supply from outside the valley would be needed. The Squaw Valley groundwater supply will be adequate if the future development is limited or the District's conservation ordinance is enforced in critically dry years to reduce demands by 31 percent. It is anticipated that five new wells will be constructed to provide the needed production capacity. In addition it is assumed that the idle capacity of the Resort Wells 18-2 and 18-3 can be made available to the District. The recommended facilities are shown on Figure ES-2. This recommended plan is to be used as a guide and can only be implemented if well sites can be acquired and the expected production is developed.

A supply connection with Alpine Springs County Water District (ASCWD) is recommended at the time the Homesites at Squaw Creek #2 subdivision is constructed. The intertie with ASCWD could be used under emergency conditions or provide water to the District on a regular basis if idle supply capacity is available and an agreement can be negotiated between the districts. The sizing of the intertie pipeline would be determined at the time the subdivision project moves ahead and the amount of water available from ASCWD is known.

It is also recommended that a water rights application be filed immediately for a surface water diversion from the Truckee River. This would establish a placeholder for this supply so the District will have some flexibility in the future should conditions change. Additional investigations should be performed to determine how the reliability of the supply could be enhanced to provide benefit to Squaw Valley during drought years. The treatment plant can easily be retrofitted with UV disinfection equipment to be able to treat this supply.

The recommended treatment plant is a pressure greensand filtration system located in a new building in back of the existing district office complex. The plant is envisioned to have a buildout capacity of 2 mgd, with the building sized to accommodate treatment facilities capable of treating up to 4 mgd. This will provide assurances of being able to build a 4-mgd treatment plant if the need arises to treat more of the groundwater supply or a surface water source. The 2 mgd treatment plant is comprised of two pressure filters with a treatment capacity of 1 mgd each. This configuration lends itself to phasing the facility by installing one filter first and waiting until demands increase before installing the second filter and associated equipment.

Recommended Improvements

The recommended facilities to be constructed to meet buildout demands are shown on Figure ES-5. The facilities and estimated capital cost are shown on Table ES-6. These costs do not include the cost of the land for the wells or the cost for pipeline easements not in public rights-of-way. These costs, when they are identified, will need to be added to the costs shown in Table ES-6.

Table ES-6. Recommended Water Supply Facilities and Estimated Capital Costs [a]

Item	Unit Cost, dollars	Estimated Cost, dollars
2 mgd Water Treatment Plant for Iron and Manganese Removal	Lump Sum	2,875,000
5 New Wells	425,000	2,125,000
3,300 feet of 4” pipe	40	132,000
3,500 feet of 6” pipe	60	210,000
2,100 feet of 8” pipe	80	168,000
1,700 feet of 10” pipe	100	170,000
Total Construction Cost		5,680,000
Engineering, Legal, & Admin Costs @ 20%		1,136,000
Total Project Cost		6,816,000

(a) Costs are in 2001 dollars

Phasing of Improvements

The facilities included in the recommended supply plan can be phased as demands increase. The initial facility that should be constructed is the drilling of the replacement well for Well 4R. Wells at this location have proven to be good producers with good water quality that does not need treatment. It is expected that this well will be similar to Well 5R in terms of production and will provide the District with additional pumping capacity and reliability in meeting peak demand periods with the largest producer out of service. Should it be found that Well 4RII produces groundwater under the influence of surface water, it can be connected to the treatment plant for treatment and disinfection; however, UV disinfection equipment would need to be added at the treatment facility.

The next activities to be undertaken include further exploration and testing of potential well sites to identify the next set of wells to be added to the system. Test Wells 4 and 5 should be test pumped to identify the source of poor water quality. Water bearing strata should be isolated and water quality samples obtained. The results of the pump testing and water quality testing should identify the feasibility of developing production wells at these locations of suitable quality for domestic purposes.

The potential for the use of the Resort at Squaw Creek Wells 18-2 and 18-3 should be investigated. The wells should be retrofitted with level monitoring equipment and the pumping amounts and drawdown should be monitored to ascertain the production capabilities and potential for use as a year-round supply for the District.

A study should be performed to identify the improvements that would be necessary to include the production from the horizontal wells near the 4th Fairway Well into the supply system. The horizontal wells' production could be pumped into the existing distribution system pipeline that serves the Resort or added to the production from the 4th Fairway Well delivered to the water treatment plant.

The treatment plant can be phased to provide just 1 mgd (700 gpm) of capacity with room in the building to provide another pressure filter later. The cost savings for phasing the treatment would only be about $500,000, which is the cost of a pressure filter; associated piping and chemical feed system. At least two additional wells will need to be added to provide the 700 gpm of production capacity, although, the treatment plant could begin operation with only one well. The most likely wells to pursue initially are the Condo Well and the 4th Fairway Well. The Condo Well was drilled in 1992 and will require some cleanup and development work before a pump and motor can be installed. The 4th Fairway Well will need to be redrilled using a larger casing.

In addition to these wells, the connections to the Resort Wells 18-2 and 18-3 should be made. The District will need to develop an agreement with the Resort at Squaw Creek for the use of the idle well production capacity and an easement for the pipelines to deliver the water to the eastern edge of the parking lot near Well 5R. The rest of the pipeline easements will also need to be acquired to connect the pipeline to the public rights-of-way along Squaw Valley Road. The combination of the two new wells and the idle capacity of the Resort wells will provide at least 700 gpm of pumping capacity to the first phase of the treatment plant. A total of 8,500 feet of the 4-inch to 10-inch pipelines shown on Figure ES-2 will also need to be constructed in the initial phase.

The treatment plant would be expanded when demands increased and the two new wells in the western parking lot are needed. The piping and much of the ancillary support structure for the new pressure filter will have been constructed as part of the initial phase of the treatment plant project. The wells will need to be sited, test drilled and the property acquired prior to the construction of the wells. The 800 feet of 6-inch and 8-inch pipelines connecting the wells to the treatment plant will also need to be constructed.

The recommended plan provides the District with flexibility for developing the needed supplemental water supply in terms of the number and location of wells required to meet future demands. The identified locations are based on previously drilled test holes. If other sites are found to be better well locations and can be developed to produce 300 to 400 gpm while maintaining or increasing the basin's sustainable yield, then they can substitute for the any of the wells. Having a pipeline through the golf course provides opportunities to develop wells along that route if reasonable water quality and production can be developed. Supply from additional horizontal wells along the valley's south side or drilled wells in the east end of the valley could be delivered to the treatment plant via the golf course pipeline. The area between the Resort at Squaw Creek and the existing well field area has been identified as the most promising target area for future successful production wells.

GROUNDWATER MANAGEMENT PLAN

Groundwater is a precious resource in Squaw Valley that is and will continue to be relied upon as the water supply for the valley. The objective in developing a groundwater management plan is to provide a long-term strategy for sustainable groundwater basin use for all entities relying on this water supply. Development and implementation of the recommended groundwater management plan will allow the entities pumping from the basin to effectively manage the basin with respect to the quantity and the quality of the water pumped. These entities should work closely to manage this resource. The goals of the plan are to keep the pumped amounts within the sustainable yield of the basin and protect the wells from potential contamination.

The groundwater model developed for this project provides the District with a tool for making management decisions on the use of the groundwater resource in Squaw Valley. It has been shown to reasonably simulate the geohydrology of the basin. It is the best tool available for estimating the effects of various pumping and recharge scenarios, and should be used for planning future groundwater basin management. Pumping rates from existing wells, placement of new wells, and effects of pumping on streamflow can all be studied with the existing model The use of the model will improve any future planning studies.

As with all groundwater models, additional data will help validate the results and direct modifications to uncertain model parameters. Data that will be particularly helpful include measured streamflow entering and leaving the basin and additional water level data from or near the pumping wells. The collection of this data will allow more accurate calibration of the basin model, particularly in terms of the annual water balance.

The definition of sustainable yield is the maximum amount of pumping that can be pumped from the groundwater basin during a critically dry year without significantly impacting water levels in existing wells. The definition of significant impact to existing wells is not lowering the pumping levels to below the top of the perforations. A pumping scenario has been identified that maximizes monthly pumping of existing wells without lowering the pumping levels below the top of the perforations. This monthly pumping pattern should be used by the District as a guide for managing the pumping from the groundwater basin.

The water levels in each well should be monitored and pumping adjusted so levels remain above the top of the perforations. During dry years, the need for increased water conservation measures should be coordinated with ongoing review of precipitation, stream flow and pumping levels and amounts. The District has recently enacted a water conservation ordinance that calls for restrictions on the use of water in the event of any threatened or existing water shortage. The ordinance also provides for the implementation of a mandatory reduction in demands by 20 percent or more during a critical water supply shortage. In any year that experiences precipitation that is below average amounts of about 55 inches per year, the District should review the condition of the groundwater basin and the need for increased water conservation. The results of this study have shown that the water supply in Squaw Valley is a limited resource. The initiation of water use restrictions should be considered in the fall of each year that the groundwater levels are at or below the top of the perforations in Well No. 2. The use of groundwater for snowmaking should also be critically reviewed if the groundwater levels have not substantially recovered prior to the onset of the ski season.

A program to obtain additional information for the model and to manage the basin has been identified. The program includes monitoring water levels of all pumping wells, establishing stream gages on Squaw Creek and providing groundwater protection by abandoning unneeded existing wells and establishing a monitoring network in the western basin. The benefits to the District and Mutual Water Company from a more complete monitoring program include the establishment of an early warning system in case of groundwater contamination from spills in the production well capture zone, and to obtain more information for the refinement of the groundwater model. The monitoring network in the western basin would include real-time water level monitoring in all production wells and a ring of monitoring wells upstream of the production wells where water quality samples could be collected and tested quarterly to identify the presence/absence of contaminants that may have entered the groundwater. The early warning monitoring and testing will give the District and Mutual time to react to the contamination and maintain adequate water supplies with wholesome quality for their customers. The information obtained from the monitoring program will also be useful in updating and improving the groundwater model with more complete information on groundwater levels in response to pumping and stream flows entering and leaving the valley. An updated model would provide additional confidence in using the model for making decisions on management of the basin and to verify and refine the estimate of sustainable yield of the resource. The recommended activities to be undertaken as part of the groundwater management plan are listed below.

1. Identify, locate and map test wells and monitoring wells in the western end of valley.
2. Determine which wells may be used for monitoring and which need to be abandoned.
3. Complete the well SCADA system to monitor pumping and level at all wells. Expand system to other pumping wells in valley, if possible.
4. Properly abandon all unnecessary wells and equip others for monitoring levels or periodic sampling to identify possible contamination plume movement.

5. Identify other locations for additional monitoring wells, construct wells and install monitoring equipment.
6. Install three stream gages within Squaw Creek; one on each major branch entering the west end of the valley and one at the upstream side of the Squaw Valley Road bridge.
7. Establish an ongoing monitoring program for the collection of surface water and groundwater data and to monitor quality of water in the production wells capture zone. Update the groundwater model when sufficient data has been collected.
8. Prepare a groundwater management report consistent with the requirements of AB 3030 and submit it to the State Department of Water Resources. Apply for grant funds to support ongoing groundwater management program activities.
9. Develop public outreach and education program as described in the Groundwater Protection Plan in Section 3.

The cost of the above-described activities is estimated to be about $250,000 and has been funded under the AB 303 Local Groundwater Assistance Fund Grant Program in June 2002. The collection of stream flow information will be included in the model update scheduled in the winter of 2003-2004. The implementation of the groundwater management plan will directly benefit all users of the Squaw Valley groundwater basin. While the District and Mutual are not required to develop a groundwater management plan as defined by AB 3030, the State's groundwater management planning legislation, the implementation of a plan provides a definitive program for the collection and use of monitoring data to help ensure the maintenance of the quality and quantity of the local groundwater resource. With this program, informed decisions in managing the available groundwater can be made to assure an available supply in the future.

SECTION 1. INTRODUCTION

PROJECT BACKGROUND AND OBJECTIVES

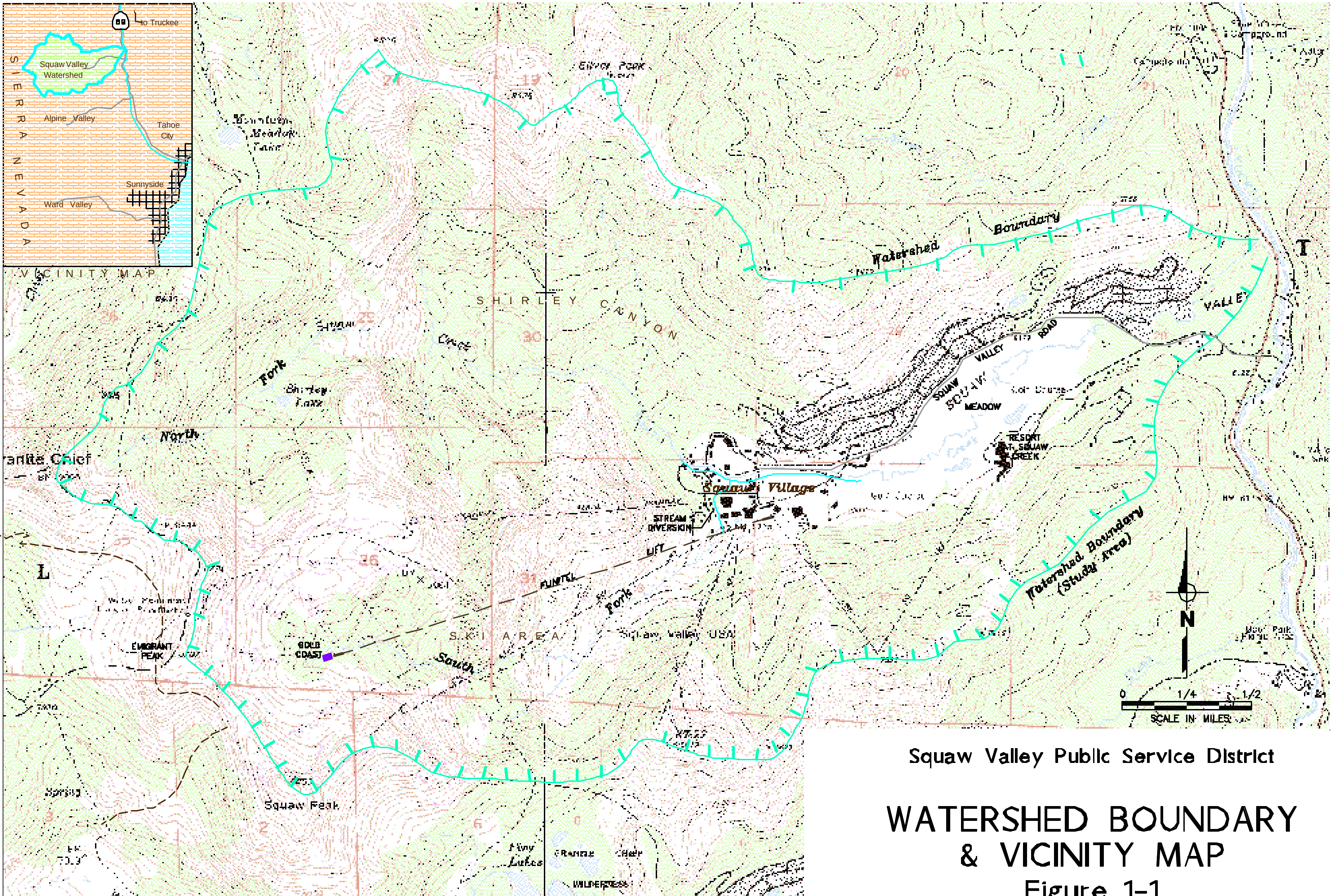
In the past ten years, there has been a significant development within Squaw Valley and as a result, water supply requirements have increased. The Squaw Valley Public Service District's (District) 1993 Water Master Plan, which has been used by the District to guide the orderly expansion of the water system, included recommendations for the collection of additional hydrogeologic data and the development of a groundwater management program for the valley. The scope of the recommended study also included the determination of the sustained yield of the basin to confirm adequate water supply will be available to meet the increased demands associated with the buildout of Squaw Valley's General Plan.

Over the past twenty years several studies have estimated the yield of the groundwater basin. These studies were based on very limited information and many simplifying assumptions to arrive at the estimated recoverable yield of the basin. The experience during the sustained drought period of 1987 through 1992 showed that the groundwater resource is capable of supplying current demands without exhibiting sustained overdrafting. However, there is concern about the capability of the groundwater basin to provide sufficient good quality water to supply increasing future demands. In addition, there is the need to replace the existing wells that are more than 40 years old and require increasing rehabilitation efforts to maintain their water production capabilities.

The groundwater basin west of the golf course provides good quality water that meets all primary and secondary drinking water standards. The watershed boundary and location map of Squaw Valley are shown on Figure 1-1. The groundwater in the rest of the basin is of lower quality due to the geology and highly mineralized geothermal springs. A number of wells have been drilled recently that have produced water with high iron and manganese concentrations that will require treatment prior to being distributed for consumption. The District is also concerned that the current supply meet future drinking water standards for radon and arsenic.

Treatment facilities will be required for several of the wells proposed to supply water for domestic use. The sustainable yield analysis should include identification of pumping limitations to avoid impacting existing production wells and eliminate the risk of drawing lower quality water into the west end of the valley. There are also groundwater contamination sites in the western end of the valley that should be analyzed to avoid potential contamination of the municipal wells.

The District is proceeding in a diligent manner to identify the needed water supply and treatment facilities to meet increasing demands. The ability to continue to serve all the District's needs from the western end of the valley without treatment is limited. The District needs a plan to guide the responsible development of additional water supply sources and treatment facilities to meet the demands of their customers. The results of the Groundwater Development and Utilization Feasibility Study were used to identify the sustainable yield of the basin, develop a watershed management plan to protect the resource, site new well locations and prepare a recommended capital improvement program for the District to continue to meet the water supply needs of the valley.



Squaw Valley Public Service District

WATERSHED BOUNDARY & VICINITY MAP

Figure 1-1

Additional information on the groundwater basin has been recently compiled that clarifies some of the complexity and unknowns of the water bearing strata in the valley. From the recent well drilling and test pumping efforts, it appears that the better quality groundwater is located near Squaw Creek, which may directly recharge the supply aquifer. The information obtained from the recent pump tests was thoroughly reviewed along with the significant volume of geologic and water supply information that has been assembled over the past three decades.

SCOPE OF STUDY

The scope of work for this feasibility study to develop supplemental water supplies included the thorough evaluation of the surface and groundwater resources in Squaw Valley, development of a water resources protection and management plan, evaluation of alternatives to meet the District's future water demands, identification of new well sites that can supply water either without and with treatment, and capital improvement program recommendations including wells, piping and treatment facilities.

The scope included the following activities:

- Review and analyze existing data including background data on historical precipitation, runoff and water use in the valley, hydrogeologic information on the groundwater basin and water quality characteristics, and information on known contamination sites and plumes in the valley.
- Install test holes at selected locations to provide additional information on the hydrogeologic conditions in the valley and perform pump and water quality tests to confirm their potential for future wells sites.
- Perform a watershed sanitary survey and source water assessments to provide information on potential contamination sites in the valley and develop a groundwater protection plan.
- Develop a computer model of the groundwater basin to create a tool to analyze the basin's sustainable yield and manage the valley's precious water resource.
- Estimate the sustainable yield of the watershed and basin using the groundwater model. Develop a groundwater management plan for pumping needed to meet the future demands.
- Update the estimate of future water demands at buildout of the General Plan.
- Identify and evaluate alternative water sources to meet the ultimate water demands including locating additional wells in the valley and importing water from nearby sources. Investigate treatment needs and identify the recommended treatment method. Estimate costs to construct alternative water supply sources.
- Compare alternative sources of supply and define recommended improvements. Identify actions required to implement the phased improvements. Complete predesign of the initial set of improvements after the completion of the draft report.
- Prepare a report that summarizes the work performed as a part of this feasibility study.

The following report sections summarize the work accomplished as part of the feasibility study and present recommendations for the protection, development and use of the water resources in Squaw Valley.

2003 GROUNDWATER MODEL UPDATE

Since the completion of the Squaw Valley Groundwater Development and Utilization Feasibility Study Report, dated October 2001, the District has undertaken an update of the groundwater model and analysis of the sustainable yield. New information had become available since the development of the original computer model and the model was updated using newly created mapping of the eastern two-thirds of the valley and the recently collected geologic and hydrologic data. The updated model was then recalibrated to the original calibration period and data. The updated model was then used to develop new estimates of sustainable yield using just existing wells and then adding new wells to meet future water requirements. This updated report incorporates descriptions of the work to update the model and the results of the new sustainable yield analyses.

SECTION 2. REVIEW AND ANALYSIS OF EXISTING DATA

The initial task in evaluating the available water resources in Squaw Valley was to collect and review available data on the occurrence and use of surface and groundwater in the valley, the physical features and characteristics of the groundwater basin and the numerous petroleum hydrocarbon spills and naturally occurring elements in the valley. The data were used in subsequent tasks including the development of a source water protection plan and watershed sanitary survey, a groundwater hydrology model of the valley, an assessment of the available water resources to meet anticipated demands, and ultimately a recommended capital improvement program to develop additional water supply to meet District's future needs.

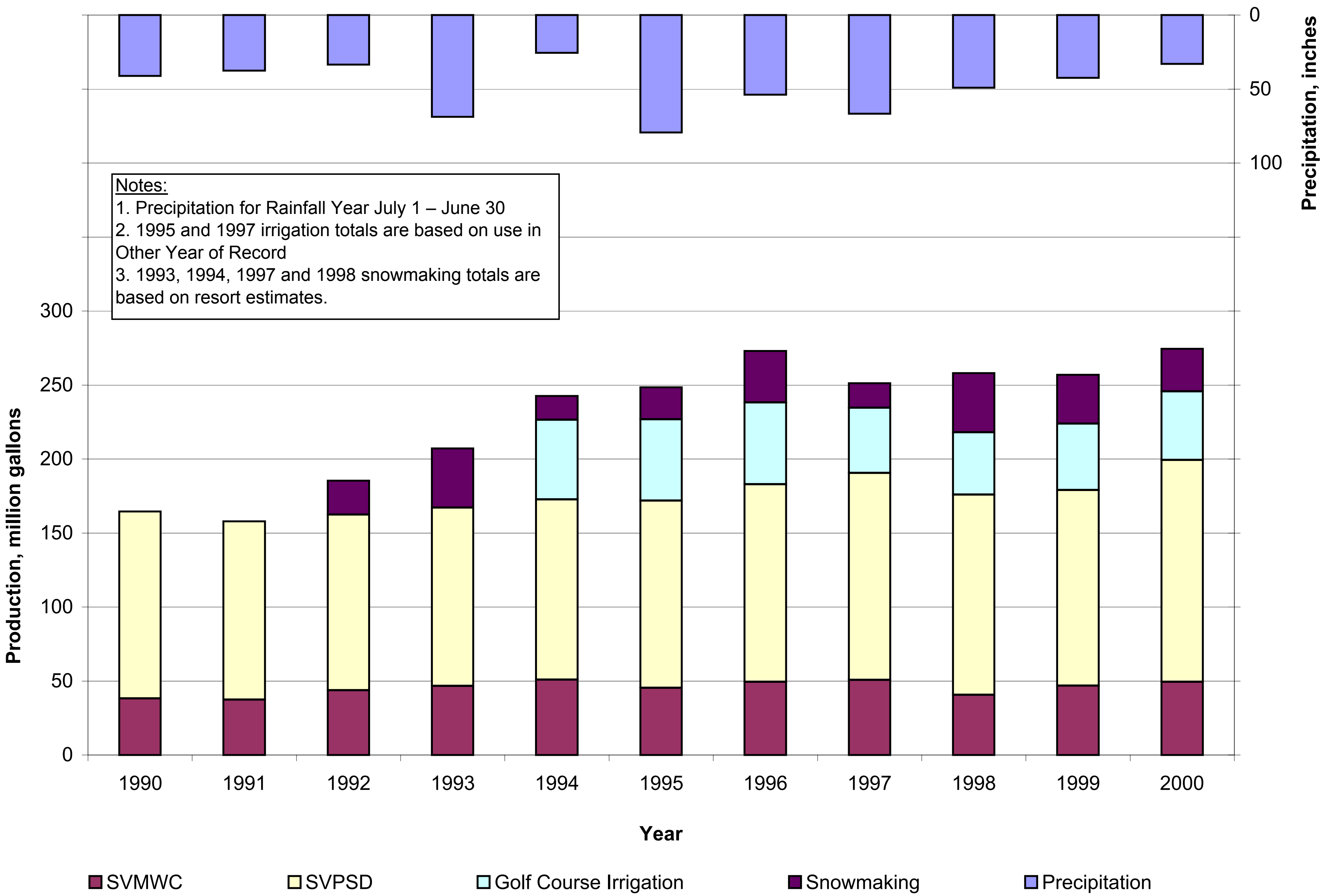
OCCURRENCE AND USE OF SURFACE AND GROUNDWATER

A significant amount of information was available on the historical precipitation, stream flows at nearby gages, and pumping of groundwater to meet municipal, irrigation, and snow making demands. The valley's historical precipitation and water production records maintained by public and private entities was collected and reviewed. A summary plot showing the annual precipitation and water production from 1990 through 2000 is shown on Figure 2-1. This plot shows the annual precipitation measured at the fire station, and the water production by the District and Squaw Valley Mutual Water Company (Mutual) for domestic and municipal use, the Resort at Squaw Creek for golf course irrigation, and the Resort and Ski Corp for snow making. This data has been summarized on an annual and monthly basis and was used to estimate monthly flows in Squaw Creek and as input to the groundwater model. Data on water production and use on the upper mountain ski area was not available for use in this study. Information on current and future use above the valley is not included in this report.

PHYSICAL FEATURES AND CHARACTERISTICS OF THE GROUNDWATER BASIN

Hydrogeologists with Kleinfelder prepared three technical memoranda (TMs) associated with the collection, review, and compilation of background data on the hydrogeology, aquifer characteristics, well construction, and water quality of the groundwater basin in Squaw Valley. The information summarized in these TMs was the primary source of data used in the subsequent tasks performed as a part of this feasibility study. There have been numerous studies, reports, investigations, explorations, and wells constructed in the valley that provide a significant amount of data useful in the development of a computer model of the basin and in analyzing the valley's water resources. During the past forty years numerous wells and borings have been installed and countless water and soil samples collected and analyzed along with pumping tests. These sources provide a large body of data to define the characteristics and occurrence of water bearing strata in the groundwater basin.

FIGURE 2-1
Squaw Valley Water Production and Precipitation, 1990-2000



Review of the published and file material was completed and a synopsis of the pertinent information was summarized by Kleinfelder in a TM entitled “Squaw Valley Groundwater Background Data” dated February 15, 2000. Copies of this TM were supplied to the District for their use and future reference. The majority of the information was obtained from the review of reports and file materials generated by Kleinfelder during their past twenty years of involvement in developing groundwater resources in the valley. This TM provided the results from a review and analysis of the data and compiled the basic information on well construction, aquifer performance testing, and water quality in a series of appendices. The TM included discussion of the valley’s geology and hydrogeology setting, the well field hydraulic parameters, and groundwater chemical characteristics. In addition, appendices include detailed information on water quality sampling of the municipal wells, groundwater basin lithology, and water quality sampling for District, Mutual, and Resort at Squaw Creek wells.

The review and analysis of the available data and reports identified a need for additional information. To augment the existing dataset a field data-gathering program was developed. The field program included the drilling of two new test holes and the pump testing of three existing but capped wells. The test holes were drilled at a site just east of the Squaw Valley Meadow Condos and at a site south of the Intrawest Discovery sales office building at the southwest margin of the golf course. The wells were drilled using roto-sonic method equipment, which resulted in the recovery of a complete core sample for the full extent of the boring. Detailed analysis of the lithology at these two locations identified the various layers of gravelly glacial till, fluvial outwash, and lacustrine clays that are characteristic of glacial valleys. This field work was conducted during the late fall of 1999. A separate TM, entitled “Report on Field Activities” dated February 15, 2000 was prepared by Kleinfelder and copies submitted to the District to be available for future reference. This TM described the work performed and summarized the information obtained on the lithology, aquifer characteristics, and water quality.

In March 2000, three additional test holes were drilled and sampled by Kleinfelder. Technical data and associated documentation relating to test holes 3, 4 and 5 installed at the Resort at Squaw Creek from March 7 to 11, 2000 were presented in a third technical memorandum dated June 6, 2000. This memorandum contained a brief discussion of the installation of the test holes, results of slug and development pumping tests, and sampling and water quality test data. The information from the five test holes augments the information contained in the background data technical memorandum.

The TMs described above were submitted to the District for review and reference. The information presented in the TMs was reviewed by and was made available to other project team members involved in the development of the groundwater model and the source water protection plan. This information was the primary source of data for the remaining tasks performed in this feasibility study.

CONTAMINANT PLUME IDENTIFICATION

The other initial task in evaluating the available water resources in Squaw Valley was to collect and review available data on the numerous petroleum hydrocarbon spills that have occurred within the valley and the naturally occurring elements, such as iron and manganese, that have been found in the valley’s groundwater. These sources of contamination could adversely impact the operation of a well field in the western end of the valley particularly when increased production is required. Significant information was available that describes historical activities in the valley that may have led to potential groundwater contamination. File information on known spill sites was also available from the Lahontan Regional Water Quality Control Board and Placer County files. These records were augmented with Kleinfelder's file information from their activities associated with the identification and cleanup of contamination sites.

A separate investigation of known, man-made chemical and natural occurrences of specific elements in groundwater of Squaw Valley was undertaken by Kleinfelder. This investigation identified and evaluated the available information on known chemical contamination sites, primarily releases of petroleum hydrocarbons. The contamination sites were identified from searches and reviewing files from county and state regulatory agencies. Historical photos were also reviewed to assist in identifying the locations of past structures and activities that may have been associated with possible releases of contaminants to the groundwater. A total of 13 petroleum hydrocarbon release sites were identified through this research. The review of the record information on each of these sites was summarized by Kleinfelder in a fourth TM entitled "Known Man-Made Chemicals and Natural Occurrences in Groundwater" dated December 17, 1999.

In addition to the petroleum hydrocarbon sites, the historical locations of sewage treatment and disposal areas were also identified. The areas where septic tanks served residential developments were mapped along with the former leach fields used for the disposal of treated sewage. Naturally occurring inorganic minerals including iron, manganese, arsenic, and radon are also present in the valley's groundwater. The occurrences of these natural elements were identified and maps prepared showing the locations of groundwater containing concentrations of these elements above the primary and secondary drinking water standards.

The TM also includes an evaluation of the risk petroleum release sites pose to groundwater quality and the drinking water supply. A risk matrix of the hydrocarbon releases was developed which identifies the risk rating for each of the 13 hydrocarbon release sites to drinking water supply wells.

Copies of the TM described above were submitted to the District for review and future reference. The information presented in the TM was reviewed and used by other project team members involved in the development of the groundwater model and the source water protection plan and sanitary survey. This information was a significant source of data for the remaining tasks performed in this feasibility study.

SECTION 3. WATERSHED INVESTIGATION, SOURCE WATER ASSESSMENTS AND GROUNDWATER PROTECTION PLAN

Source water protection is the first and foremost barrier required for inclusion in a well-developed multiple-barrier protection and treatment plant for public drinking water supplies. A comprehensive source water protection program can prevent contaminants from entering the public water supply, reduce treatment costs, and increase public confidence in the quality, reliability and safety of its drinking water.

This section presents the findings of a sanitary survey of the Squaw Valley watershed that provides the surface and groundwater supply for valley drinking water. This is followed by a summary of the Source Water Assessments Report and the proposed Groundwater Protection Plan. The valley community and commercial entities depend on these local water resources for their present and future water supplies and have a vested interest in protecting their quality.

WATERSHED SANITARY SURVEY

The watershed sanitary survey was conducted to obtain initial information on existing contaminant sources and to identify development and activities in the watershed that may contribute contaminants to surface water and subsequently to the groundwater resources of the valley. Identification of these sources is an important step toward the subsequent development of a watershed management plan to protect the water resources of the valley so that they may continue to serve the vital community needs. The Watershed Sanitary Survey report was submitted to the District for their use in June 2001.

The purpose for conducting the watershed sanitary survey was to develop contaminant source information to support the Source Water Assessments and the development of concepts for watershed management to protect source water quality at Squaw Valley.

Water treatment processes can reduce concentrations of contaminants, but the cost of treatment and the potential risks of residual contaminants can be high. To maximize public health protection, the best available source water should be obtained and watershed management practices should be designed and implemented to protect water quality.

The water users in Squaw Valley depend on groundwater supplies. The groundwater aquifer is fed by surface water runoff and recharge, and the residence time is relatively short because underlying materials consist primarily of highly porous granitic soils. There appears to be a direct relationship between surface water and groundwater in some areas (as revealed by the results of pump tests) and portions of the basin near the creek may be considered to be under the direct influence of surface water.

Purpose and Objectives

The objectives of the watershed sanitary survey were to:

- Identify key activities on the watershed to be addressed in further detail in the development of source water protection strategies to be included in a watershed management plan
- Address the primary concerns targeted in the Safe Drinking Water Act
- Help convince the State Department of Health Services (DOHS) that reasonable progress is being made toward the identification of surface contaminant sources, control of those sources, and protection of the Squaw Valley groundwater resource

After the survey identified the key watershed sources and activities of concern, a watershed management program was then developed to protect the quality of the surface and ground water sources.

The watershed sanitary survey included the following items:

- Watershed Environment and Water Supply – watershed description, geographic characteristics, existing hydrology, biology, land use, land ownership, and water development and supply system
- Watershed Activities and Potential Sources of Contamination – discussion of urbanized areas, waste management, hazardous materials storage, land management, transportation systems, recreational facilities and activities, and drainage and erosion control infrastructure and practices
- Water Quality Review and Assessment

The principal conclusions of the Watershed Sanitary Survey are summarized in the following tabulation. Qualitative assessments of the potential risks to the drinking water supply are keyed to the principal contaminants, uses, activities and other potential sources of contamination at Squaw Valley.

Table 3-1. Principal Watershed Sanitary Survey Results

<table>
<tr><th>Key Uses, Activities and Other Potential Contaminant Sources</th><th>Potential Water Supply Quality Contaminants</th></tr>
<tr><td>Industrialized areas</td><td>Equipment fuels including diesel and gasoline, lubricating oils, grease, polycyclic aromatic hydrocarbons (PAHs), coolants including ethylene glycol, deicing compounds, suspended solids producing particulates, heavy metals (especially copper, zinc and lead) and asbestos.</td></tr>
<tr><td colspan="2">Qualitative Assessment of the Potential Risks to the Drinking Water Supply</td></tr>
<tr><td colspan="2">The Red Dog and Gold Coast Industrial Fabrication areas, and similar areas in and adjacent to Bldg. 1920 and the main parking lot, are considered to be high-risk sources because of the hazardous nature of materials used. The risks of contamination are increased by the observed lack of adequate protective measures, and the relative ease of contaminant movement over paved or compacted surfaces into storm drains, surface runoff, Squaw Creek, and eventually the aquifer.</td></tr>
<tr><td>Key Uses, Activities and Other Potential Contaminant Sources</td><td>Potential Water Supply Quality Contaminants</td></tr>
<tr><td>Maintenance associated with commercial recreational use of the ski area and associated motorized equipment and vehicles.</td><td>Gasoline and its additives (especially MTBE), diesel fuel, oil and grease, sediments, explosives, snowmaking additives.</td></tr>
<tr><td colspan="2">Qualitative Assessment of the Potential Risks to the Drinking Water Supply</td></tr>
<tr><td colspan="2">The Red Dog and Gold Coast Maintenance/refueling/snowmaking areas, and the area in and adjacent to Bldg. 1920, are considered to be high risk sources due to the hazardous nature of materials used, the observed lack of adequate protective measures, and the relative ease of contaminant movement over paved or compacted surfaces into storm drains, surface runoff, Squaw Creek, and eventually the aquifer. The risk of MTBE contamination at fuel storage and refueling sites is considered to be very high due to the high mobility of MTBE.</td></tr>
<tr><td>Key Uses, Activities and Other Potential Contaminant Sources</td><td>Potential Water Supply Quality Contaminants</td></tr>
<tr><td>Stables, corrals and horseback riding.</td><td>Livestock wastes, microbiological contaminants including Giardia and Cryptosporidium, nitrates, total dissolved solids, potassium, DBP precursors.</td></tr>
</table>

Qualitative Assessment of the Potential Risks to the Drinking Water Supply	
The risk of contamination of the public water supply by pathogens is considered to be moderate due to the filtering effects of soil particles in the ground following infiltration. The risk level may be higher in areas of the aquifer that may be under the direct influence of surface water.	
Key Uses, Activities and Other Potential Contaminant Sources	**Potential Water Supply Quality Contaminants**
Historical contaminant storage, spillage and leakage sites.	Hydrocarbon fuels, including diesel and gasoline, lubricating oils, grease, and polycyclic aromatic hydrocarbons (PAHs).
Qualitative Assessment of the Potential Risks to the Drinking Water Supply	
The risk of contamination from these sources is considered to be very high. However, extensive groundwater quality remediation activities are ongoing.	
Key Uses, Activities and Other Potential Contaminant Sources	**Potential Water Supply Quality Contaminants**
Recreational use of the backcountry, ski area, valley floor, and Squaw Creek (body contact and non-water contact recreation).	Microbiological contaminants (bacteria, viruses and protozoa), sediments.
Qualitative Assessment of the Potential Risks to the Drinking Water Supply	
The risk of contamination of the public water supply by pathogens is considered to be moderate due to the filtering effects of the soil particles in the ground following infiltration. The risk level may be higher in areas of the aquifer that may be under the direct influence of surface water.	
Key Uses, Activities and Other Potential Contaminant Sources	**Potential Water Supply Quality Contaminants**
Revegetation and maintenance of vegetation on ski slopes, golf course and common areas.	Fertilizers, phosphates, nitrates, potassium, pesticides, DBP precursors.
Qualitative Assessment of the Potential Risks to the Drinking Water Supply	
The degree of risk from these sources is considered low from the forest areas due to low rates of use and considerable travel time. However it is judged moderate to high from the golf course due to its position, directly over the main aquifer, the relative porosity of the underlying soils, and the proximity of Squaw Creek.	
Key Uses, Activities and Other Potential Contaminant Sources	**Potential Water Supply Quality Contaminants**
Wastewater collection and transport infrastructure.	Pathogens, trace organics/metals, DBP precursors, phosphates, nitrates.

Qualitative Assessment of the Potential Risks to the Drinking Water Supply	
The risk of contamination from wastewater infrastructure is considered to be low, except during extreme flood events and along the high-pressure mountain run, when/where the risk is judged high.	
Key Uses, Activities and Other Potential Contaminant Sources	**Potential Water Supply Quality Contaminants**
Commercial areas, overnight accommodations, residential households, associated landscaping, paved areas, garages, and other parking facilities.	Leakage from sewer lines and old septic systems containing microbiological contaminants, heating oil leaks, nitrates and household chemicals; lawn and garden care products including fertilizers, pesticides and herbicides; automobile fluids including gasoline, diesel fuel, oil and coolants; deicing compounds such as salt.
Qualitative Assessment of the Potential Risks to the Drinking Water Supply	
The large number of vehicles parking at Squaw Valley overnight and the even larger number using the area during the day represent a large potential source. The most significant problem is the main parking lot, which slopes toward Squaw Creek. Storm drains lead to direct infiltration areas and the Creek, and historically contaminated snow is piled immediately adjacent to the Creek. There are no protective measures in place. The risk of contaminants entering the water supply is considered high.	
Key Uses, Activities and Other Potential Contaminant Sources	**Potential Water Supply Quality Contaminants**
Camping and unauthorized overnight occupancy of vehicles.	Microbiological contaminants and refuse.
Qualitative Assessment of the Potential Risks to the Drinking Water Supply	
This source increases the already high potential risks of contamination from the parking lot.	

For this assessment, the relative risks of contamination from identified activities (or the susceptibility of the source of supply to the contaminants) relates primarily to the potential for the District to draw drinking water contaminated by the identified activities and sources of contamination at concentrations that would pose concern to public health and safety.

To determine the relative risks, the following factors were taken into account:

- Characteristics of the contaminants (i.e. pathogenicity, toxicity, environmental fate, and rate of transport)
- Characteristics at or downstream of the sources of the contaminants (i.e. location, likelihood of release, effectiveness of mitigation measures, best management practices, environmental absorption and buffering capability)

A semi-quantitative ranking of the relative risk levels posed by detrimental activities and sources based on engineering judgment and taking into account the type of contaminant, the magnitude of the source, and the location of the sources, was completed as part of the Source Water Assessments.

SOURCE WATER ASSESSMENTS

The District and the Mutual Water Company completed the first step in developing a Groundwater Protection Plan that included the following elements:

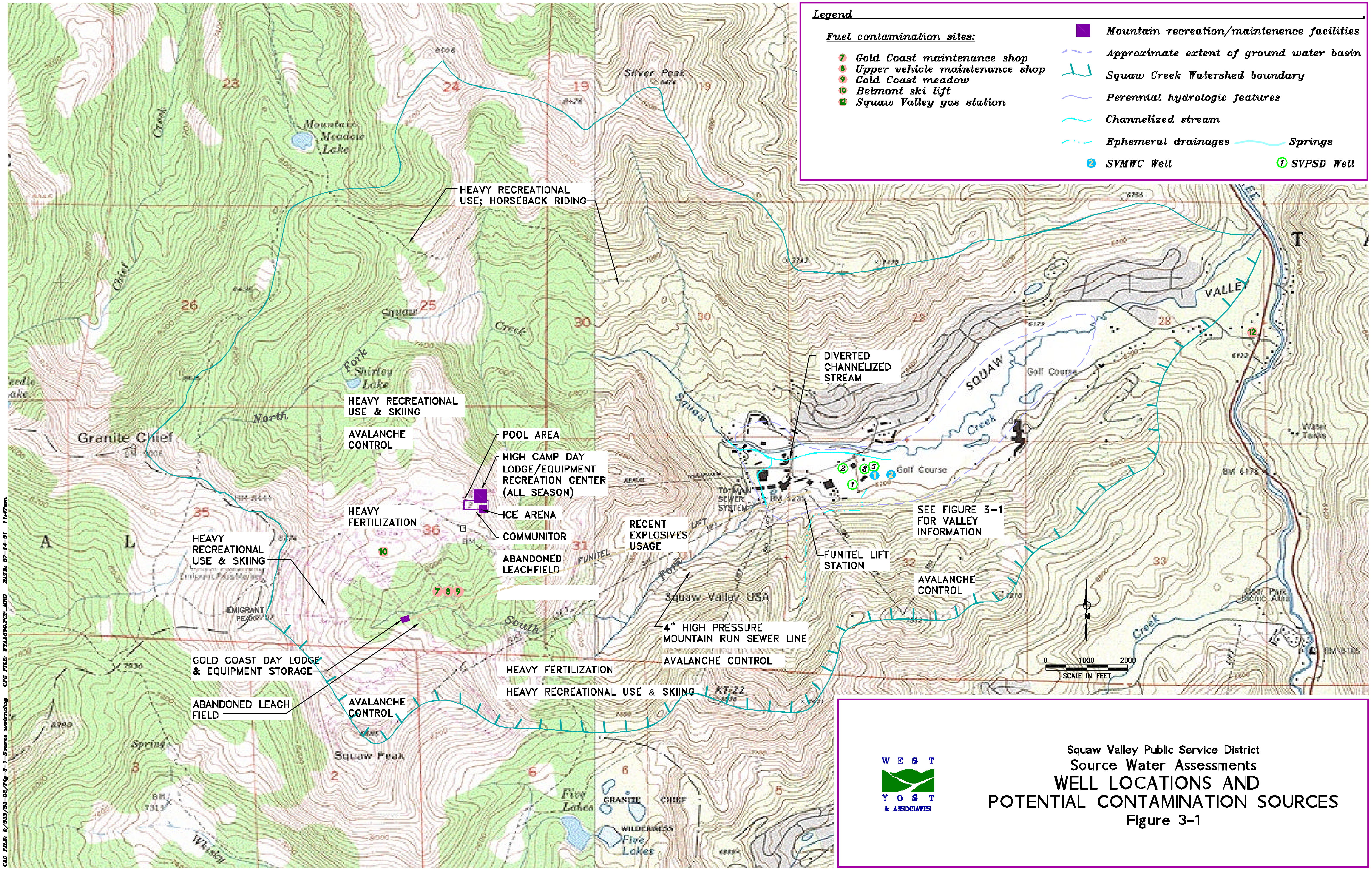
- Delineation of the boundaries of the protection areas for wells providing source water for District customers
- Inventory of the sources of regulated and certain unregulated, contaminants of concern in the delineated areas/capture zones (to the extent practical)
- Determination of the vulnerability of the wells to contamination
- Public education and outreach

The Source Water Assessments Report contained the delineation of capture zones for each of the production wells for the 1-, 2- and 5-year periods. The delineated areas or capture zones were determined using the groundwater model developed for the valley as part of the District's Groundwater Development and Utilization Feasibility Study. The delineated protection areas allow the District to focus protection, management strategies, and resources on areas providing the most benefit to the water resource.

The initial activity was completion of the data forms prepared by DOHS that included information on the drinking water source location, delineation of groundwater protection zones, physical barrier effectiveness checklist and well data sheets. These forms were completed for all District and Mutual production wells, including the horizontal wells located outside the groundwater basin.

An essential element of the assessment program is an inventory of Potential Contaminating Activities (PCAs). PCAs are facilities or land uses that can generate significant contamination in delineated source water protection areas or capture zones. The potential contamination locations, and production well locations are presented in Figures 3-1 and 3-2.

Delineated source water protection areas or capture zones are the portions of the groundwater basin that contribute water to pumping wells within a designated period of time typically 1, 2, or 5 years. The capture zones for the District and Mutual wells were determined using the computer model of the groundwater basin that is described in Section 4. Model analyses were performed to track the travel path of water pumped by each well. The area within the outer limit of travel paths represents the 1, 2 and 5-year travel time or capture zone of water flowing to that particular well. The 1, 2, and 5-year travel paths and capture zones are shown in Figures 3-3 through 3-5. Review of these figures suggests that there is little potential for the existing wells to be impacted by the poorer water quality in the basin to the east.

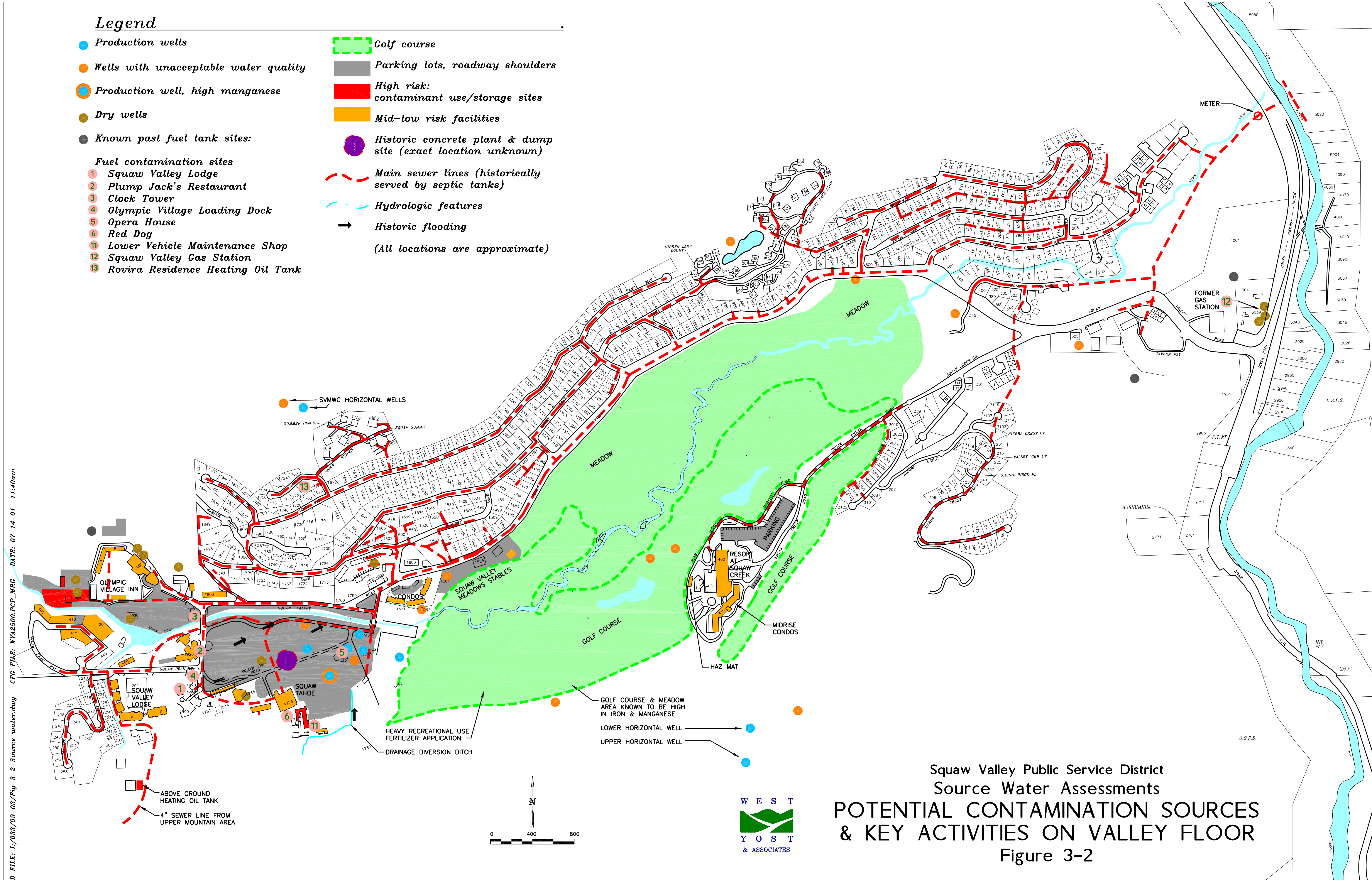


Squaw Valley Public Service District

Source Water Assessments

WELL LOCATIONS AND POTENTIAL CONTAMINATION SOURCES

Figure 3-1



Squaw Valley Public Service District
Source Water Assessments
POTENTIAL CONTAMINATION SOURCES & KEY ACTIVITIES ON VALLEY FLOOR
Figure 3-2

1-Year Capture Zone

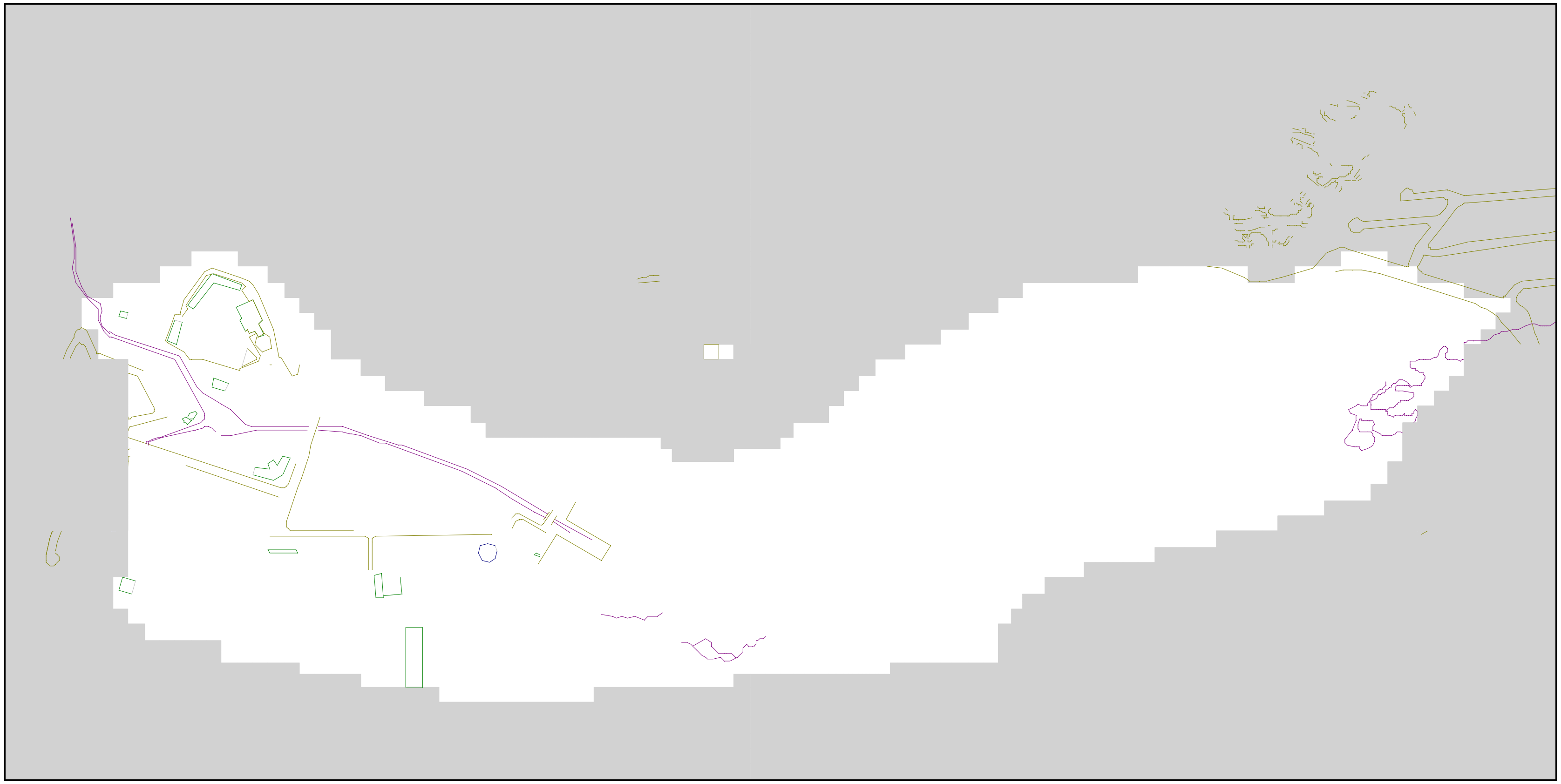


Figure 3-3. Squaw Valley Source Water Assessments

2-Year Capture Zone

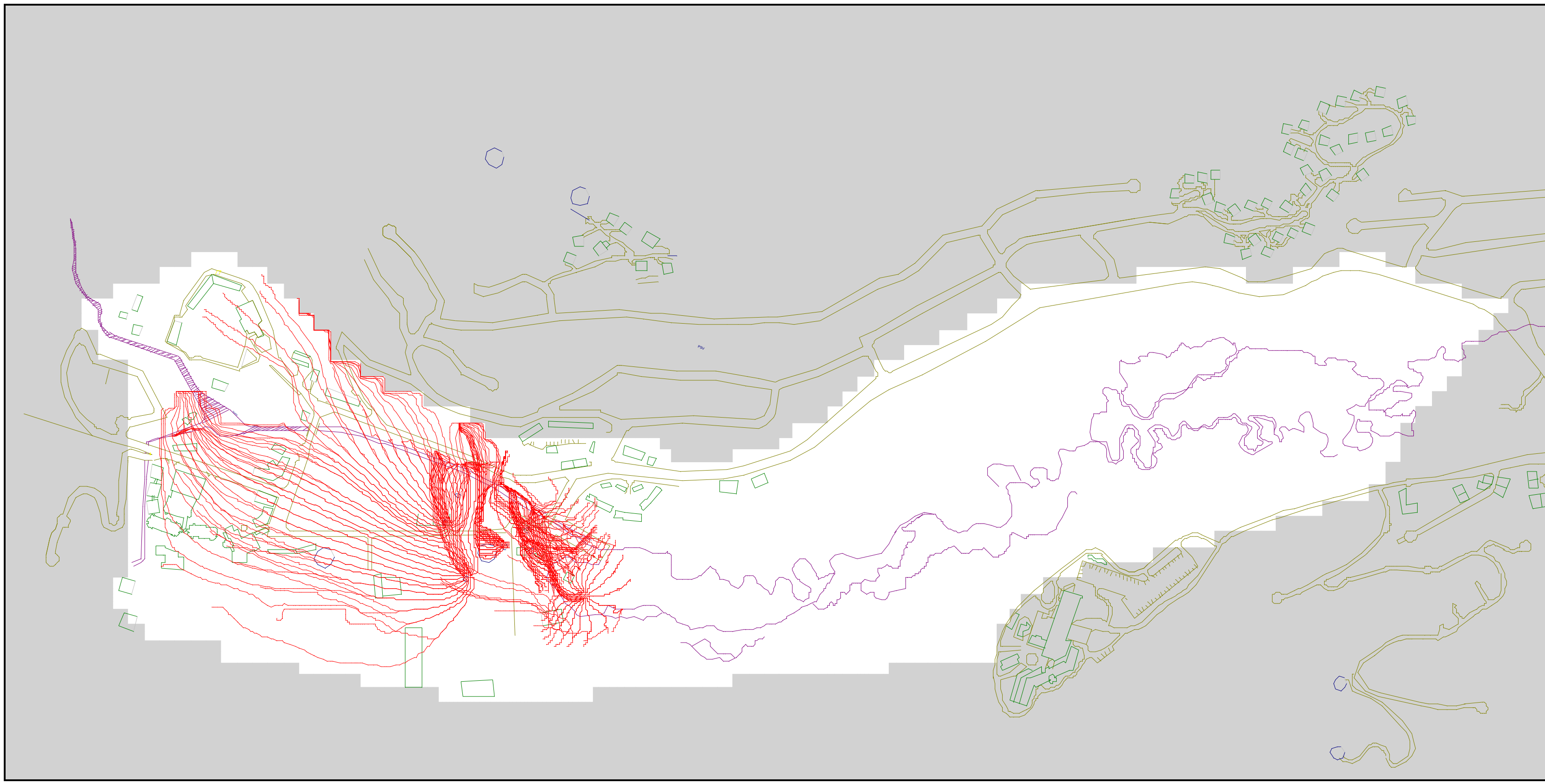


Figure 3-4. Squaw Valley Source Water Assessments

5-Year Capture Zone

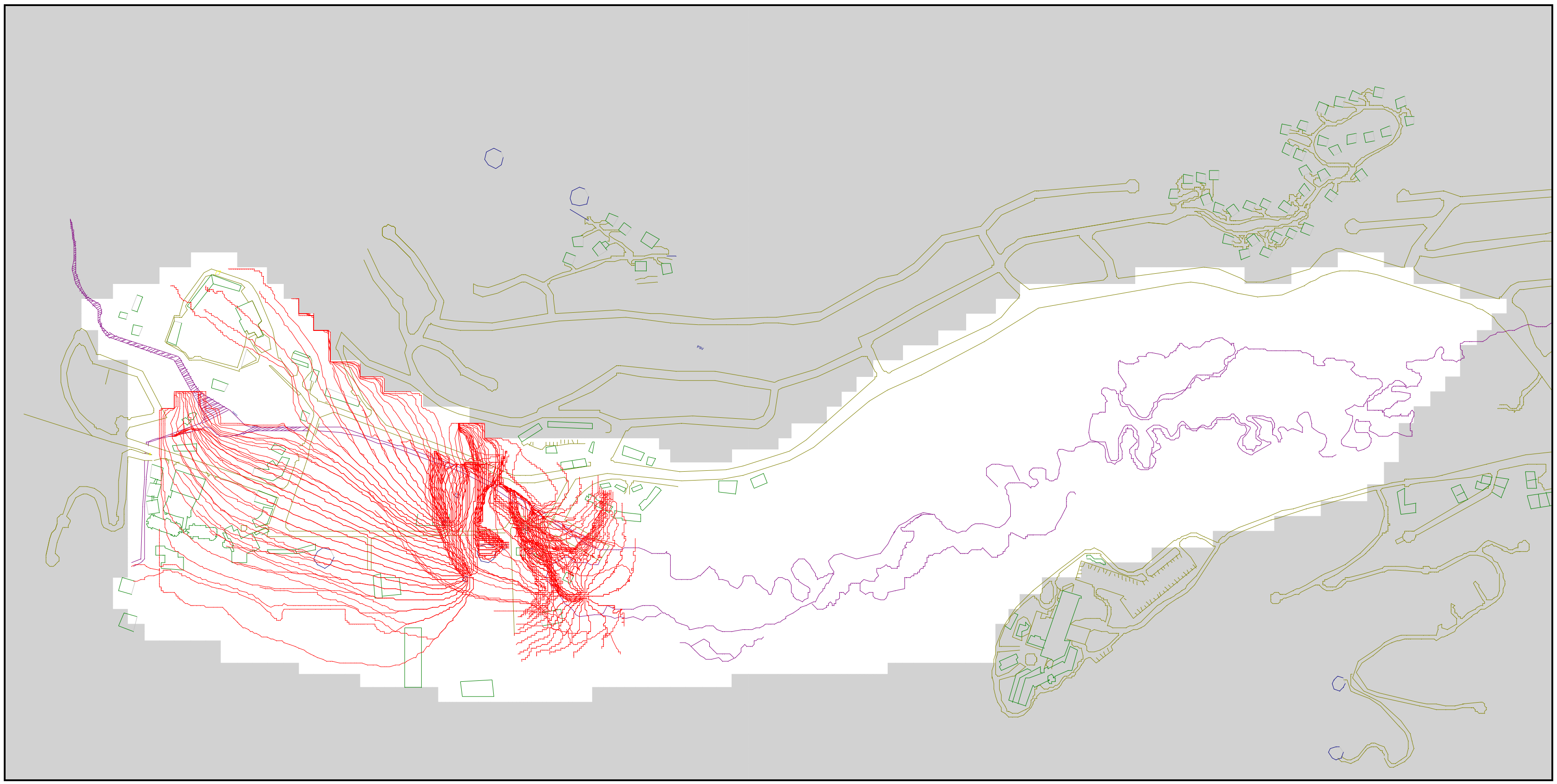


Figure 3-5. Squaw Valley Source Water Assessments

The Vulnerability Ranking is a prioritized list of the PCAs identified in the source water assessment and a relative ranking of the well exposure to potential sources of contamination. The order of the ranking is based on the class of PCA, its respective risk ranking (relative risk to drinking water supplies), the protection zone in which the PCA occurs, and the Physical Barrier Effectiveness rating (how effective the source and site are at preventing contaminants from reaching the drinking water). These factors are used to identify the PCAs to which the drinking water source is most vulnerable. Summary tables listing the vulnerability scores and risk ranking for the District's wells were prepared and are included in the Source Water Assessments Report.

The highest ranked activities for of the District's Vulnerability Ranking include: historic gas stations, confirmed leaking underground storage tanks, known contaminant plumes, chemical/petroleum storage, machine shops, utilities maintenance areas, pesticide/fertilizer storage areas and non-regulated storage tanks. Other lower ranked PCAs include: parking lots, a golf course, high density housing, above ground storage tanks, storm drain discharge points, storm water detention facilities, construction staging area, and other water supply wells.

The District invited stakeholders to form the Squaw Valley Groundwater Protection Advisory Group to help identify, develop and implement local measures that will advance the protection of the District's groundwater supply. A series of meetings were then held and a proposed Groundwater Protection Plan prepared.

GROUNDWATER PROTECTION PLAN

Source water protection is not a mandated element of EPA's Source Water Assessment Program. The 1996 SDWA Amendments do not impose regulatory or enforcement provisions requiring drinking water source protection by or upon the states and water purveyors. However, many of the amendments require EPA to consider further incorporation of source water protection measures into drinking water regulations, particularly as a basis to move toward increased regulatory flexibility. These provisions are intended to encourage states and local agencies to go beyond source water assessment toward the implementation of management techniques to protect sources of drinking water supplies from identified Potential Contaminating Activities (PCAs). Prevention of source water contamination provides great benefits to the public and is almost always less expensive than the treatment and monitoring that is required after a drinking water source has been contaminated.

The goals of a local source water protection program are to identify, develop and implement local measures that provide protection to the drinking water supply. Groundwater protection provides one more "barrier" to contamination in a multi-barrier protection treatment train. Moreover, an active and effective source water protection program can save the money that would otherwise be required to provide additional treatment processes and chemicals.

Drinking water purveyors are encouraged to develop management strategies to mitigate the risk of contamination of drinking water supplies and improve water quality. Management strategies are aimed at reducing the risk of contamination through activities such as pollution prevention, use of Best Management Practices (BMPs), and public education.

Management of wellhead protection areas to prevent groundwater contamination involves several steps:

- Identification of protection options appropriate for the types of PCAs present
- Selection of those that are technically and politically feasible
- Implementation
- Monitoring the effectiveness of management options and the application of additional Best Management Practices (BMPs), if required
- Development of contingency plans to address threats to the water supply that may result from accidents

A groundwater protection plan was developed through the stakeholder process. The plan provides direction and focus for groundwater protection efforts undertaken by the District and the community. The plan outlines management strategies that together will provide the key to a successful prevention program.

The Process

Following the completion of the Source Water Assessment the District assembled a group of stakeholders to discuss and recommend a wellhead protection program.

The first step in the process was review of the source water assessment plan. The plan followed the State of California guidelines entitled “Drinking Water Source Water Assessment and Protection (DWSAP) Program”.

The assessment concluded that the activities at the top of the District’s vulnerability ranking (i.e. those that posed the highest levels of risk to the quality of the groundwater) include:

- Historic gas stations,
- Confirmed leaking underground storage tanks,
- Known contaminant plumes,
- Chemical/petroleum storage,
- Machine shops,
- Utilities stations maintenance areas,
- Pesticide/fertilizer storage areas and non-regulated storage tanks.

Other PCAs include: Parking lots, a golf course, high density housing, above ground storage tanks, storm drain discharge points, storm water detention facilities, construction staging areas, other water supply wells, and monitoring wells.

The second step was to review the reasons for inviting stakeholders to participate in protection plan development. A successful source water protection program requires public involvement. The principal reasons for encouraging public involvement are to:

- Build support for the program
- Ensure that interested parties understand the program and have "ownership" in it
- Provide technical review of the program elements and build on local knowledge
- Help develop consensus among parties affected by the program
- Help promote awareness of source water quality issues and build support for source control activities in the community

The District invited the following entities to participate in the Groundwater Protection Stakeholders Group:

State of California Department of Health Services
California Department of Water Resources
Lahontan Regional Water Quality Control Board
Squaw Valley Mutual Water Company
Local Residents
Squaw Valley Ski Corporation
Sierra Club Local Chapter
The Resort at Squaw Creek
Placer County Department of Environmental Health
Squaw Valley Lodge
Olympic Village Inn

The Advisory Group met four times during 2000 to help develop the District's Groundwater Protection Plan. The District's purpose in initiating the stakeholder participation was to:

- Enable interested parties to understand the assessment program.
- Obtain and provide technical assistance.
- Help develop a community based Groundwater Protection Plan
- Develop consensus among affected parties.
- Address any concerns of the public.
- Encourage a good working relationship between local, state and federal agencies and other affected parties.

Key activities comprising the proposed Squaw Valley Groundwater Protection Plan are described below. These activities will be accomplished at the local level with support from local agencies and stakeholders. The stakeholders identified a series of activities that will raise awareness of groundwater vulnerability and help promote programs to protect the valley's groundwater supply.

The key elements of the program are:

- Public education:
  - Newsletters – the District issues two newsletters a year and so does the Mutual Water Company. The Homeowners Association also sends newsletters.

Information regarding source protection and tips to be used around the home will be included in these newsletters.

- Ski Area – issues daily reports that can include information and tips for visitors.
- Web page – the District budgeted for the development of a web page for 2000 – 2001. A good outreach program can be incorporated into the page with tips for local residents and visitors alike.
- Bill stuffers – information will be included when bills are issued.
- Consumer Confidence Reports (CCRs) – the annual report will include information regarding the source assessment and can include some information regarding source control efforts.
- Posters – will be produced and posted in resorts and lodges
- Signs – will be placed on the well buildings and around the parking lot indicating that they are in a groundwater protection zone.

- Community Involvement:
  - Truckee River Day - a clean the river day will also include Squaw Creek.
  - Sierra Watershed Education Partnership can supply information regarding the event.

- Pollution Prevention Programs:
  - Educating employees in the resorts to help reduce accidents/spills can be combined with the Placer County Health Department orientation program.
  - Hazardous Waste Collection – the County has a site open on Saturdays during the summer months. Days and location information will be posted and also sent out in bill stuffers and newsletters.
  - Education materials can also be distributed addressing the issues of hazardous compounds and their disposal and a list of suggested alternatives homeowners can use.
  - A guide can be made available for the application of fertilizers, herbicides and pesticides for homeowners. Existing guides can be explored for their applicability to the community.
  - Pollution prevention plans should be developed by commercial establishments to prevent spills and operator errors. Good house keeping guidelines can help prevent accidents and spills.
  - Establish pollution prevention recognition programs will be adopted to recognize those establishments that have proactive programs.
- Trail maps can include information for hikers regarding behavior on trails, near creeks, and at overnight camping sites to help protect water quality. Trail head signs can also provide information
- Water conservation programs –

- — metering the Mutual's water customers
- — signs at hotels and restaurants
- — landscaping tips to conserve water
- — differential billing by water use; "inclining rate structure" will be evaluated by the District during the next fiscal year.

- Continue groundwater monitoring and increase monitoring for elements of concern to track any suspected contamination threat.
- Develop an integrated water quality database for the Valley. Coordinate the effort with the Regional Board's development of a database for surface water quality.
- Use the CEQA process as the vehicle to review the impact of any proposed developments and to avoid or mitigate the impact of those that can threaten groundwater quality.
- Request the Fire Department to increase inspections/reports of any facilities storing and handling hazardous materials. Existing regulations require inspection once every three years; a frequency of once a year with follow-up on any violations is more appropriate for a protection zone.
- The District will consider including the imposition of liability for pollution in their code.
- The District will work closely with the RWQCB and Placer County to obtain information on spills and/or accidents that could impact water quality in the basin. This is an ongoing practice that should be continued to ensure quick responses to spills to prevent groundwater contamination.
- Agreements in the Groundwater Management Action Plan regarding groundwater quantity should also address the need to protect groundwater quality. It is also recommended that the stakeholders in the valley work cooperatively to develop an integrated water management plan to ensure an adequate water supply for the Valley's future.
- The District should consider moving its wells out of pits, and when possible, away from the parking lot. Wells placed in pits can be easily contaminated from surface water flows.
- It is recommended that an inventory of heating fuel tanks be conducted that includes above ground and underground tanks. A tank replacement program will help prevent accidents involving improperly installed, operated and maintained tanks. The District can explore funding sources for tank replacement such as air quality funding or rebates from the local gas and electric company.
- There are numerous monitoring wells in and around the well field that can introduce surface contaminants into the aquifer. These wells need to be identified. Those that are no longer used should be properly abandoned; the others should be constructed to prevent groundwater contamination and should be inspected on a regular basis.

- There are numerous sub-surface drainage infiltration systems in and around the well field. A study to assess the impact of these infiltration systems on groundwater quality should be undertaken especially in view of future developments such as parking garages and future phases of Intrawest Development.
- The wells operated by the District, Mutual, and others should be properly operated and maintained to prevent contamination of the water supply. New wells should be constructed to meet well construction standards including a 50 feet sanitary seal.
- The District should maintain minimum standards for operator certification and distribution system operator certification. Operators should be encouraged to continue their education program and be recognized for successfully completing certification exams.
- It is recommended that the District acquire additional land around their supply wells to act as buffer zones against contaminating activities.
- Some of the District's wells are in close proximity to the creek and are subject to some surface water influence. Additional testing to determine the impact of adverse conditions such as flooding and drought on water quality is recommended.
- RV pump out facilities should be provided and RVs should only be permitted in a specially designated parking area. Overnight camping should be discouraged unless the site provides RV hook-ups and is controlled by qualified personnel.
- The District should join the Groundwater Guardian Program administered by the Groundwater Foundation and submit entries for AWWA Wellhead Protection Program Awards.
- Table 3-2 includes a summary of measures to protect groundwater from identified activities on the watershed.

Table 3-2. Summary of Groundwater Protection Measures

Activity (PCA)	Mitigation/Control Measures
Historic gas stations	Obtain verification from the RWQCB that all clean-up activities were completed.
Confirmed leaking underground storage tanks	Keep in contact with regulatory agencies to ensure timely clean-up. Review monitoring data and collect samples from close-by supply wells to make sure no contamination reaches wells.
Known contaminant plumes	Work with regulatory agencies to ensure that clean-up activities are followed and review all monitoring data to verify that the plumes are not contaminating drinking water supply wells. Conduct additional sampling, where necessary.

Activity (PCA)	Mitigation/Control Measures
Chemical/petroleum storage	Request the Fire Department to increase inspections/reports of any facilities storing and handling hazardous materials. Existing regulations require inspection once every three years, a frequency of once a year with follow-up on any violations is more appropriate for a protection zone. Empty or unused chemical containers should be removed to hazardous waste disposal site. All chemicals should be properly stored to prevent spills.
Machine shops	Recommend that pollution prevention plans be developed by commercial establishments to prevent spills and operator errors. Good housekeeping guidelines can help prevent accidents and spills. Request the Fire Department to increase inspections/reports of any facilities storing and handling hazardous materials.
Utilities stations maintenance areas	Recommend that pollution prevention plans be developed by commercial establishments to prevent spills and operator errors. Good housekeeping guidelines can help prevent accidents and spills. Empty or unused chemical containers should be removed to hazardous waste disposal site. All chemicals should be properly stored to prevent spills.
Pesticide/fertilizer storage area	Recommend that pollution prevention plans be developed by commercial establishments to prevent spills and operator errors. Good housekeeping guidelines can help prevent accidents and spills. Empty or unused chemical containers should be removed to hazardous waste disposal site. All chemicals should be properly stored to prevent spills.
Non-regulated storage tanks	Should meet the same requirements as regulated tanks (double containment, monitoring for leaks, etc.)
Parking lots	Post signs to make public aware of a groundwater protection area. Develop public information/education materials for visitors. RV pump out facilities should be provided and RVs should only be permitted in a specially designated parking area. Overnight camping should be discouraged unless the site provides RV hook-ups and is controlled by qualified personnel.

Activity (PCA)	Mitigation/Control Measures
Golf course	Follow existing programs
High density housing	Develop public education, pollution prevention programs for hotels, lodges and other commercial establishments. Request the Fire Department to increase inspections/reports of any facilities storing and handling hazardous materials.
Above ground storage tanks	It is recommended that an inventory of heating fuel tanks be conducted that includes above ground and underground tanks. A tank replacement program will help prevent accidents involving improperly installed, operated and maintained tanks. The District can explore funding sources for tank replacement such as air quality funding or rebates from the local gas and electric company.
Storm drain discharge points, storm water detention facilities	There are numerous sub-surface drainage infiltration systems in and around the well field. A study to assess the impact of these infiltration systems on groundwater quality should be undertaken especially in view of future developments such as parking garages and future phases of Intrawest Development.
Construction staging area	Make sure construction sites are using erosion control BMPs. Ensure that all materials are properly stored and handled to prevent contamination.
Water supply wells	The wells operated by the District and others should be properly operated and maintained to prevent contamination of the water supply. New wells should be constructed to meet well construction standards including a 50-feet. sanitary seal. The District should maintain minimum standards for operator certification and distribution system operator certification. Operators should be encouraged to continue their education program and be recognized for successfully completing certification exams. It is recommended that the District acquire additional land around their supply wells to act as buffer zones against contaminating activities. It is recommended that the District move wells out of pits, and when possible, away from the parking lot. Wells placed in pits can be easily contaminated by surface water flows.

Activity (PCA)	Mitigation/Control Measures
Monitoring wells.	There are numerous monitoring wells in and around the well field. Each can act as a vehicle for introducing surface contaminants into the aquifer. These wells need to be identified. Those that are no longer used should be properly abandoned; the others should be constructed to prevent groundwater contamination and should be inspected on a regular basis.

Funding Sources for Source Protection Activities

There are several sources of funding to assist the District in implementing the proposed groundwater protection activities including:

1. Local Groundwater Management Assistance Fund (AB 303)
2. SDWA SRF loans for source protection.
3. Nonpoint Source Implementation Grants (319 Program)

The local Groundwater Management Assistance Act of 2000 (AB 303) is designed to help local public agencies better manage groundwater resources to ensure the safe production, quality and proper storage of groundwater in California. AB 303 authorizes grants up to $250,000 to local agencies to conduct groundwater studies or to implement groundwater monitoring and management activities. The applications for funding in the 2001-2002 fiscal year are due on October 23, 2001. The District should submit an application for this funding assistance.

The State Revolving Fund Loan includes funds for source water protection. Funds are to be used for source protection projects and land/easement acquisition. DOHS is the contact agency for these loans.

Nonpoint Source Implementation Grants (319 Program) provide grants to implement nonpoint source projects and programs in accordance with Section 319 of the Clean Water Act. The grant can be used to implement BMPs for education programs and others. The RWQCB reviews the applications and administers the program in California.

Implementation of the Groundwater Protection Plan

The next step in the implementation of the Groundwater Protection Plan is to circulate the plan with a letter that describes the plan and includes a signature form for interested parties to endorse and support implementation of the protection plan. The protection measures can then be implemented by the supporting members. In addition, the District should submit copies of the Watershed Sanitary Survey, Source Water Assessments and Groundwater Protection Plan to DOHS for their information and review to assist in the protection of drinking water in Squaw Valley.

SECTION 4. GROUNDWATER MODEL DEVELOPMENT

A groundwater flow model of the Squaw Valley Basin has been developed as part of this Groundwater Development and Utilization Feasibility Study. Development of the model was discussed in the report titled, “Groundwater Model Report” prepared by Derrik Williams, R. G. The primary modeling objective was to develop a tool for the District’s future water planning efforts that could evaluate future groundwater management alternatives. This section presents a summary of the conceptual model of the basin, discusses how the flow model was constructed, presents calibration results from the flow model, discusses results of the model sensitivity analyses, and presents conclusions on the use of the model.

The groundwater model should be updated and revised, as additional data become available. Although the current model adequately simulates the general flow characteristics of the Squaw Valley Basin, and is a useful tool for water planning efforts, ongoing data acquisition and analyses will likely reveal areas where the model could be improved.

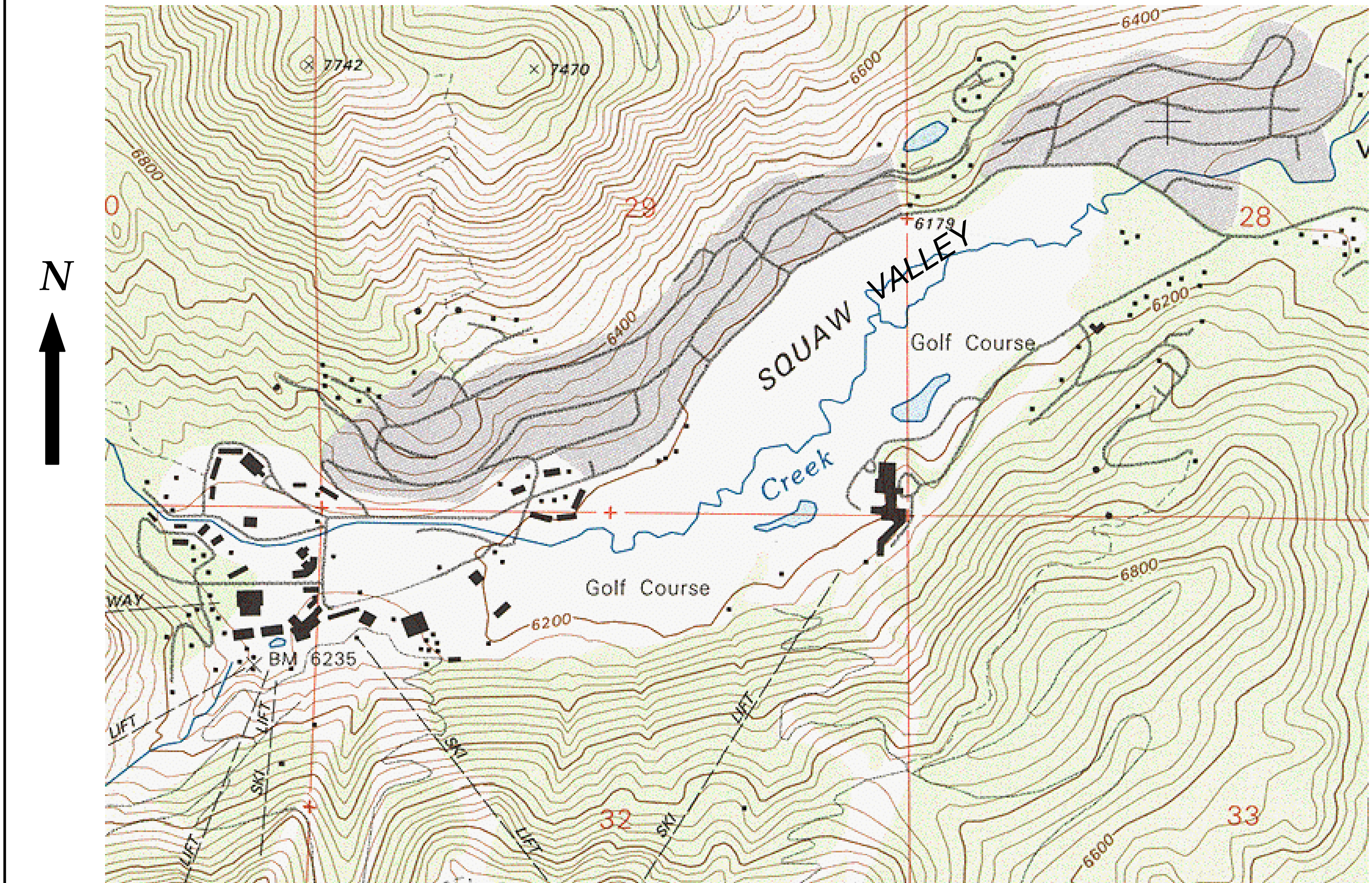
2003 GROUNDWATER MODEL UPDATE

The Squaw Valley Groundwater Model has been modified and updated based on data not available during the initial model development and calibration. These data include numerical data such as from the detailed mapping of the valley and water level information from the Intrawest parking structure construction dewatering drawdown, and supplementary non-numerical data such as an assumed depth of the Plumpjack well. This update was not intended as a complete re-evaluation of the groundwater model, rather it was an attempt to simply incorporate new data. A description of the model update and calibration results are presented in a technical memorandum that is included in the Appendix at the back of this report.

The Appendix details the data incorporated into the model, the model calibration and verification, presents a new evaluation of the sustainable yield, and outlines further steps that should be undertaken to improve the groundwater model. The original Section on the Groundwater Model Development has not been modified and is included in this update report because most of the information used in the model development is still valid. The results of the calibration effort of the updated model is summarized at the end of this section.

CONCEPTUAL MODEL

Squaw Valley is a glacially carved valley in the Sierra Nevada of California. The valley is situated north of Lake Tahoe, at an elevation of approximately 6,200 feet (Figure 4-1). The valley floor measures approximately 1.5 miles by 0.4 miles, covering an area of approximately 370 acres. Steep mountains bound the valley on the north, west, and south. A terminal moraine on the valley’s eastern end separates the valley from the Truckee River. The valley is drained by Squaw Creek. The north and south forks of Squaw Creek enter along the valley’s western side. Squaw Creek exits the valley through the terminal moraine on the valley’s eastern end.



SVPSD Groundwater Model Development

Figure 4-1. Study Area

derrik williams

Land use in the valley is mainly commercial, recreational, and open space. Commercial and recreational development is prominent in the western portion of the valley. This portion of the valley is largely paved and covered with commercial buildings. The Resort at Squaw Creek, and the associated golf course, dominates the southeastern portion of the valley. An undeveloped meadow overlies the northeastern portion of the valley. Residential development partially rings the valley on the North and East sides.

Geologic Framework

The general basin stratigraphy consists of basement crystalline rocks, overlain by a collection of glacial, alluvial, and lacustrine sediments. Plutonic and volcanic rocks underlie the basin. These same rocks form the mountains that flank the valley on the north, west, and south. The basement rocks are often divided into two classes: Cretaceous granitic rocks and Pliocene volcanic rocks. Kleinfelder (1987), further divided the bedrock into four units:

Jurassic highly metamorphosed sediments
Cretaceous granitic rocks, predominantly quartz diorite and quartz monzonite
Pliocene basalts
Pliocene to recent pyroclastic flows and fragmented, bedded volcanics

The basin sediments comprise unconsolidated glacial, alluvial, and lacustrine deposits (Kleinfelder, 1991). The most prominent glacial feature is the terminal moraine at the eastern end of the valley. This moraine formed a dam in the valley outlet. Various alluvial and lacustrine sediments collected behind this dam, filling in the valley to its present elevation. The alluvial and lacustrine deposits do not appear to display any substantial lateral continuity. The only available generalization is that sediments appear to become finer toward the northeastern portion of the basin. The basin bedrock forms a trough that trends generally east to northeast. The deepest portion of the trough appears to run near the axis of the basin.

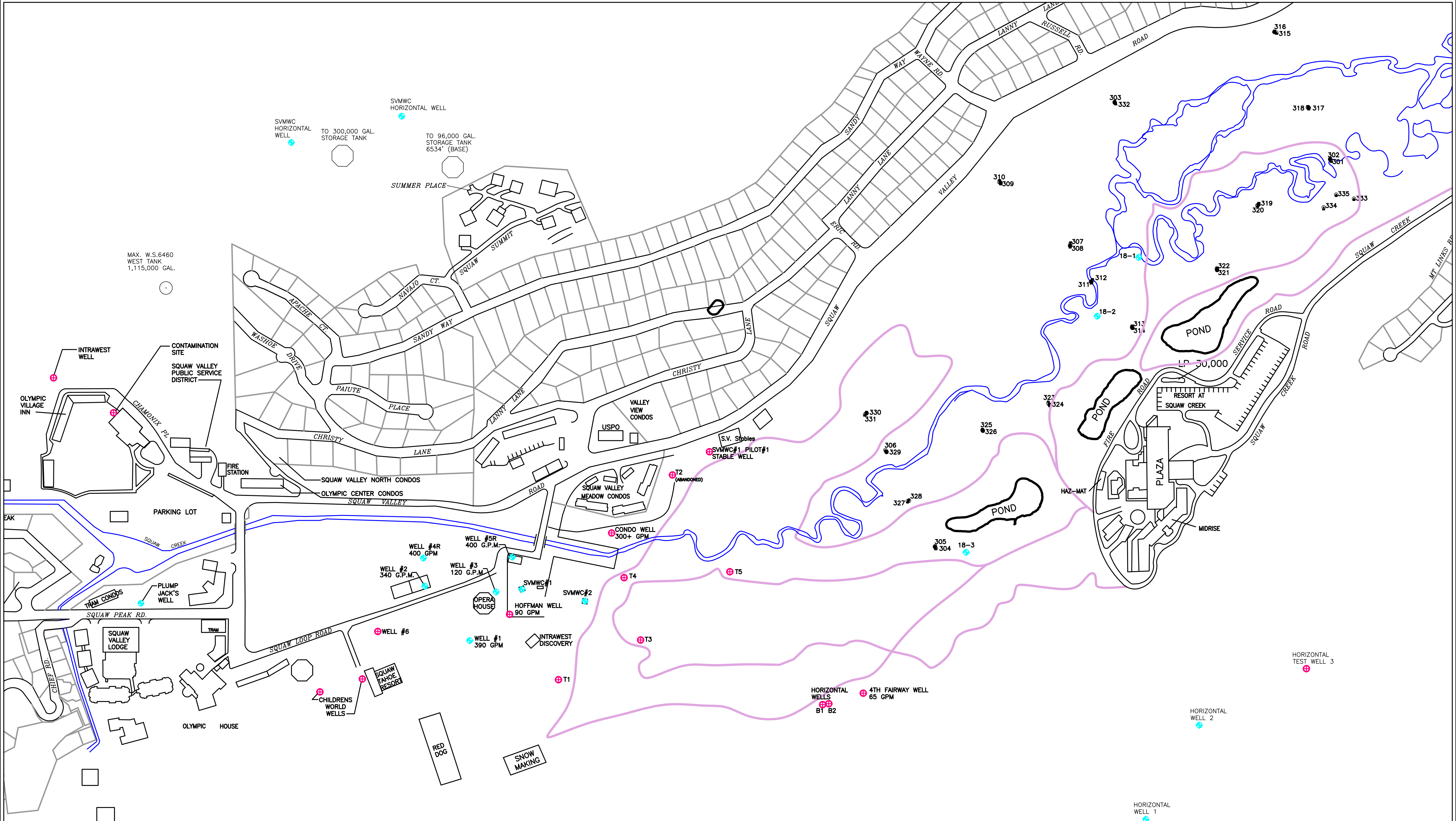
GROUNDWATER OCCURRENCE

Groundwater is found in fractures in the crystalline rocks surrounding and underlying the basin sediments. These fractures appear to dip steeply; Kleinfelder (1991) mapped three sets of fractures, all dipping between 85º and 90º.

The fractures feed at least four springs identified by Kleinfelder (1991). Flow rates for the four springs were not available. The fractures also provide water to horizontal wells on both the north and south sides of the valley. The Mutual Water Company owns two horizontal wells north of the valley. The west well produces 20 to 50 gpm, but the east well was taken out of service several years ago because of iron bacteria problems. The District owns and operates two horizontal wells south of the valley. The horizontal wells are reported to produce between 20 and 50 gpm. Two additional horizontal wells were installed south of the valley, near the fourth fairway of the Resort at Squaw Creek Golf Course. They are not in service at this time.

Groundwater in Basin Sediments

In Squaw Valley, groundwater is encountered in the sediments throughout most of the basin. Data defining groundwater conditions in the basin sediments are derived from production wells, test holes, and monitoring wells in the eastern meadow area. Figure 4-2 shows the well locations in Squaw



LEGEND

- PRODUCTION WELL
- TEST WELL
- SHALLOW MONITORING WELL
- DEEP MONITORING WELL
- APPROXIMATE GOLF CART PATH

NOTE: WELL LOCATIONS ARE APPROXIMATE
MAP SOURCE: SQUAW VALLEY PUBLIC SERVICE DISTRICT, 1999 WITH WELL LOCATIONS PROVIDED BY KLEINFELDER

N

0 250 500
SCALE IN FEET

Figure 4-2

Squaw Valley Public Service District
TEST HOLE & WELL LOCATIONS

WEST YOST & ASSOCIATES

CAD FILE: I:/033/99-03/P1WellLocation.dwg CFG FILE: WYA2500.PCP_MRG DATE: 07-10-01 2:04pm

Valley. This figure was based on information from the Groundwater Background Data Report (Kleinfelder, 2000). Construction data for a number of the wells were summarized by Kleinfelder (2000), and are reproduced on Table 4-1.

Example hydrographs showing water levels in District Well 2 and Mutual Well 1 are presented in Figures 4-3 and 4-4. These hydrographs show that water levels are fairly consistent over time; there is no apparent long-term rise or fall in water levels. The water levels appear to respond to annual pumping stresses in the summer and autumn, then rebound to consistent pre-pumping levels in winter and spring.

Both hydrographs show a marked response to recharge availability. The low water levels recorded in summer and autumn of 1994 are a response to a rainfall of only 65 percent of normal. The relatively higher water levels recorded during the summer and autumn of 1995 are a response to the preceding winter's rainfall, which was approximately 129 percent of average. These hydrographs suggest that the basin sediments are recharged to some maximum level every winter and spring. The basin sediments are then dewatered every summer and autumn, with the level of dewatering dependent on the availability of long-term recharge from the previous winter.

Groundwater Flow Directions

Groundwater generally flows northeast and parallel to Squaw Creek, from the western portion of the basin toward the terminal moraine. Overlain on this general flow direction is a propensity for groundwater to flow from the basin margins toward Squaw Creek. This demonstrates that Squaw Creek is a discharge point for groundwater in the meadow.

Head differences between shallow monitoring wells and deeper monitoring wells have been observed in the Squaw Valley basin. Both downward and upward vertical gradients have been observed in the basin. Downward vertical gradients have been observed near the western edge of the meadow (e.g. well pair MW-327 and MW-328). Upward vertical gradients have been observed near the eastern edge of the meadow (e.g. well pair MW-317 and MW-318). This is consistent with the generally observed recharge and discharge relationships in Squaw Valley. Downward vertical gradients are expected in recharge areas such as the western basin, and upward vertical gradients are expected in natural discharge areas, such as the meadow.

There is no laterally continuous aquitard in the basin that is associated with the vertical gradients. The vertical gradients appear to result from a combination of groundwater recharge and discharge conditions, combined with localized low permeability sediments. The collection of low permeability clay and silt stringers encountered in the basin effectively imparts a mild, confinement. In this situation, there is no identifiable confined aquifer. Rather, the lower sediments in the basin show a degree of confinement due to overlying low permeability sediments.

AQUIFER PARAMETERS

The hydraulic conductivity and storage coefficient of an aquifer represent its capacity to transmit and store groundwater. Estimates of these properties are needed to model three-dimensional, transient groundwater flow. These properties are generally estimated from the results of long-term aquifer tests performed on production wells. Accurate measurements of these properties are rare

Table 4-1. Well Construction Information

WELL	Year Drilled	Well-Head Elevation, ft msl	Screen Depth, ft bgs	Cased Depth, ft bgs	Casing Diameter, inches	Total Depth Drilled, feet	Depth to Bedrock, feet
Resort at Squaw Creek 18-1	1990	6180	45-60, 81-69	97	12	102	99
Resort at Squaw Creek 18-2	1990	6180	45-75	N/A	12	79.5	71
Resort at Squaw Creek 18-3	N/A	6180	N/A	N/A	N/A	N/A	N/A
SVWMC#1	1958	6195.00 [1]	60-?	100 [3]	14	100	N/A
SVWMC#2	1966	6190.50 [1]	35-?	105 [4]	14 [5]	105	N/A
Stable Well	1991	6190	32-72	72	4	72	68
Hoffman Well	1990	6190	43-83, 113-133	133	4	157	151
Condo Well	1992	6190	55-120	120	14	120	120
SVPSD #1	1958	6201.77 [1]	77[2]-112	N/A	14	112	N/A
SVPSD #2	1958	6202.22 [1]	33-74	N/A	14	74	N/A
SVPSD #3	1958	6200.32 [1]	77[2]-114	N/A	14	112	N/A
SVPSD #4R	1989	6204.65 [1]	40-70	71	12	74	73
SVPSD #5	1961	6200	71-137	N/A	12	137	N/A
SVPSD #5R	1999	6199.28 [1]	73-128	133	16	139	N/A
SVPSD #6 (MW)	N/A	6240	23-63	63	4	63	59
All data from Kleinfelder (20	N/A	6203.84 [1]	30-60	61	12	77	76.5
4th Fairway Well	N/A	6220	?-86.5	N/A	6	N/A	N/A
SVPSD Test Well #1	1999	6200	50-110	110	4	112	101

All data from Kleinfelder (2000) unless otherwise noted.

N/A - Information not available

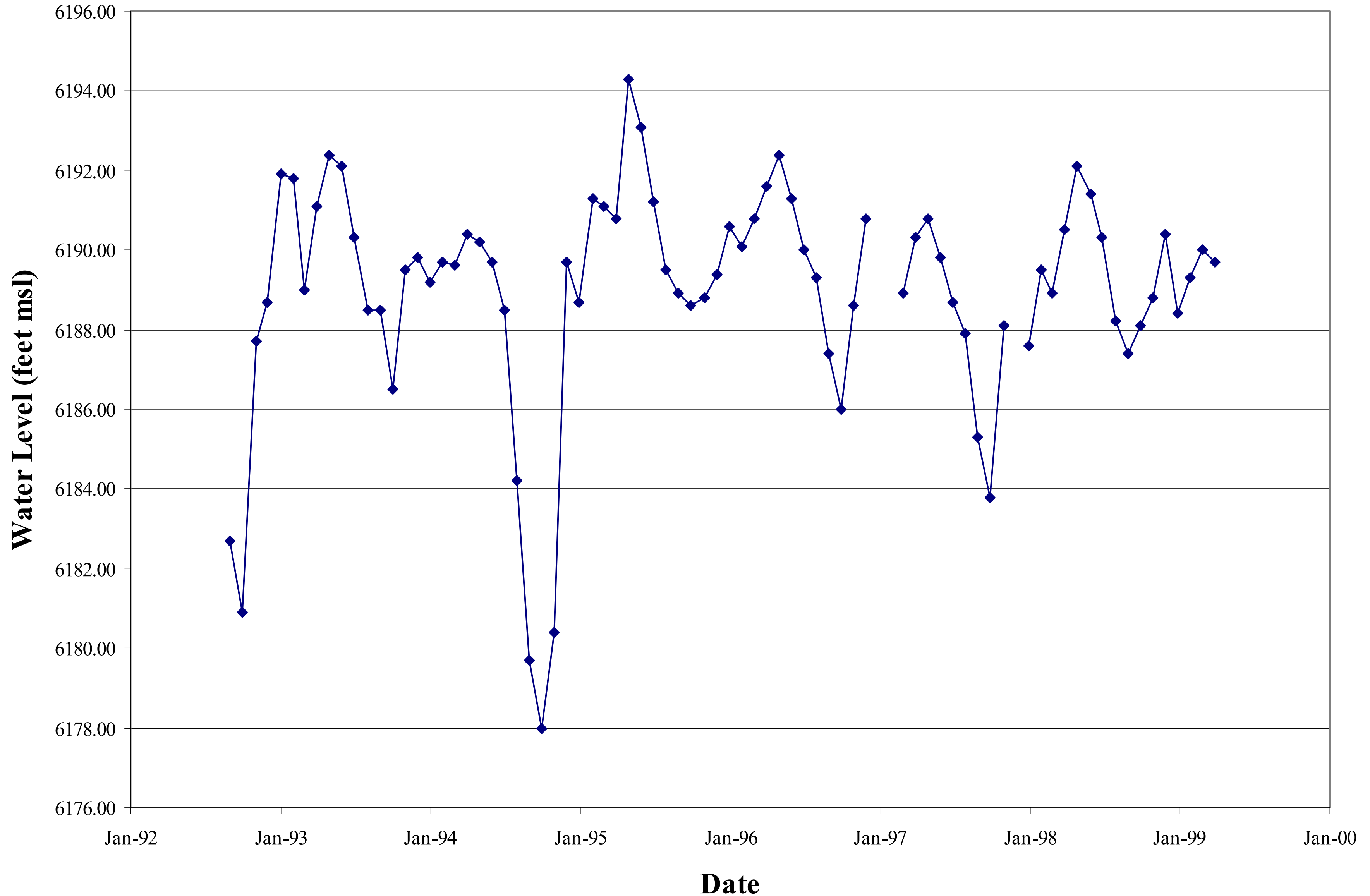
(1) From Squaw Valley Production Wells Survey by West Yost & Associates, 1/21/00

(2) From January 22, 2001 letter from Jesse McGraw/SVPSD to Steve Dalrymple/ West Yost & Associates

(3) 72 foot now-bottom 30 feet filled with concrete

(4) 62 feet now-bottom 40 feet filled

(5) Now has 10-inch insert



SVPSD Groundwater Model Development

Figure 4-3. SVPSD Well 2 Water Level

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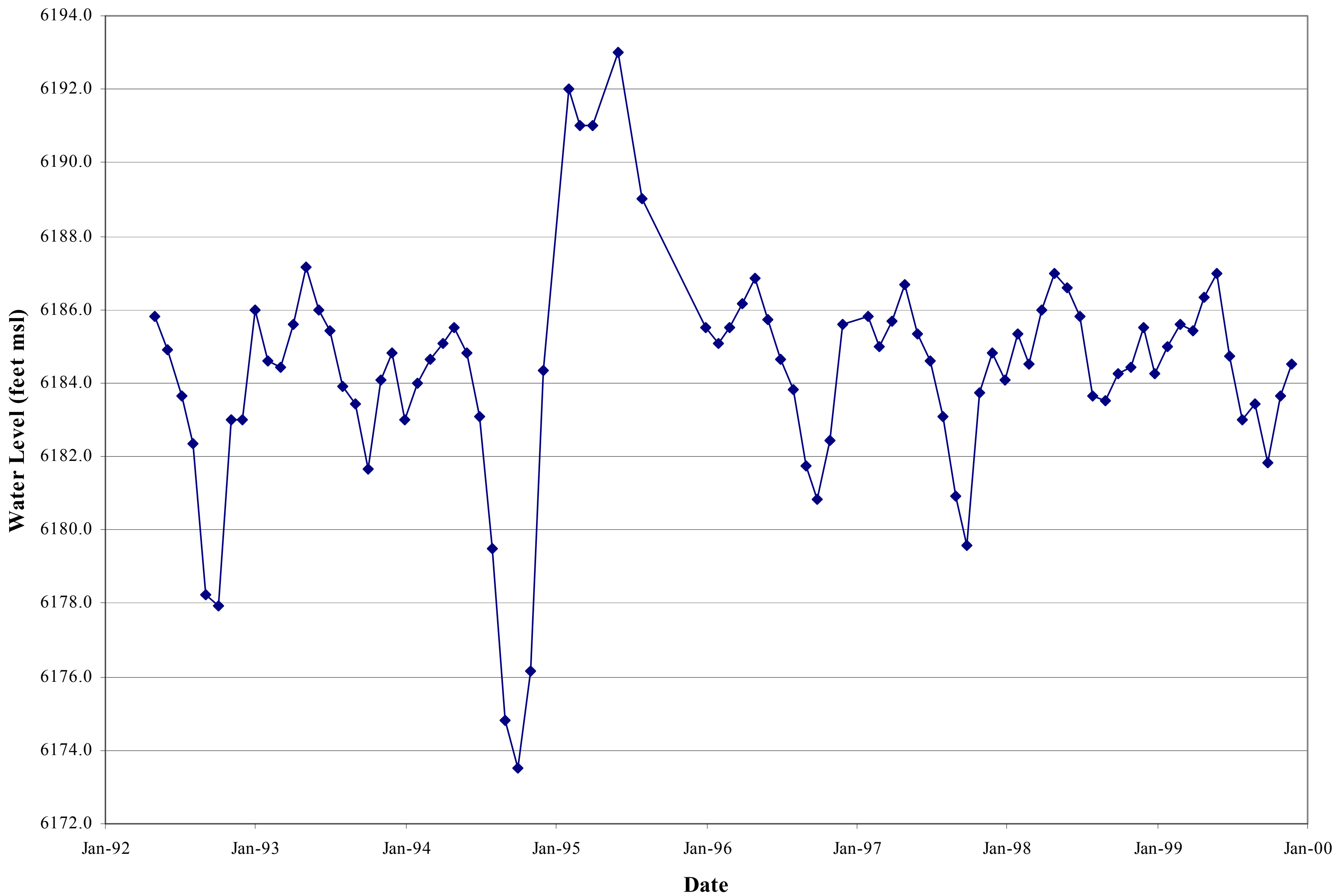

SVPSD
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Figure 4-4. Mutual Well 1 Water Level

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for wells in Squaw Valley, however a few aquifer tests have been performed in the valley that can provide guides for estimating aquifer properties.

Hydraulic Conductivity

Hydraulic conductivity measures an aquifer's capacity to transmit groundwater. Groundwater flows more readily through an aquifer with a relatively higher conductivity. Hydraulic conductivity is measured in units of length per time. An optional measure of an aquifer's capacity to transmit groundwater is transmissivity. Transmissivity is equal to hydraulic conductivity multiplied by aquifer thickness, and is measured in units of square length per time.

Table 4-2 lists previous estimates of transmissivity and hydraulic conductivity, compiled by Kleinfelder (1999). The transmissivity estimates range between 234 square feet per day to 43,700 square feet per day. These transmissivities correspond to hydraulic conductivities of between 6 feet per day and 1,457 feet per day. The average hydraulic conductivity is 186 feet per day.

Kleinfelder (2000) re-analyzed the aquifer test of District Well 4R. Table 4-3 presents the results of the re-analyses. The transmissivity estimates range between 1,500 and 304,000 square feet per day. These transmissivities correspond to hydraulic conductivities of between 25 and 5,070 feet per day. The average hydraulic conductivity is 1,388 feet per day.

The average hydraulic conductivity of 186 feet per day is relatively high, and the average hydraulic conductivity of 1,388 feet per day for the District Well 4R test is extremely high. These hydraulic conductivity estimates may be too high because the aquifer test was influenced by flow from Squaw Creek. Kleinfelder (2000) notes that pumping District Well 4R resulted in a 0.15-foot decrease in the water level of Squaw Creek during the aquifer test. This suggests that Squaw Creek acts as a partial recharge boundary for District Well 4R. Because this recharge boundary is not explicitly accounted for during analyses of the aquifer test, the resulting hydraulic conductivity estimate is higher than expected.

Although the estimate of 186 feet per day is high, it is well within the range of possible hydraulic conductivities for silty sand deposits. The conductivity of 1,388 feet per day would indicate a very clean sand or gravel deposit, which is unlikely to be laterally continuous in this basin.

Storage Coefficient

Storage coefficient measures an aquifer's ability to store groundwater. Storage coefficients are expressed as a specific yield in unconfined aquifers, and as a storativity in confined aquifers. Both coefficients are unitless.

Specific yields generally range between 0.01 and 0.3, while storativities range between 0.005 and 0.00005 (Freeze and Cherry, 1979). Values between these ranges may represent semi-confined conditions.

Tables 4-2 and 4-3 include estimated values of storage coefficients derived from aquifer tests in the Squaw Valley Basin. The storage coefficients range between 0.0024 and 0.2. These values are indicative of semi-confined to unconfined conditions.

Table 4-2. Previous Estimates of Transmissivity and Hydaulic Conductivity

	Test Type	Well Screen Length	Pumping Rate, gpm	Duration, hours	Estimated Transmissivity, square feet per day	Estimated Hydraulic Conductivity, feet per day	Estimated Storage Coefficient
Resort at Squaw Creek 18-2	Step	30	52-130	--	1187	40	0.059
Resort at Squaw Creek 18-3	Constant Rate	--	266	23	495 to 1250	--	--
Stable Well	Step	40	34-110	10	2674	67	--
Hoffman Well	Step	70	500	--	1497	21	0.065
	Step	70	15	8	1187	17	--
Condo Well	Step	65	320	--	1718	26	--
SVPSD #2	Constant Rate	41	406	24	103610	2527	--
SVPSD #4	Constant Rate	30	93	48	43716	1457	0.0173
SVPSD #6	Develop	30	250	--	2219	74	0.11
4th Fairway Well	Constant Rate	36.5?	67	8	234	6	--
SVPSD Test Well #1	Slug	60	--	--	6150	103	--
	Slug	60	--	--	12433	207	--
	Develop	60	--	--	1791	30	--

All data from Kleinfelder (1999)

Table 4-3. Results of Re-Analyses of District Well 4R Aquifer Test

Monitor Well	Test	Analysis Method	Transmissivity	Hydraulic Conductivity	Storage Coefficient
SVPSD #4	Step Test	Theis, 1946	141000	2,350	0.0024
MW-98-05	Constant Rate	Theis, 1935	45000	750	0.037
Piezometer #1	Constant Rate	Theis, 1935	172000	2,867	0.015
Piezometer #2	Constant Rate	Theis, 1935	304000	5,067	0.091
SVPSD #4	Constant Rate	Theis, 1935	1500	25	0.2
SVPSD #2, SVMWC #2, MW-98-05	Constant Rate	Theis, 1935 Distance Drawdown	14800	247	0.085
SVPSD #2, MW98-05	Constant Rate	Thiem, 1906	15000	250	
SVPSD #2, SVPSD #1	Constant Rate	Thiem, 1906	12000	200	
SVPSD #3, SVMWC #2	Constant Rate	Thiem, 1906	44000	733	

All data from Kleinfelder (2000)

WATER BALANCE

A water balance was developed for the valley using quantitative estimates of all of the inflows, outflows, and change in storage. The sum of all recharge, discharge, and boundary flows over a period of time equals the change in groundwater storage during that same time period. The standard water balance equation is written as:

Sum of Inflows – Sum of Outflows = Change in Storage

The following sections describe each component of the Squaw Valley Water Balance.

Inflows

Inflows to Squaw Valley include all recharge mechanisms that add water to the groundwater system including:

1. Infiltration from rainfall and snowfall
2. Pipeline losses from District delivery system
3. Pipeline losses from Mutual delivery system
4. Return flow from irrigation of commercial properties.
5. Return flow from construction.
6. Return flow from irrigation of residential properties.
7. Return flow from golf course irrigation.
8. Pipeline losses from sewer system
9. Inflow from Squaw Creek
10. Inflow from bedrock fractures

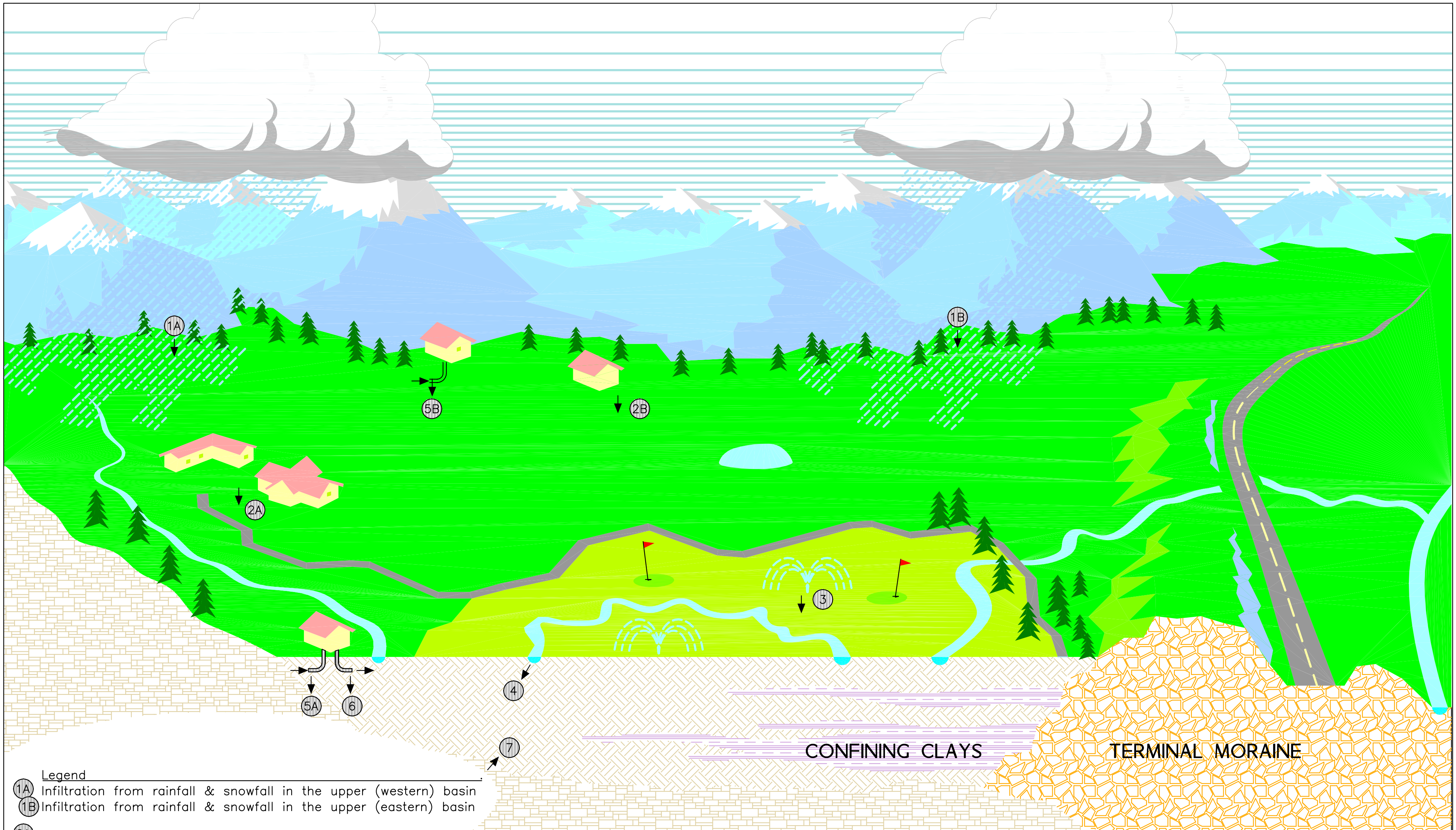
Figure 4-5 presents a conceptual diagram of the Squaw Valley Basin showing the various inflows. Discussions of each of the recharge mechanisms are presented in the Groundwater Model Report.

Outflows

Outflows from Squaw Valley include all discharge mechanisms that remove water from the groundwater system including:

1. Groundwater pumping by District
2. Groundwater pumping by Mutual
3. Groundwater pumping for golf course irrigation
4. Groundwater pumping for snow making
5. Outflows to Squaw Creek
6. Infiltration into the sewer system
7. Subsurface flow across the terminal moraine
8. Evapotranspiration

Figure 4-6 presents a conceptual diagram of the Squaw Valley Basin showing the various outflows. Discussions of each of the discharge mechanisms are also presented in the Groundwater Model Report.



Legend

1A Infiltration from rainfall & snowfall in the upper (western) basin
1B Infiltration from rainfall & snowfall in the upper (eastern) basin

2A Return flow from irrigation of commercial properties
2B Return flow from irrigation of residential properties

3 Return flow from golf course irrigation

4 Inflow from Squaw Creek

5A Pipeline losses from SVPSD delivery system
5B Pipeline losses from SVMWC delivery system

6 Pipeline losses from sewer system

7 Inflow from bedrock fractures

Figure 4-5

Squaw Valley
Groundwater Development & Utilization Feasibility Study

POTENTIAL GROUNDWATER RECHARGE MECHANISMS

W E S T
Y O S T
& ASSOCIATES

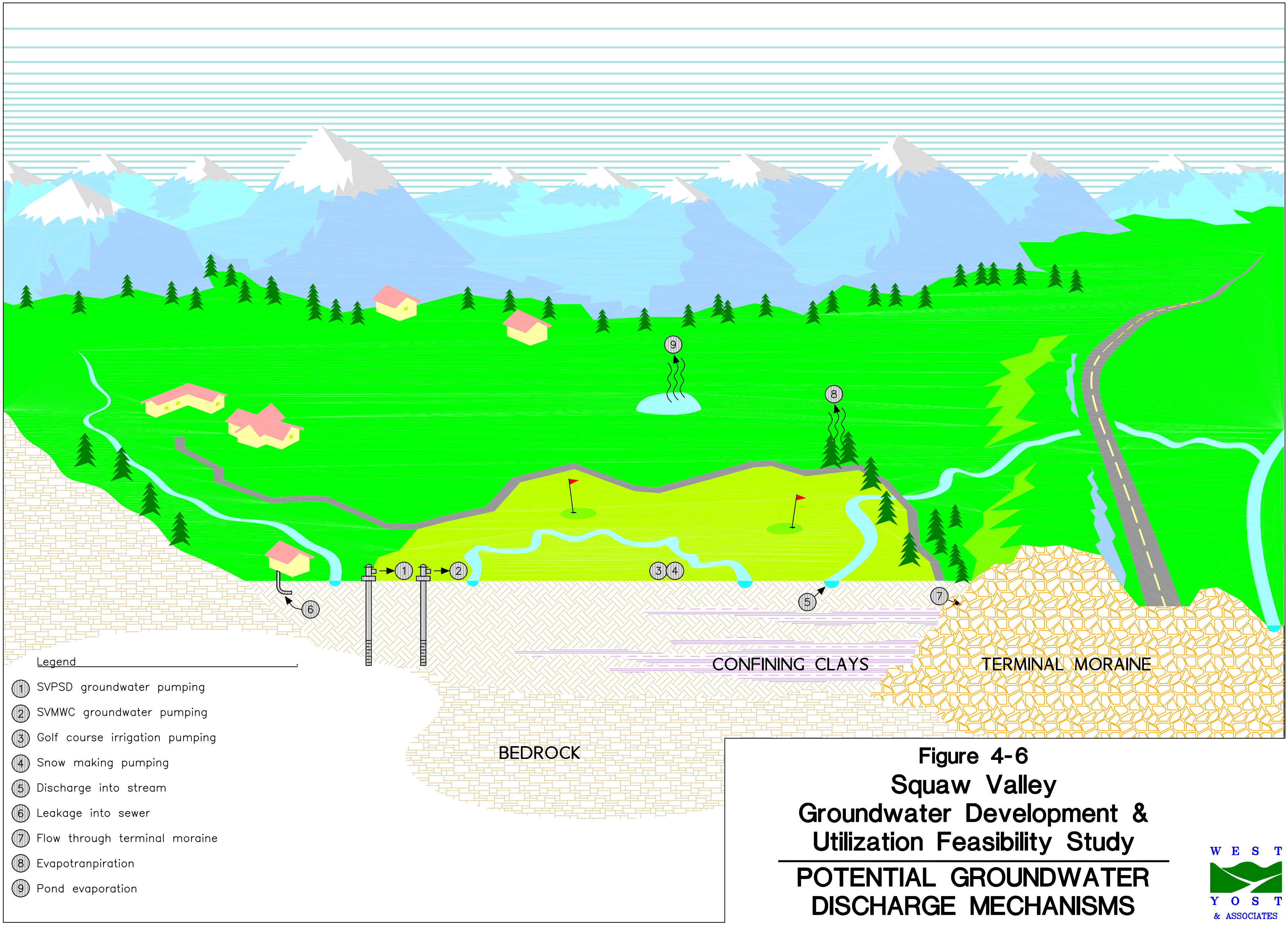


Figure 4-6

Squaw Valley Groundwater Development & Utilization Feasibility Study

POTENTIAL GROUNDWATER DISCHARGE MECHANISMS

Change in Storage

Change in groundwater storage is commonly estimated by multiplying changes in observed water table elevations over a certain time period by a storage coefficient. The Squaw Valley basin appears to fill with water every winter, and water levels are fairly consistent over time; there is no apparent long-term rise or fall in water levels. Therefore, the long-term change in storage is assumed to be zero on an average annual basis, although the basin experiences substantial change in storage seasonally.

Water Balance

The standard water balance equation balances basin inflows, outflows, and change in storage. The water balance equation is commonly written as:

$$\sum Inflows - \sum Outflows = Change\ in\ Storage$$

The average annual change in storage is assumed to be zero. Therefore, in an average annual water balance the inflows should approximately equal the outflows.

Inflows to the groundwater basin, based on estimates of the average annual flow rates, yields the following:

Infiltration from rainfall and snowfall .... 688 af/year
Pipeline losses from District delivery system .... 60 af/year
Pipeline losses from Mutual delivery system .... 22 af/year
Return flow from irrigation of commercial properties.. 12 af/year
Return flow from construction. .... 0 af/year
Return flow from irrigation of residential properties. .... 80 af/year
Return flow from golf course irrigation. .... 47 af/year
Pipeline losses from sewer system .... 137 af/year
Inflow from Squaw Creek ....Unknown
Inflow from bedrock fractures .... 436 af/year

Estimated outflows from the groundwater basin, using the average annual flow rates, yields the following:

Groundwater pumping by District .... 491 af/year
Groundwater pumping by Mutual.. 95 af/year
Groundwater pumping for golf course irrigation .... 158 af/year
Groundwater pumping for snow making .... 84 af/year
Outflows to Squaw Creek ....Unknown
Infiltration into the sewer system .... 0 af/year
Subsurface flow across the terminal moraine .... 220 af/year
Evapotranspiration .... 0 af/year

The net inflow of groundwater from Squaw Creek or outflow of groundwater into Squaw Creek is unknown. If we assume the change in storage is zero, the flow into or out of Squaw Creek can be treated as a single term in the water budget, as follows.

$$\sum \mathit{Known\ Inflows} - \sum \mathit{Known\ Outflows} - \sum \mathit{Squaw\ Creek\ Interaction} = 0$$

Using the values estimated above:

1482 af/year – 1048 af/year - Squaw Creek Interaction = 0

Squaw Creek Interaction = 434 af/year

The estimated net discharge of 434 af of groundwater into Squaw Creek annually has a large uncertainty, but is reasonable in light of Tom Sheahan's estimate that the total amount of water in storage is approximately 4,600 af (Sheahan, 1990). Because Squaw Creek only intercepts the upper portion of the aquifer, only groundwater stored in the uppermost portion of the aquifer is available for discharge into Squaw Creek. Therefore, it can be anticipated that only a relatively small percentage of the total groundwater in storage can discharge to Squaw Creek. The estimated annual discharge of 434 af is approximately 10 percent of the estimated groundwater storage of 4,600 af.

The complete conceptual water balance is presented on Table 4-4. This water budget is based on a number of assumptions that introduce potential error into the calculations. The net interaction between Squaw Creek and the groundwater basin is derived as the remainder of the water balance. An independent estimate of this interaction is not available.

The total potential recharge and pumping discharge estimates for each month are shown on Figure 4-7. This figure shows how pumping exceeds recharge in the summer and autumn months, resulting in the observed water level declines. Figure 4-7 also shows how recharge exceeds pumping in winter and spring months, when water level rises are observed.

NUMERICAL FLOW MODEL CONSTRUCTION

Numerical flow model construction consists of defining the structure of the model, then incorporating the data from the conceptual model. Defining the model structure includes defining the model domain, constructing a model grid, and defining model layers. Incorporating the conceptual model includes assigning boundary conditions, assigning hydrogeologic parameters, and incorporating components of the water balance. The recharges and discharges in the water balance are expressed in the model through areal recharge rates, well pumping rates, and flow rates across model boundaries.

Model Code Selection

The model code MODFLOW (McDonald and Harbaugh, 1988) was selected for the groundwater flow model. This model program is well documented and verified, and the District or other future consultants can easily run the model.

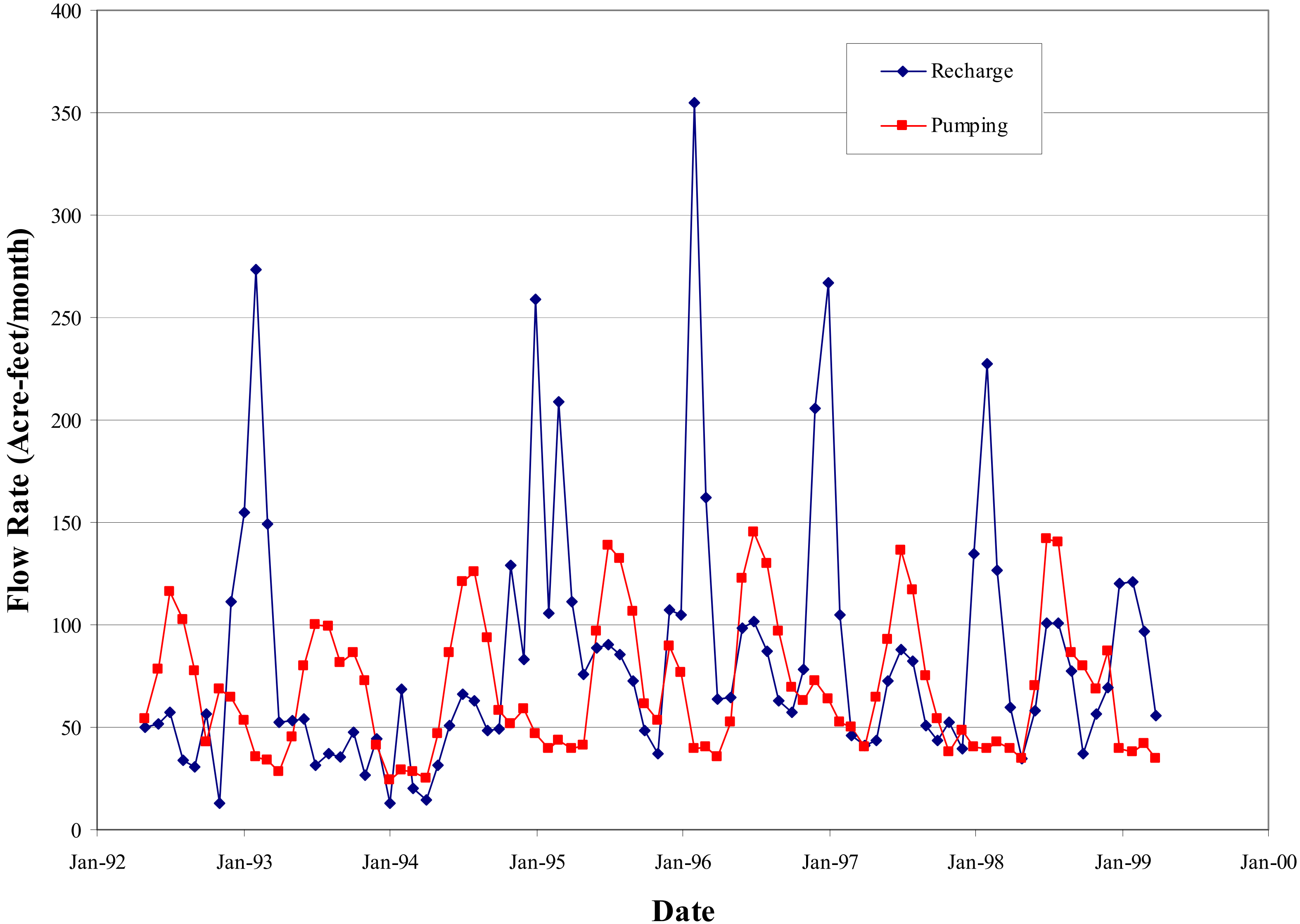
Model Domain

The groundwater model covers the approximately 370 acres of the valley floor as shown on Figure 4-8. The model domain is bounded on the North, West, and South by the estimated location of bedrock outcrops. The model domain is bounded on the East by the estimated location of the terminal moraine.

Table 4-4. Conceptual Squaw Valley Water Balance

	af/year
Inflows	
Potential Precipitation Infiltration	688
Pipeline Losses – SVPSD	60
Pipeline Losses – SVMWC	22
Irrigation Return Flow - Commercial	12
Construction Return Flow	0
Irrigation Return Flow – Residential	80
Golf Course Irrigation Return Flow	47
Sewer Losses	137[a]
Inflow from Squaw Creek	_[b]
Inflow from Bedrock Fractures	436
Total	1483
Outflows	
Groundwater Pumping – SVPSD	491
Groundwater Pumping – SVMWC	95
Groundwater Pumping – Golf Course	158
Groundwater Pumping – Snowmaking	84
Outflow to Squaw Creek	435[b]
Sewer Infiltration	0[a]
Flow Across Terminal Morraine	220
Evapotranspiration	0
Total	1483
Change in Storage	0

(a) Sewer losses and gains summed into net sewer losses
(b) Calculated as difference between inflows and outflows



SVPSD Groundwater Model Development

Figure 4-7. Estimated Total Recharge and Pumping

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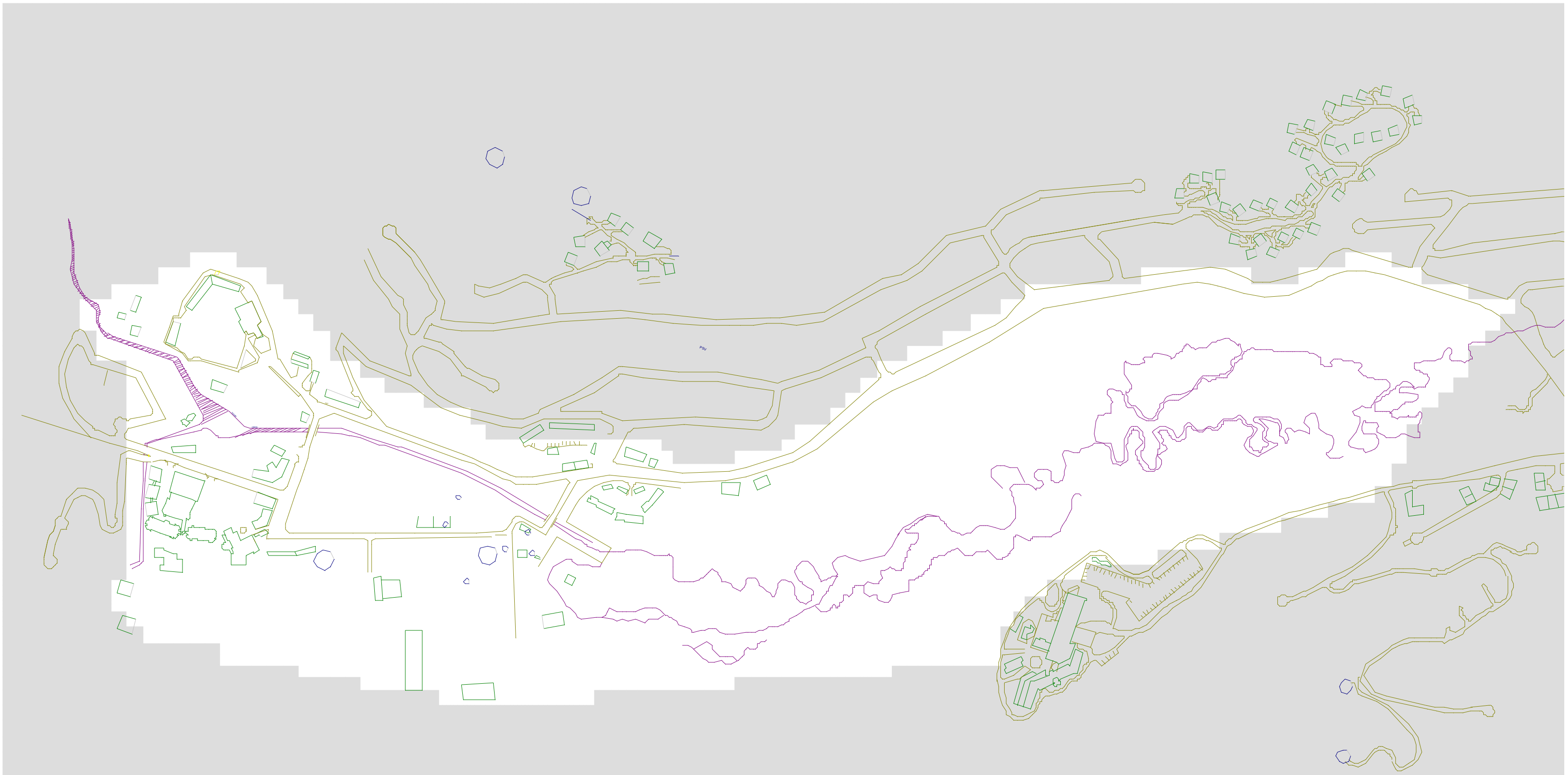

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Figure 4-8. Extent of Groundwater Model

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Finite Difference Grid

Figure 4-9 shows the finite difference model grid on which the numerical model is built. The grid comprises 74 rows and 146 columns. The largest grid dimension is 100 feet, and the smallest grid dimension is 25 feet. The smallest grid dimensions are placed around municipal and irrigation wells to help define the steeper groundwater gradients expected near these wells. Smaller grid cells are required to accurately model strongly curving piezometric surfaces, such as those induced by groundwater pumping (Anderson and Woessner, 1992). To minimize numerical errors, cell dimensions are always less than or equal to 1.5 times any neighboring cell dimension.

The finite difference grid was oriented parallel to the assumed primary flow directions where possible. Groundwater generally flows along the length of the valley, from the southwest towards the terminal moraine in the Northeast. To best accommodate this general flow direction, the model grid was rotated 20 degrees counterclockwise. On Figure 4-8, the underlying map of Squaw Valley has been rotated 20 degrees clockwise rather than rotating the model grid 20 degrees counterclockwise. This was done to maximize the area of the rectangular grid on the paper.

Model Layers

Vertical gradients have been observed in the Squaw Valley Basin. To represent the multiple piezometric surfaces, hydro-stratigraphic complexity, and three-dimensional flow characteristics of the Squaw Valley Basin, the numerical groundwater model is divided into three layers. The alluvial deposits do not appear to display any basinwide continuity or structure. Therefore, layering did not follow any stratigraphic horizons or geologic structures. The depth and thickness of these layers were based on existing groundwater monitoring and groundwater pumping data. Example east-west and north-south cross sections of the model layering are shown on Figures 4-10 and 4-11.

Layer 1 is the top layer in the model grid. Layer 1 simulates the top 15 feet of basin sediments. The water table lies in Layer 1 across most of the basin. Layer 1 is not monitored by any monitoring wells, and no groundwater pumping takes place in Layer 1.

Layer 2 represents the basin sediments between 15 and 30 feet below ground surface (bgs). This is the layer monitored by the shallow monitoring wells installed in the golf course and meadow in the eastern half of the basin. No groundwater pumping takes place in Layer 2.

Layer 3 represents the remaining thickness of the basin below 30 feet bgs. Layer 3 ranges between 5 and 140 feet thick. The depth of the bottom of Layer 3 was determined using a combination of the geophysical data developed by Gasch & Associates (1973) and well logs. Discrete depths to the base of the basin were estimated along the five seismic lines run by Gasch & Associates. These depths were combined with depth to bedrock data from all wells that were drilled to bedrock. This combination of seismic data and well log data were contoured to estimate a depth to bedrock surface. The contoured surface was then compared with the logs of deep wells that did not encounter bedrock. The contoured surface was modified as necessary to reflect the minimum depth to bedrock implied by the deep wells that did not encounter bedrock. A map of the elevation of the bottom of the model is shown in Figure 4-12.

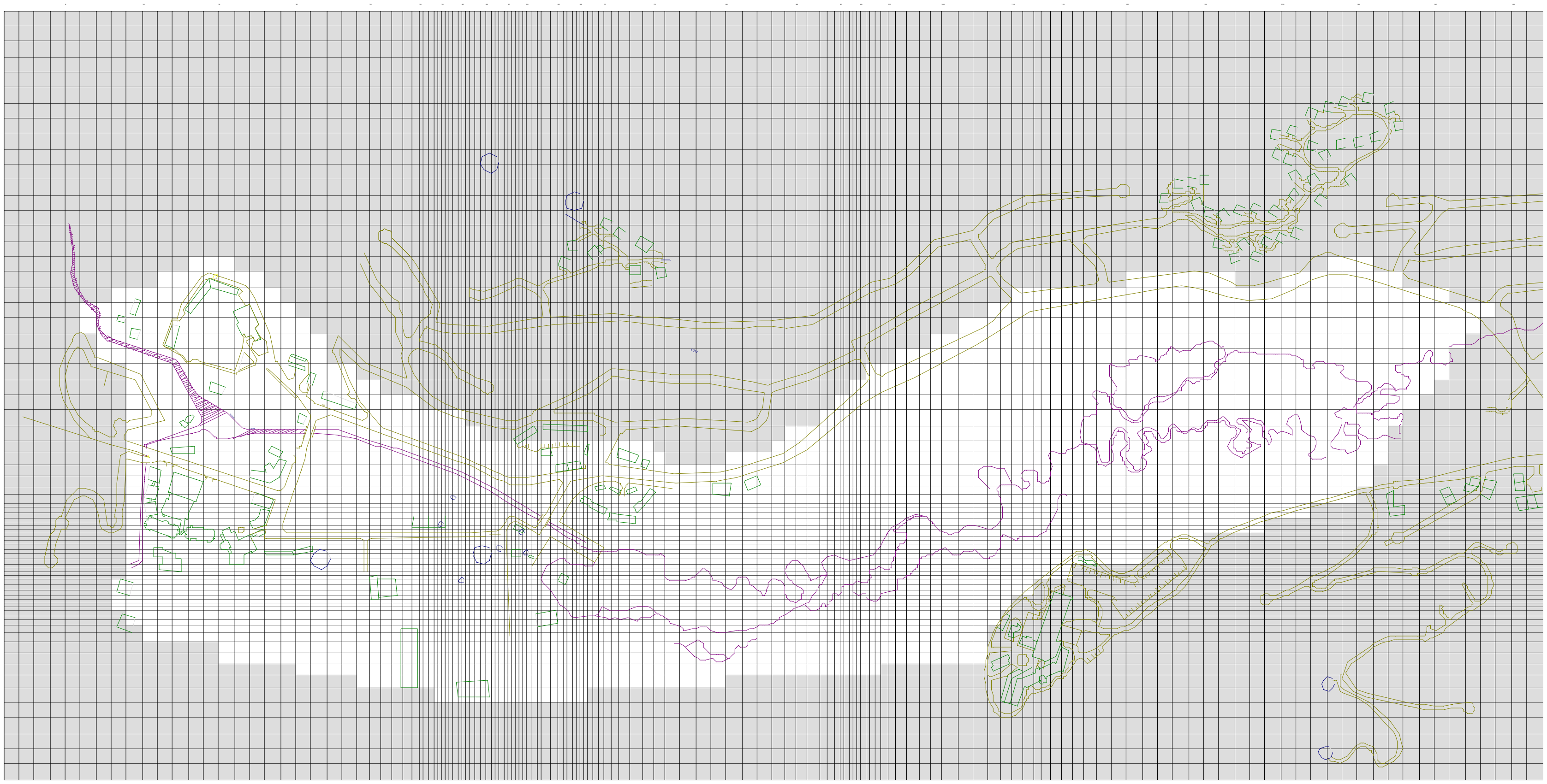

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Figure 4-9. Groundwater Model Grid

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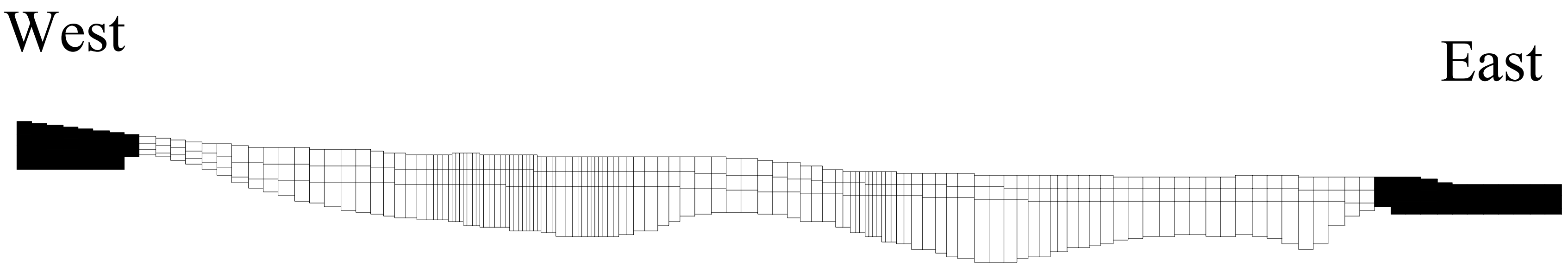

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Figure 4-10. Example East-West Model Grid Cross Section

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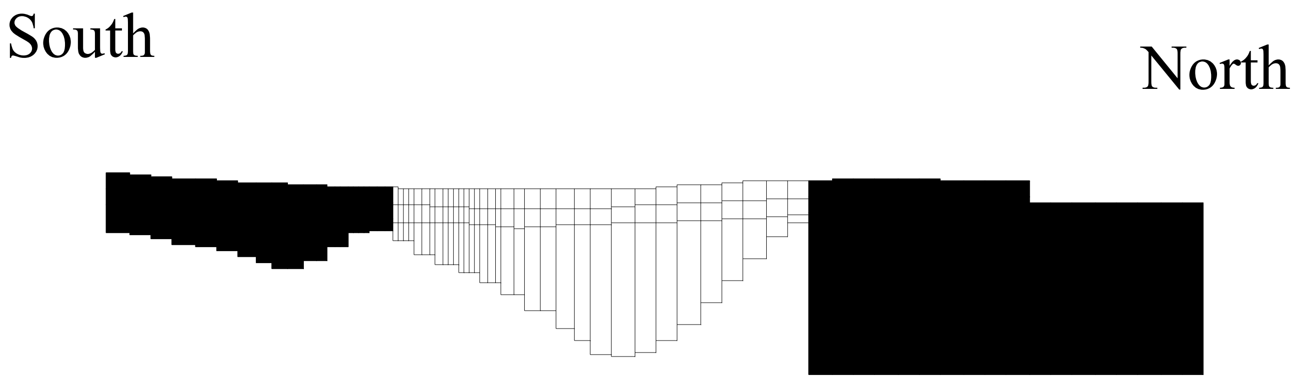

SVPSD Groundwater Model Development

Figure 4-11. Example North-South Model Grid Cross Section

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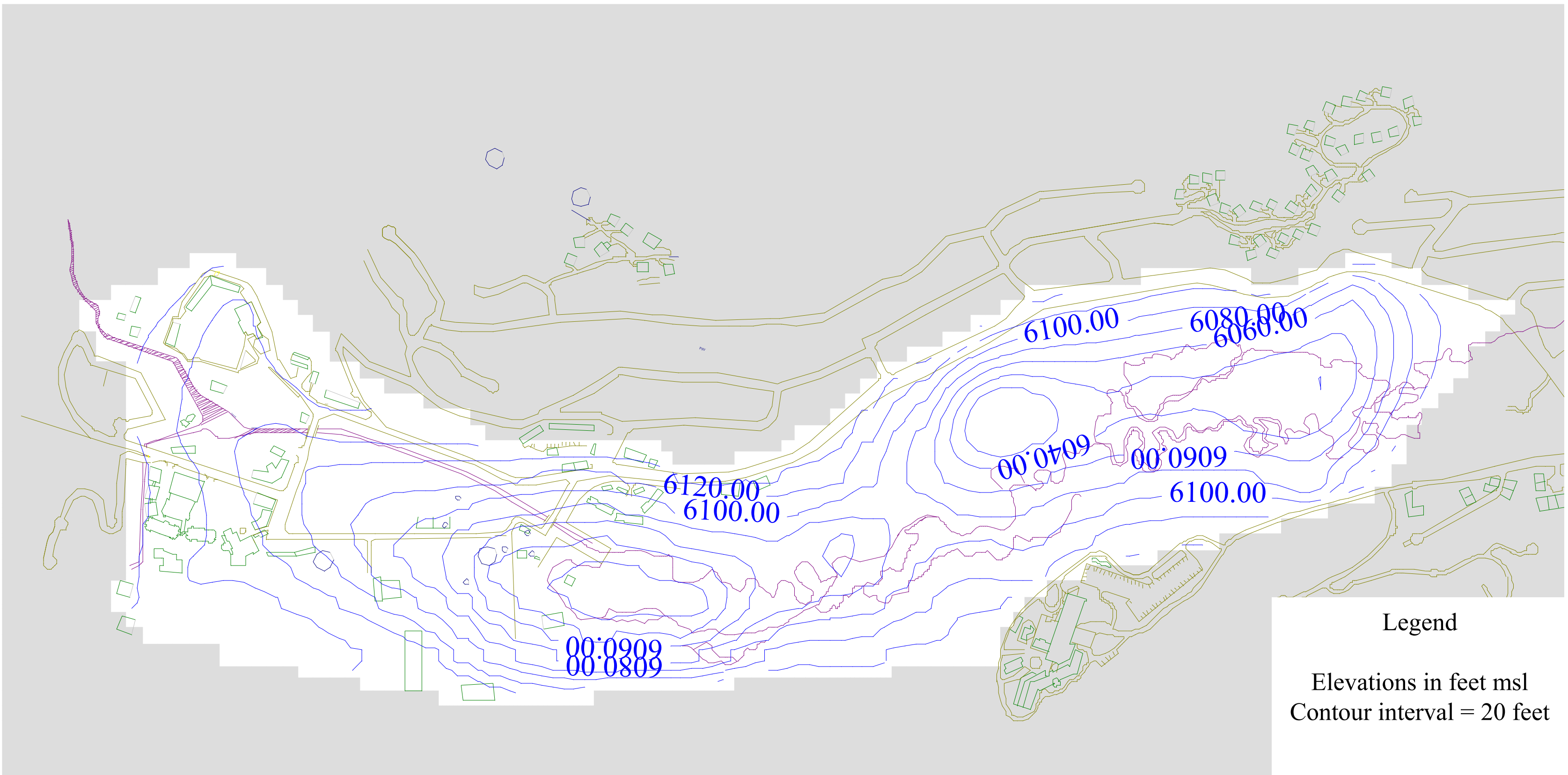

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Figure 4-12. Base of Model Elevation

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Layer 3 is the layer monitored by the deep monitoring wells installed in the golf course and meadow in the eastern half of the basin. This layer is also monitored by the municipal and irrigation production wells. All groundwater pumping from the municipal and irrigation production wells occurs in Layer 3.

Boundary Conditions

The numerical model requires defined boundary conditions for all sides, the top, and the bottom of the model. Boundary conditions along the top and sides of the model are designated explicitly in the model input; the boundary condition along the bottom surface of the model is controlled by layer definition. Horizontal boundaries that are not explicitly assigned boundary conditions are treated as no-flow boundaries by MODFLOW.

Mountain Front Boundaries. The Squaw Valley Basin is bounded by mountains on the North, West, and South sides. Two types of boundary conditions are imposed along the contacts between water bearing alluvium and relatively non-water bearing crystalline rock: no-flow boundaries and general-head boundaries. No-flow boundaries are used to represent most of the alluvium/bedrock contact. These boundaries reflect the fact that the alluvium is far more transmissive than the crystalline bedrock. General head boundaries represent areas where fractures in the crystalline rock intercept the basin boundary. The fracture locations were estimated from maps of fractures produced by Kleinfelder (1991). The general head boundaries allow water to flow into the basin, with a maximum flow rate of 30 gpm for each fracture set. Figure 4-13 shows the location of the general head boundaries representing fractures.

Terminal Moraine Boundary. The terminal moraine at the eastern edge of the basin is represented by a constant head boundary. This boundary allows some flow out of the basin through the terminal moraine. During calibration, flow through this boundary was checked to ensure that the flow through the moraine was reasonable, based on previous estimates. The location of the constant head boundary cells for Layers 2 and 3 is shown on Figure 4-14.

Top and Bottom Boundaries. The Squaw Valley Basin appears to fill with water every winter. After the basin is full, excess recharge is rejected, i.e. the excess recharge flows overland to Squaw Creek. To simulate the overland flow of rejected recharge, the top boundary of the model is a represented by drain cells. Drain cells function by removing water from the model when water levels reach a pre-determined drain elevation. In the current model, the drain elevation is set to the ground surface at all points in the model. Ground surface elevations were based on surveyed elevations of wells provided by West Yost & Associates. Additional surface elevation points were picked off the Intrawest contour map of the Squaw Valley Parking Lot. By setting the drain elevation to the ground surface elevation, water levels are allowed to rise to the ground surface, and all water recharge above the ground surface is drained away.

The bottom boundary is a no flow boundary. This boundary represents the contact between the bottom of the alluvium and the underlying crystalline rock.

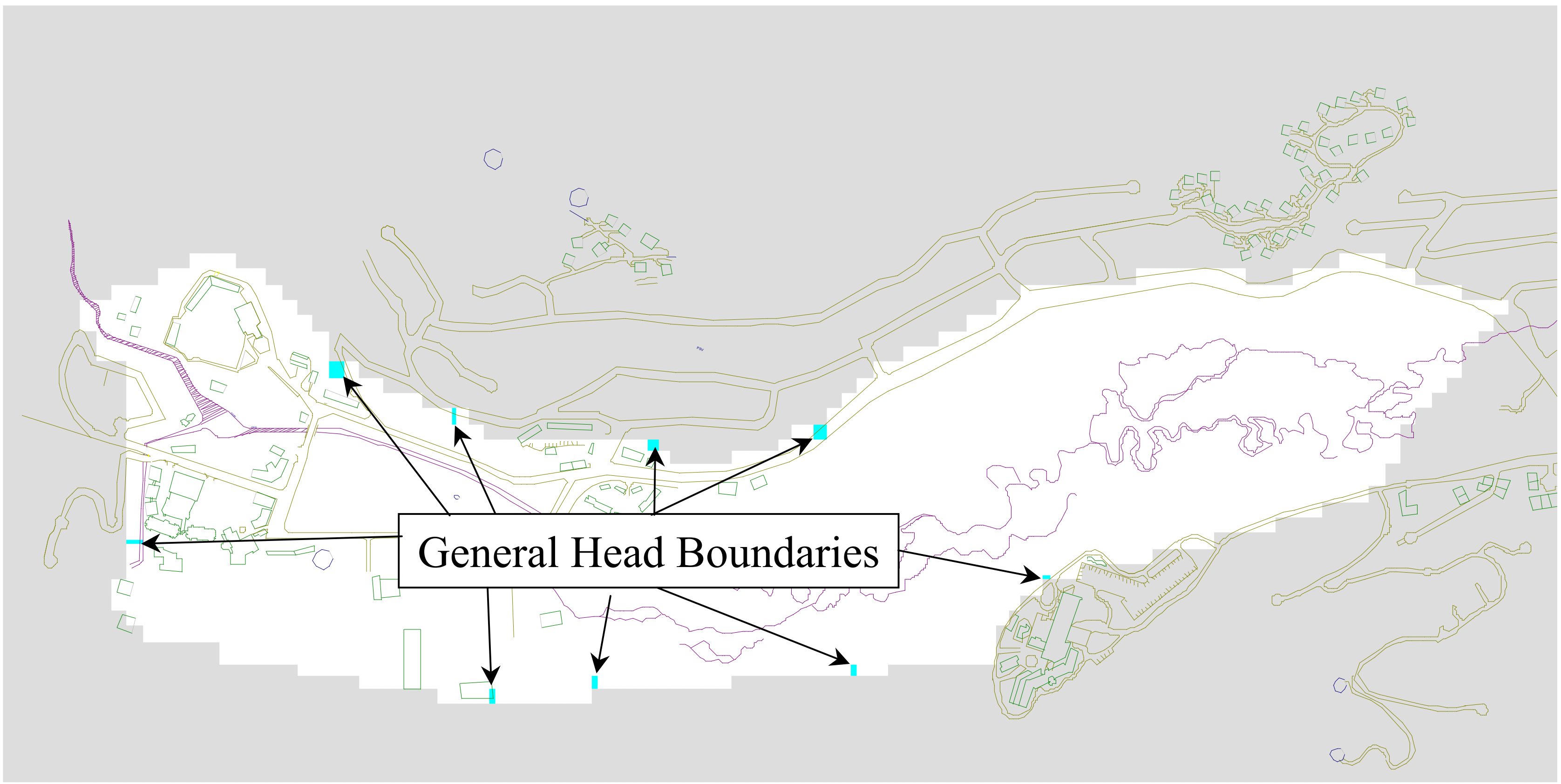

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Figure 4-13. General Head Boundary Locations

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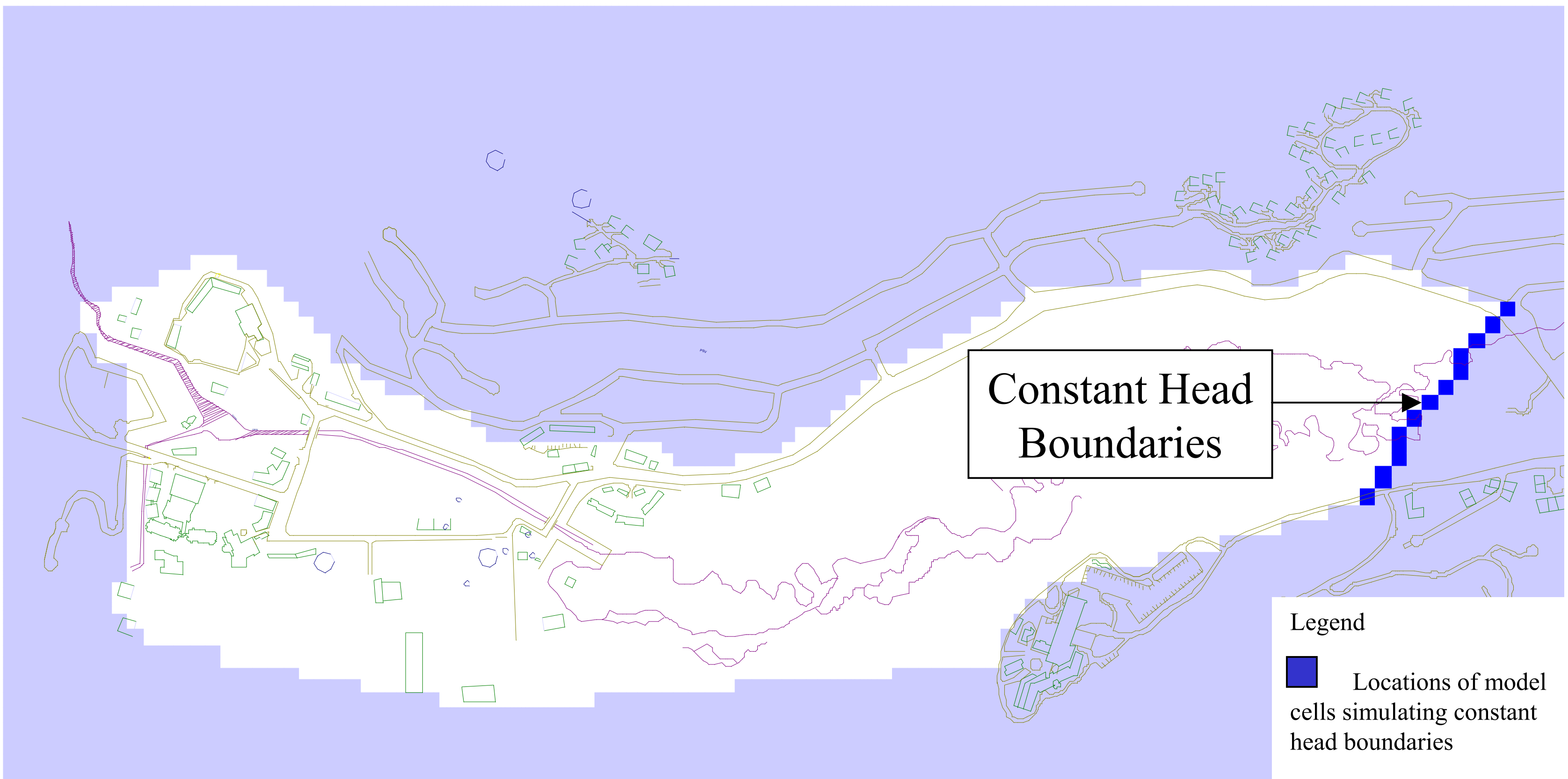

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Figure 4-14. Constant Head Boundary Locations

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Stream Boundary. Squaw Creek was simulated with a stream boundary condition. The stream boundary condition estimates the water level in Squaw Creek, allowing water to flow into or out of Squaw Creek. If the water level in the creek is higher than the water level in the adjoining aquifer, water will flow out of the creek, recharging the basin. If the water level in the creek is lower than the water level in the adjoining aquifer, water will flow into the creek from the basin. Figure 4-15 shows the location of the model cells that simulate the stream boundary.

Creek bottom elevations were derived from survey data, and estimated from observations. Where possible, survey data provided by West Yost & Associates were used to determine the stream bottom elevation. Additional survey points were picked off the Intrawest contour map of the Squaw Valley Parking Lot. Where survey points were not available, the creek bottom elevation was estimated. Generally, the creek bottom elevation was estimated to be approximately ten feet bgs in the western portion of Squaw Valley, and approximately three feet bgs in the meadow and golf course areas.

Hydrogeologic Parameters

The groundwater model program requires hydrogeologic parameters to be assigned to every active cell in the model domain. Required hydrogeologic parameters may include horizontal conductivity or transmissivity, vertical hydraulic conductivity, and storage coefficients.

Models with good predictive ability generally incorporate a limited number of aquifer parameters (Freyberg, 1988). The number of parameters is kept small by replacing individual values at every cell with average values applied over a group of cells with similar hydrogeologic properties. A group of cells with the same parameter value constitute a parameter zone.

Parameter zones are based on the distribution of available parameter measurements, and other measurements that indirectly control or indicate a parameter value. Available data are incorporated with discretion, based on such factors as data quality, hydrogeological interpretation, and professional judgment. For example, aquifer test results, lithologic data, and general piezometric surface features may be used to define zones of horizontal hydraulic conductivity. Results of representative aquifer tests, however, will likely influence the conductivity zonation more than piezometric surface features, because aquifer test results more directly estimate hydraulic conductivity.

The following sections present the aquifer parameter values assigned in the numerical groundwater model, and the justification for the values.

Horizontal Hydraulic Conductivity and Transmissivity. Horizontal hydraulic conductivity and transmissivity values are assigned to each model layer based on the following data:

- Lithologic and geophysical data
- Aquifer test results
- Piezometric surface features

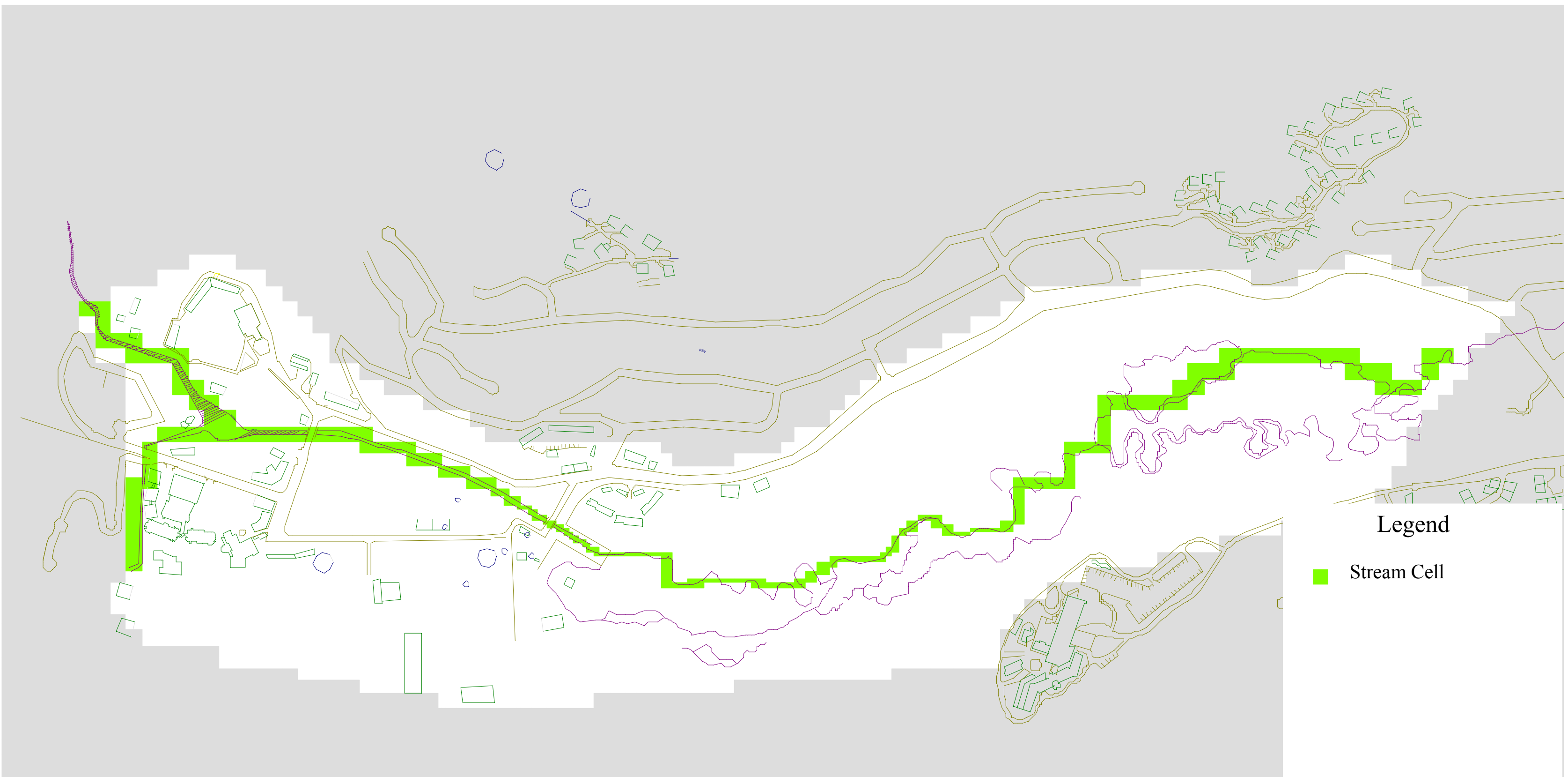

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Figure 4-15. Location of Model Cells Simulating Squaw Creek

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In many basins, aquifer tests represent the best source of horizontal hydraulic conductivity data. Results from longer-term aquifer tests that included observation wells are considered more representative than results from low-yield, short-term aquifer tests conducted on monitoring wells. Because of the limited number of long-term aquifer tests in Squaw Valley, the distribution of aquifer parameters was primarily estimated using lithologic data.

Lithologic descriptions for every well in Squaw Valley were assigned to a lithologic zone. The description of each zone and the zone numbers are listed on Table 4-5.

Table 4-5. Relative Conductivity Values Assigned Lithologic Descriptions

Lithologic Descriptions	Value
Peat, organic clays, inorganic clays, sandy clays, silty clays, silts	1
Very fine sands/Silts	2
Silty sand/silt mixtures, clayey sands	3
Silty sands	4
Silty sand/sand mixtures	4.5
Clayey gravels, silty gravels	5
Clean Sands	5.5
Clayey gravel/gravel mixtures, silty gravels	6
Sandy gravels	6.5
Clean gravels	7

Average lithologic zone numbers were determined for each model layer at each well location. For example, at District Well 1, the average lithologic zone number for the upper 15 feet is 1.8. This corresponds to a silty to clayey lithology. The average lithologic zone number in the depth range between 15 and 30 feet bgs is 3.6. This corresponds to a silty sand. The average lithologic zone number in the depth range below 30 feet bgs is 5.76. This corresponds to a clean sand or gravel. The average lithologic zone numbers were contoured across the basin to develop generalized lithologic maps.The parameter values assigned to each lithologic zone were based on the results of the limited aquifer tests and hydrogeologic experience. The initial values assigned to each zone are listed in Table 4-6.

Table 4-6. Initial Parameter Estimates

Parameter Zone	Horizontal Hydraulic Conductivity, feet per day	Vertical Hydraulic Conductivity, feet per day	Storativity	Specific Yield
1	0.05	0.001	$1x10^{-6}$	0.1
2	0.1	0.002	$1x10^{-6}$	0.1
3	0.5	0.01	$1x10^{-6}$	0.1
4	5	0.1	$1x10^{-5}$	0.12
5	50	2	$1x10^{-5}$	0.15
6	200	20	$1x10^{-5}$	0.15
7	250	25	$1x10^{-5}$	0.15

Vertical Hydraulic Conductivity. In the groundwater flow model, vertical hydraulic conductivity controls the ease with which groundwater flows vertically between model layers. No direct measurements of vertical hydraulic conductivity exist in Squaw Valley. Therefore, estimates were derived based on hydrogeologic experience.

The initial zone pattern for vertical hydraulic conductivity was identical to the zone pattern for horizontal hydraulic conductivity. The initial values for vertical hydraulic conductivity are presented in Table 4-6.

Storage Coefficients. Storage coefficients are expressed as a specific yield in unconfined aquifers, and as a storativity in confined aquifers. Both coefficients are unitless. Very few measurements of storage coefficients exist in Squaw Valley; therefore initial estimates of storage coefficients were based on average published values.

The initial zone pattern for storativity and specific yield was identical to the zone pattern for horizontal hydraulic conductivity. The initial values for storativity and specific yield are presented in Table 4-6.

Model Water Balance

Specific elements of the water balance described in the conceptual model section are simulated using the MODFLOW recharge and well packages. Flow across boundaries is incorporated through the boundary conditions described previously. Special considerations for both the recharge and well inputs are described below.

Recharge Package

Ten recharge zones were identified and the recharge was distributed in the numerical model. The ten recharge zones include:

1. No Recharge
2. Rainfall recharge in the western basin
3. Rainfall recharge in the eastern basin
4. Rainfall recharge in the eastern basin and return flow from golf course irrigation
5. Rainfall recharge in the western basin, pipeline losses from Mutual distribution system, return flow from irrigation with Mutual water, and sewer inflows and outflows.
6. Rainfall recharge in the eastern basin, pipeline losses from Mutual distribution system, pipeline losses from District distribution system, return flow from irrigation with Mutual water, and sewer inflows and outflows.
7. Rainfall recharge in the eastern basin, return flow from irrigation with District water, and sewer inflows and outflows.
8. Rainfall recharge in the eastern basin, pipeline losses from District distribution system, return flow from irrigation with District water, and sewer inflows and outflows.
9. Rainfall recharge in the eastern basin, pipeline losses from District distribution system.
10. Rainfall recharge in the western basin, pipeline losses from District distribution system, return flow from irrigation with District water (commercial and residential), and sewer inflows and outflows.

Return flow from irrigation was applied to assumed irrigated properties. Residences north of the valley irrigate during summer months. These residences lie among the streets north of the modeled area shown on Figure 4-8. Therefore the return flow from these properties was assigned to the northern boundary of the model (Zones 5, 6, and 8). The District supplies commercial irrigation water to businesses in the western portion of the basin. The area covered by commercial irrigation is estimated by recharge Zone 10.

Pumping Data

The pumping data supplied by the District and the Mutual were incorporated directly into the model. Pumping for these municipal wells was assigned to Layer 3 of the model cell where the wells are located. Pumping for Wells 18-1, 18-2, and 18-3 were estimated based on typical operations and relative pumping capacity of each well. The pumping for these three wells was assigned to Layer 3 of the model cell where the wells are located. Figure 4-16 shows the locations of the pumping wells.

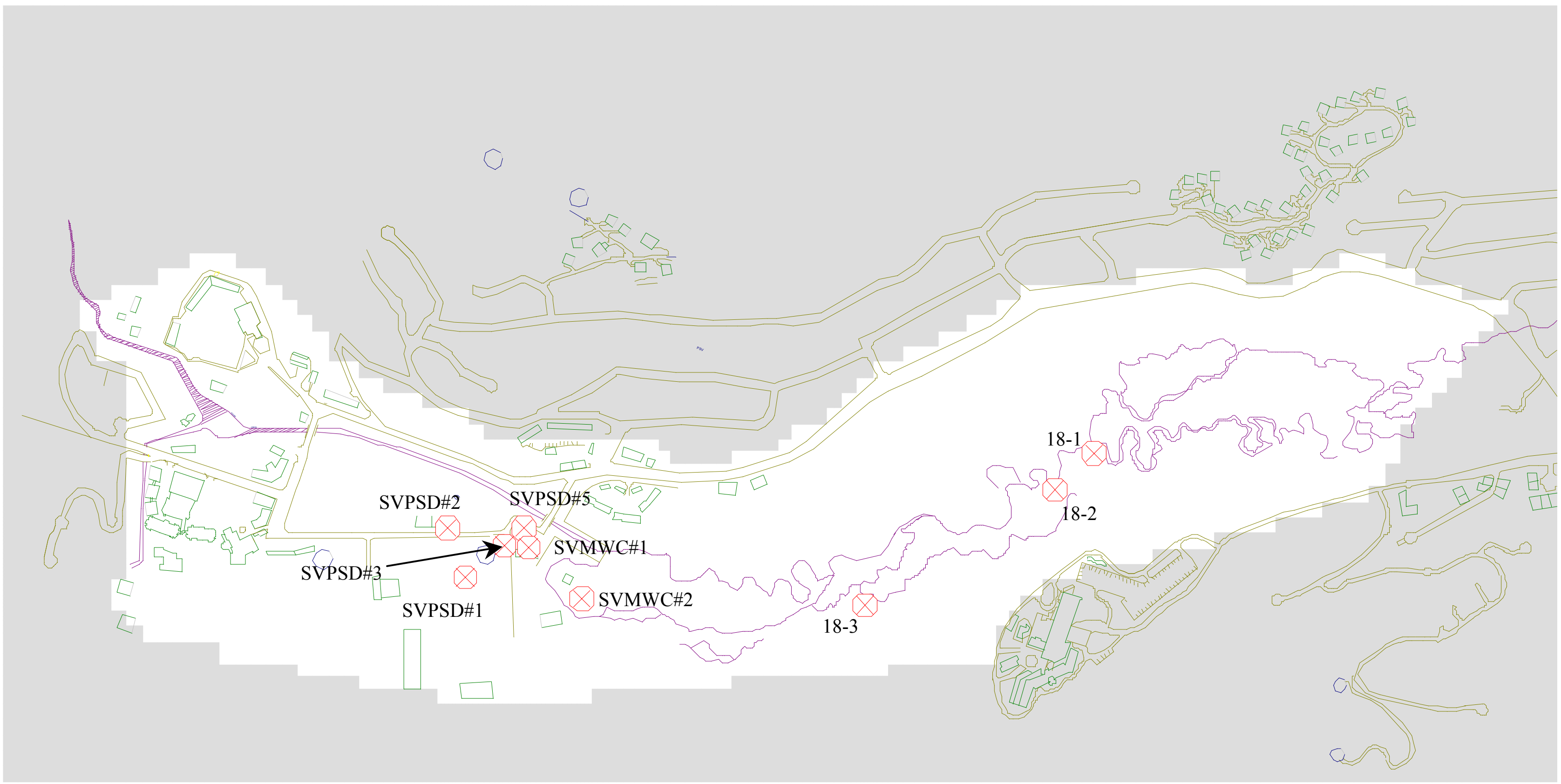

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Figure 4-16. Modeled Well Locations

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MODEL CALIBRATION

Calibrating the groundwater flow model involved successive attempts to match model output to measured data from the calibration period. Simulated hydraulic heads, stream flows, and subsurface flows through the terminal moraine were compared against available measured water levels, estimated stream flows, and estimated subsurface flows during calibration. The model was considered calibrated when simulated results matched the measured data within an acceptable measure of accuracy typical for groundwater modeling and when successive calibration attempts did not notably improve the calibration statistics. Calibration was conducted by varying relatively uncertain and sensitive parameters such as horizontal and vertical hydraulic conductivities, over a reasonable range of values.

Calibration Period

The primary criterion for choosing the appropriate calibration period was the availability of a relatively complete set of data. The necessary data included complete pumping data; recharge data, and water level data from the network of groundwater monitor wells. The first complete set of data necessary for modeling was from May 1992. The last complete set of pumping and recharge data available at the beginning of this project was for April 1999. Therefore, the seven-year period of May 1992 through April 1999 was chosen as the calibration period.

Stress Periods

Stress periods define a time period in the groundwater model during which hydraulic stresses such as pumping and recharge are held constant. Stress period selection depends on the model objective and the time frame of interest. The primary objective of the current model is to assist with groundwater management strategies over the time of one year to multiple years. Because seasonal fluctuations in groundwater levels are important in groundwater management, the stress periods must be at least seasonal. Based on the existing data and model objectives, monthly stress periods were chosen. These stress periods allow adequate resolution of seasonal water level fluctuations while performing the simulations in a reasonable amount of time.

Stress periods can be subdivided into separate time steps in MODFLOW. In the current model, a single time step simulates each stress period. Comparisons of simulations using a single time step and multiple time steps for every stress period showed little difference in the calibration results.

CALIBRATION RESULTS

Model Parameter Modifications

Model parameters are adjusted during calibration to improve the model's ability to simulate known conditions. Most of the calibration of the Squaw Valley Basin model consisted of modifying the distribution and magnitude of hydraulic conductivity values. The final distributions of hydraulic conductivity values are shown for each of the three model layers in Figures 4-17 through 4-19. The final parameter estimates are listed in Table 4-7.

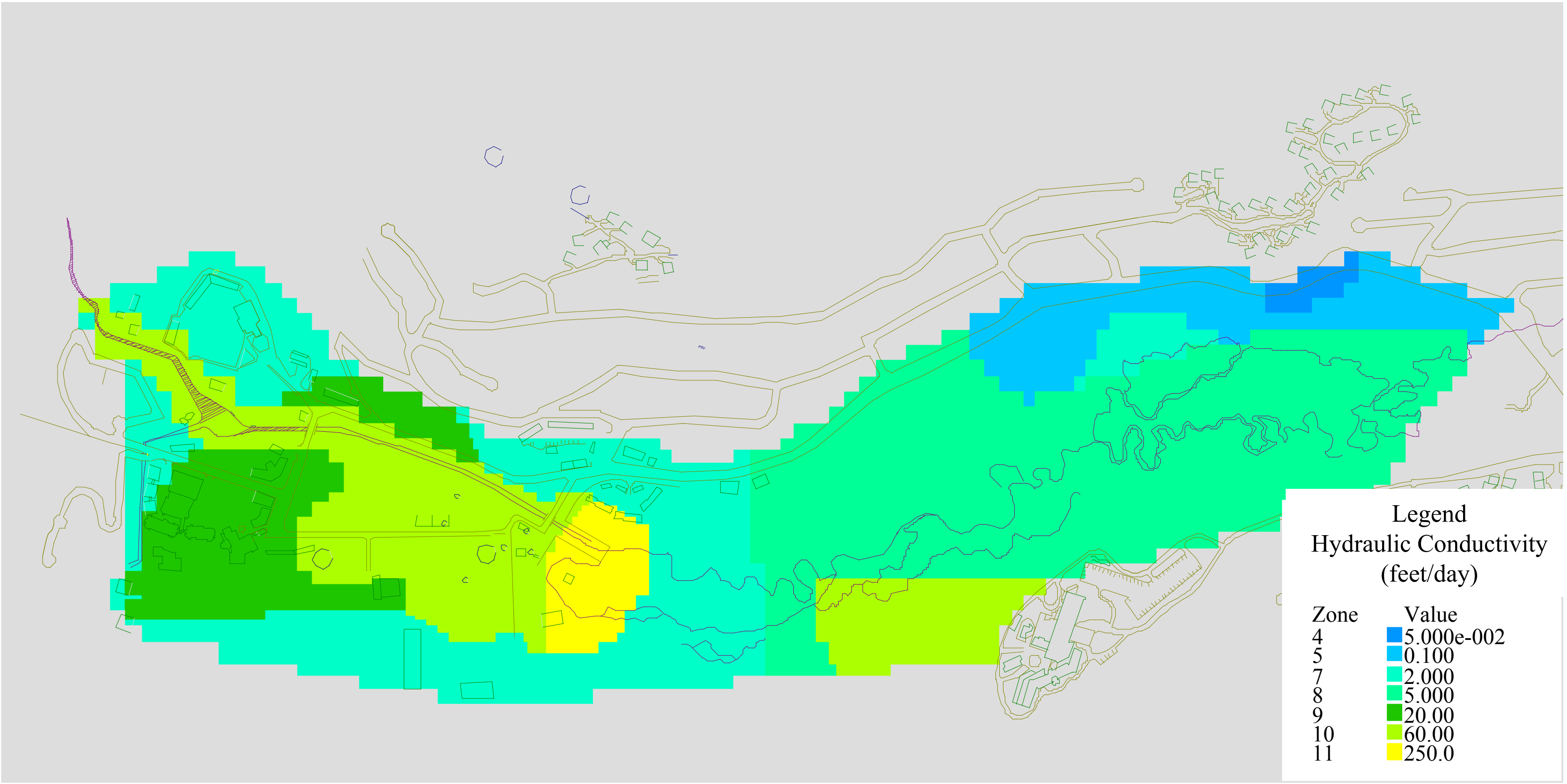

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Figure 4-17. Final Hydraulic Conductivity Distribution, Layer 1

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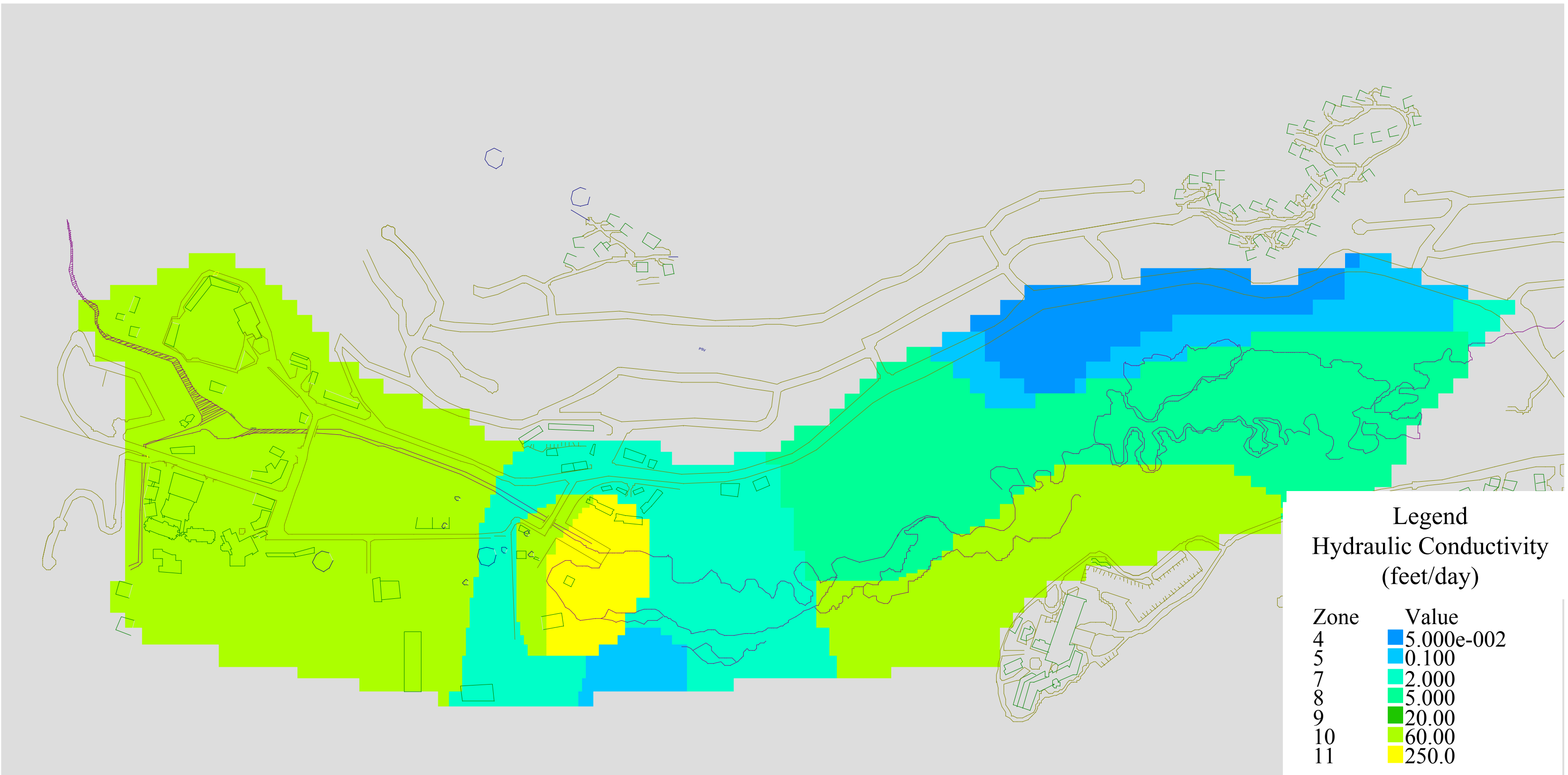

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Figure 4-18. Final Hydraulic Conductivity Distribution, Layer 2

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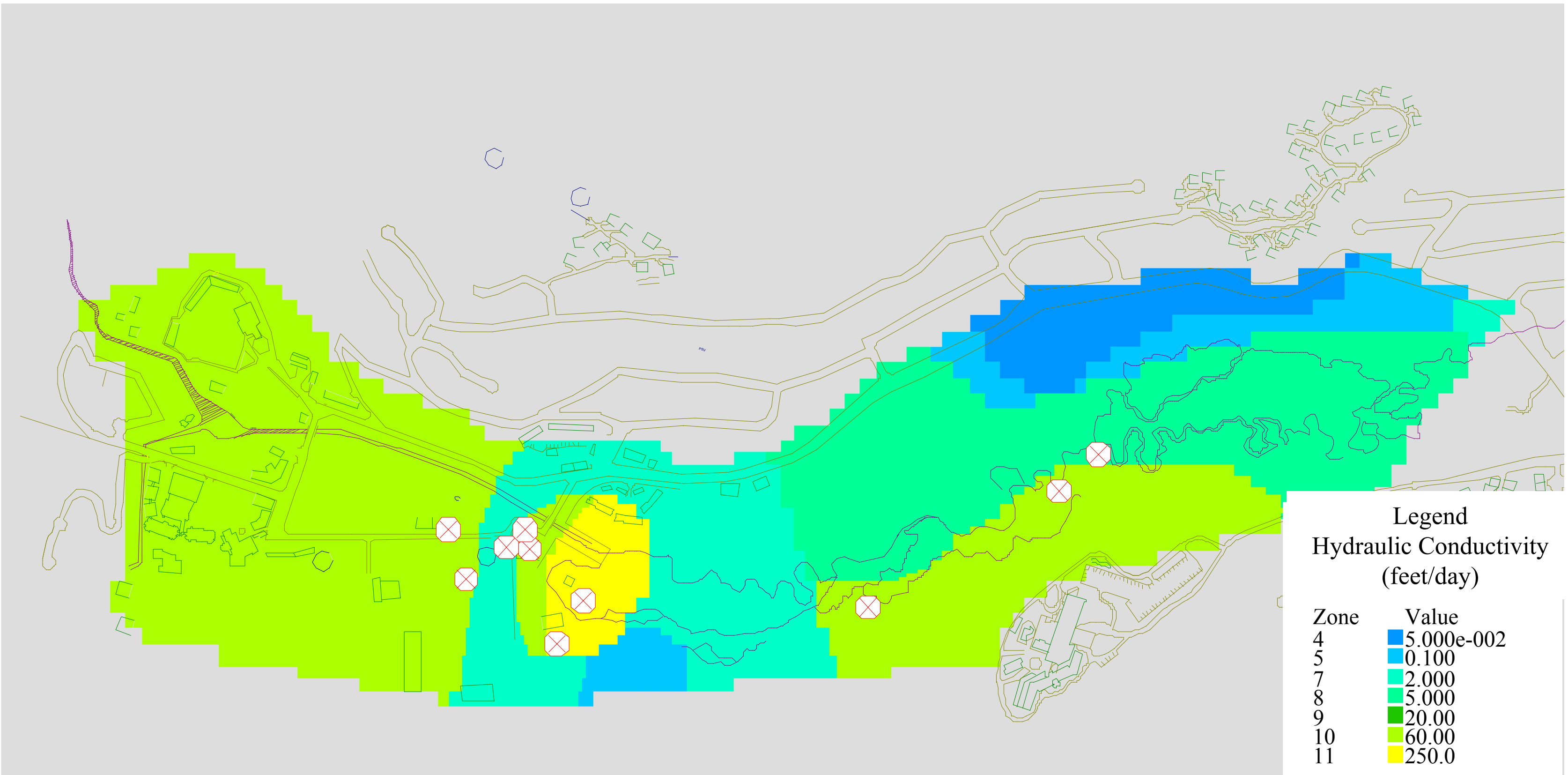

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Figure 4-19. Final Hydraulic Conductivity Distribution, Layer 3

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Table 4-7. Final Parameter Estimates

Parameter Zone	Horizontal Hydraulic Conductivity, feet per day	Vertical Hydraulic Conductivity, feet per day	Storativity	Specific Yield
4	0.05	0.007	$1x10^{-6}$	0.1
5	0.1	0.003	$1x10^{-6}$	0.1
7	2	0.07	$1x10^{-6}$	0.1
8	5	0.1	$1x10^{-5}$	0.12
9	20	0.02	$1x10^{-5}$	0.15
10	60	0.6	$1x10^{-5}$	0.15
11	250	2.5	$1x10^{-5}$	0.15

Water Level Calibration

Flow model calibration is commonly evaluated by comparing simulated water levels with measured water levels from monitoring wells. Hydrographs of simulated water levels should generally match the trends observed in measured hydrographs. Furthermore, the average errors between observed and simulated water levels should be relatively small and unbiased. The monitoring well locations used for calibration of the groundwater flow model are shown in Figures 4-20 and 4-21.

Water level calibration focused primarily on water levels measured in the production wells in the western valley. The production well water levels were favored over the monitoring wells in the eastern Valley primarily because complete monthly water level records exist for production wells; District Well 2, Mutual Well 1, and Mutual Well 2. Water levels have been measured in the monitoring wells only sporadically. While District Well 2 has 77 data points, no monitoring well has more than eight data points. The complete water level records at the production wells allow comparison of water level trends. Water level trends cannot be discerned at many monitoring wells that have only two or three water level measurements.

A secondary consideration for favoring production well water levels over monitoring well water levels was the location of the wells. Future groundwater development will likely take place in the western portion of the basin. The model should therefore be focused on accurately simulating this western portion of the basin. This results in a focus on the production well water levels rather than the monitoring well water levels.

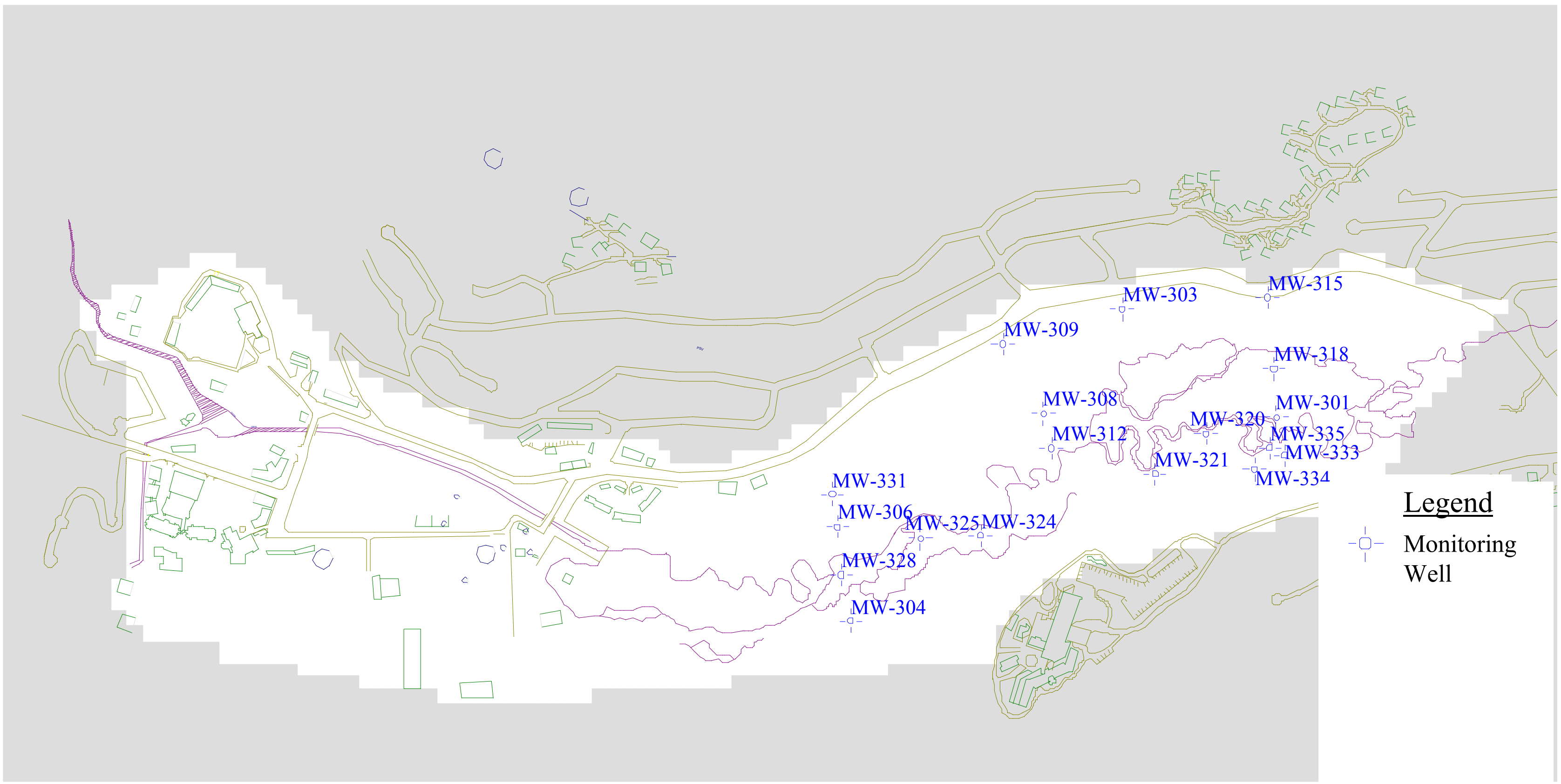

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Figure 4-20. Target Monitor Well Locations, Layer 2

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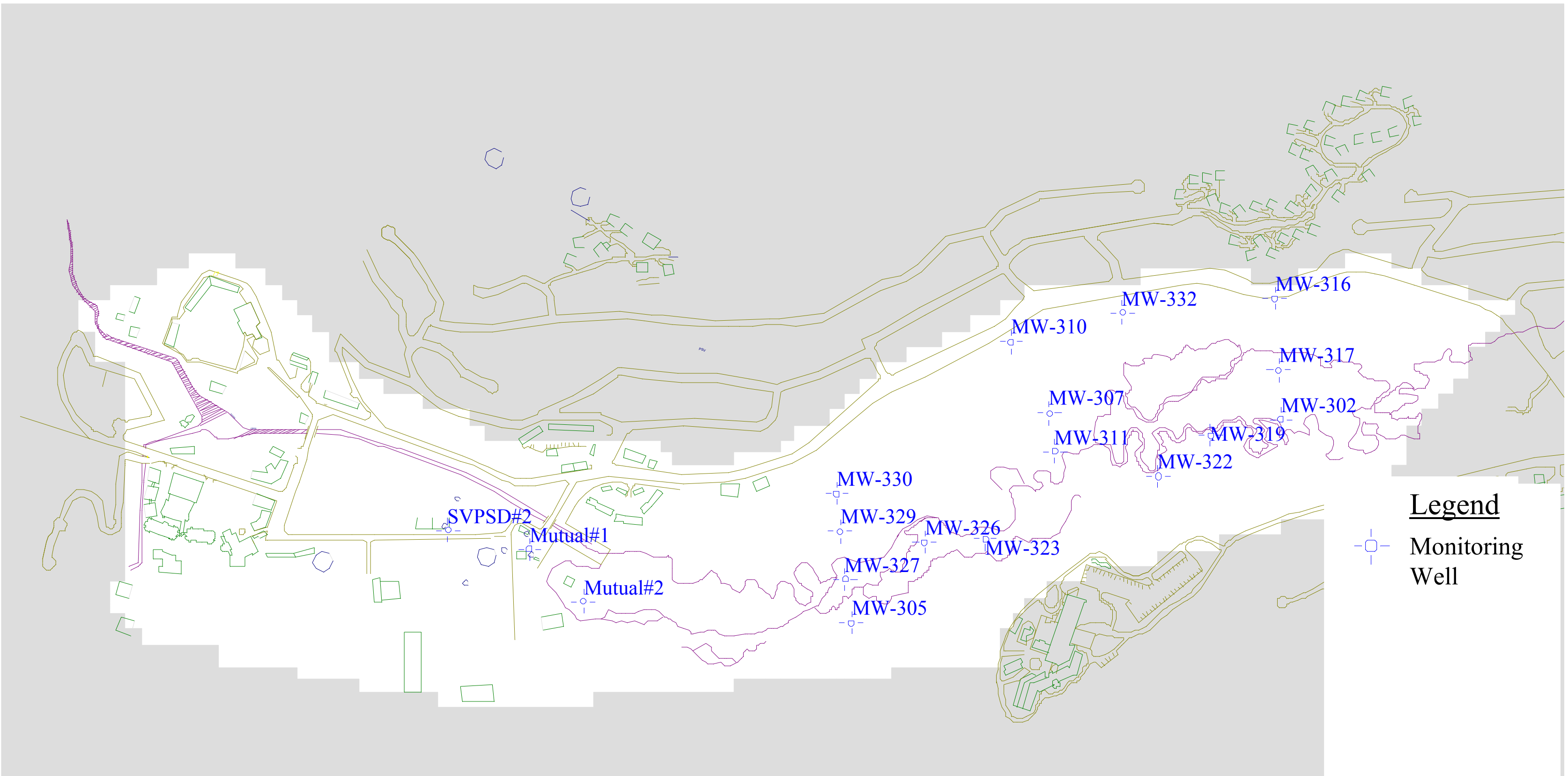

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Figure 4-21. Target Monitor Well Locations, Layer 3

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Hydrographs of observed and simulated water levels for the three production wells are included in Figures 4-22 through 4-24. Hydrographs of observed and simulated water levels for three example-monitoring wells are included in Figures 4-25 through 4-27. The hydrographs show that the simulated water levels are generally consistent with the magnitude and trends observed in production and monitoring well data.

Various graphical and statistical methods can be used to demonstrate the magnitude and potential bias of the calibration errors. Figure 4-28 shows all simulated water levels plotted against observed water levels for all stress periods in the calibration. Results from an unbiased model will scatter around a 45º line on this graph. If the model has a bias such as exaggerating or underestimating water level differences, the results will diverge from this 45º line. The line drawn on Figure 4-28 demonstrates that the results lie close to a 45º line, suggesting that the model results are not biased towards overestimating or underestimating average water level differences.

Figure 4-28 also includes various statistical measures of calibration accuracy. The four statistical measures used to evaluate calibration are the mean error (ME), the mean absolute error (MAE), the standard deviation of the errors (STD), and the root mean squared error (RMSE). The mean error is the average error between measured and simulated water levels for all data on Figure 4-28.

$$ME = \frac{1}{n}\sum_{i=1}^{n}\left(h_m - h_s\right)_i$$

Where h_m is the measured water level, h_s is the simulated water level, and n is the number of observations.

The mean absolute error is the average of the absolute differences between measured and simulated water levels.

The standard deviation of the errors is one measure of the spread of the errors around the 45º line in Figure 4-28. The population standard deviation is used for these calculations

$$STD = \sqrt{\frac{n\sum_{i=1}^{n}\left(h_m - h_s\right)_i^2 - \left(\sum_{i=1}^{n}\left(h_m - h_s\right)_i\right)^2}{n^2}}$$

The RMSE is similar to the standard deviation of the error. It also measures the spread of the errors around the 45º line in Figure 4-28, and is calculated as the square root of the average squared errors.

$$RMSE = \sqrt{\frac{1}{n}\sum_{i=1}^{n}\left(h_m - h_s\right)^2{}_i}$$

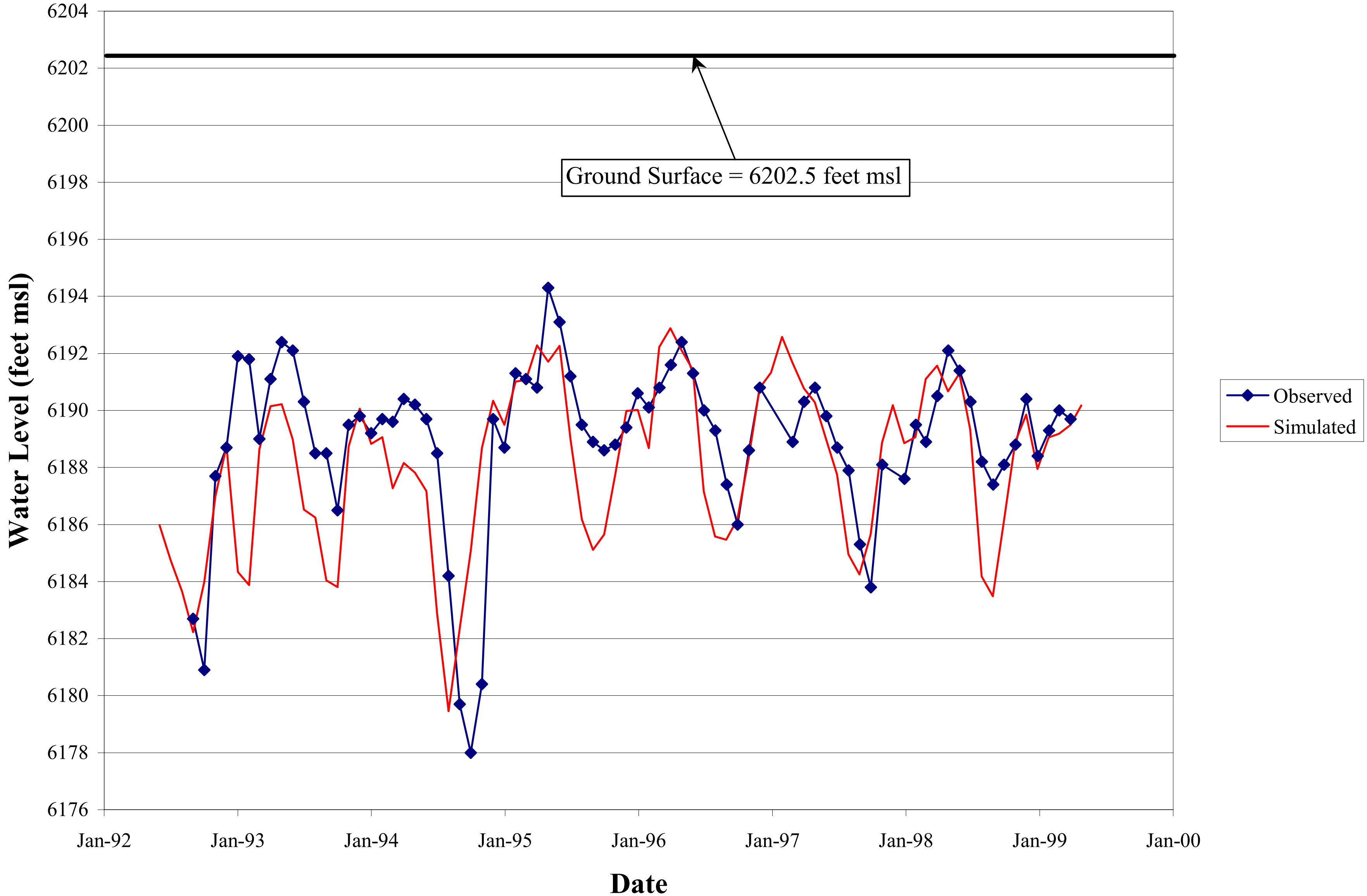

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Figure 4-22. Observed and Simulated Water Levels, Well SVPSD #2

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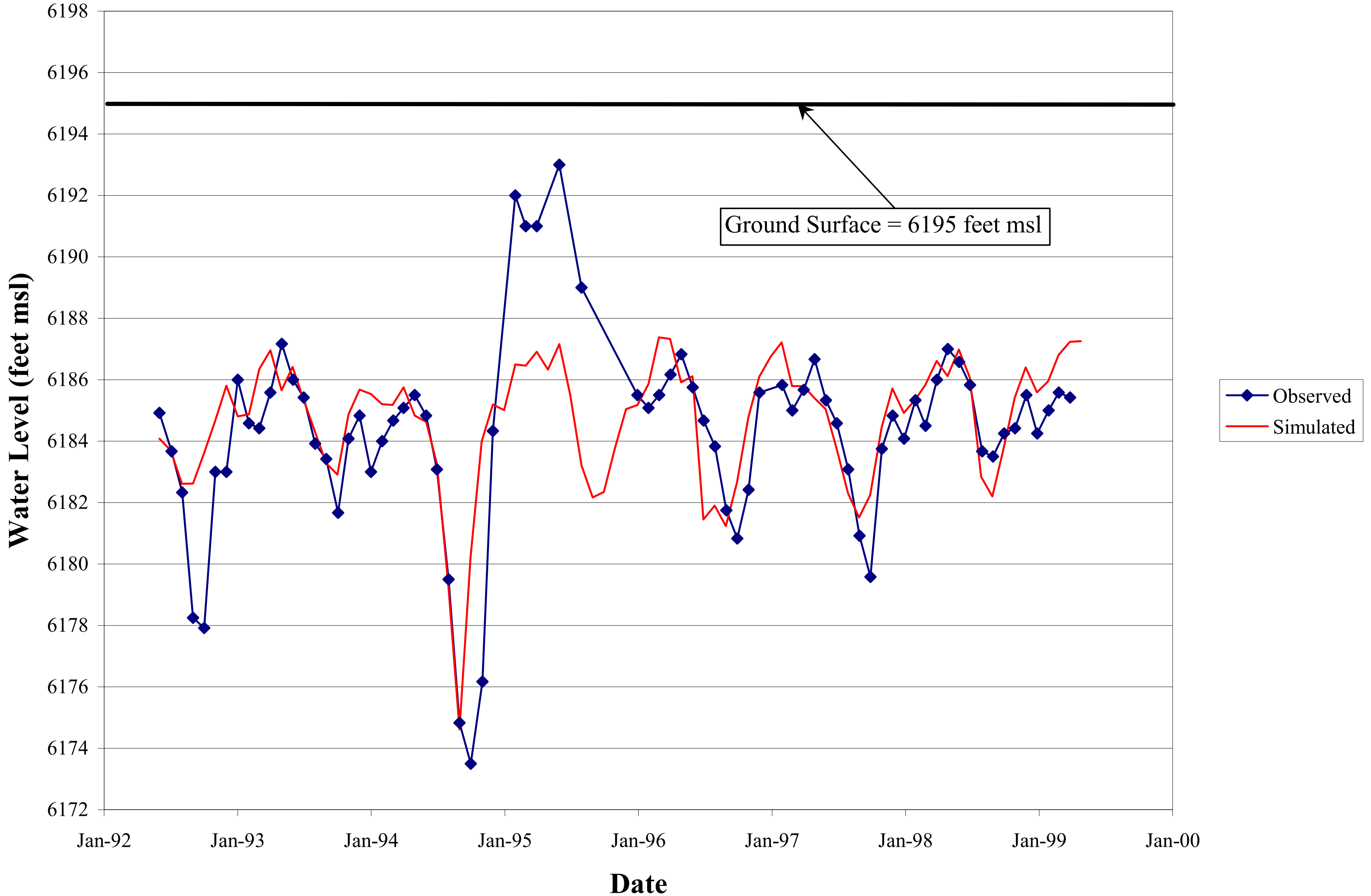

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Figure 4-23. Observed and Simulated Water Levels, Well SVMWC #1

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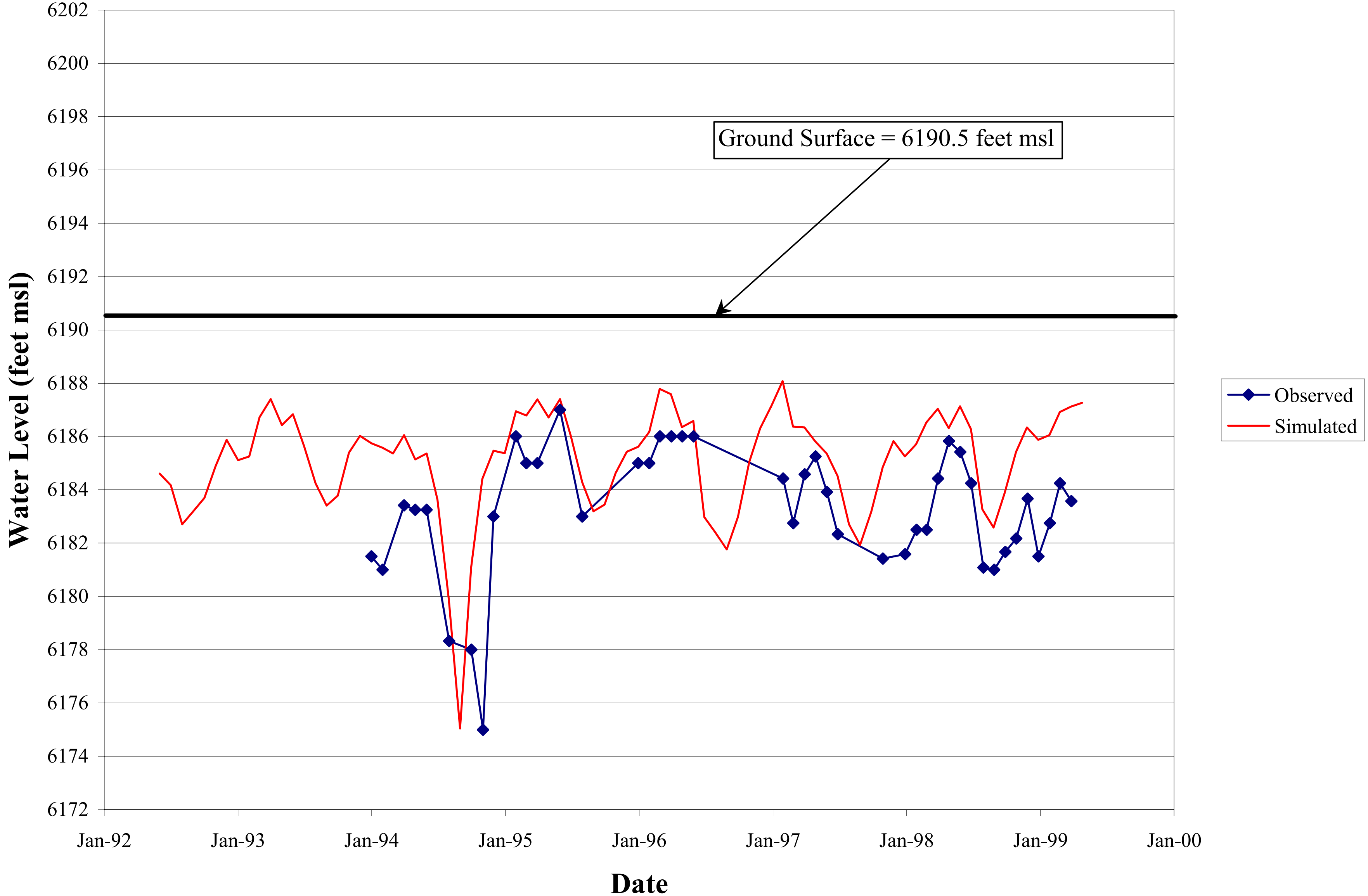

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Figure 4-24. Observed and Simulated Water Levels, Well SVMWC #2

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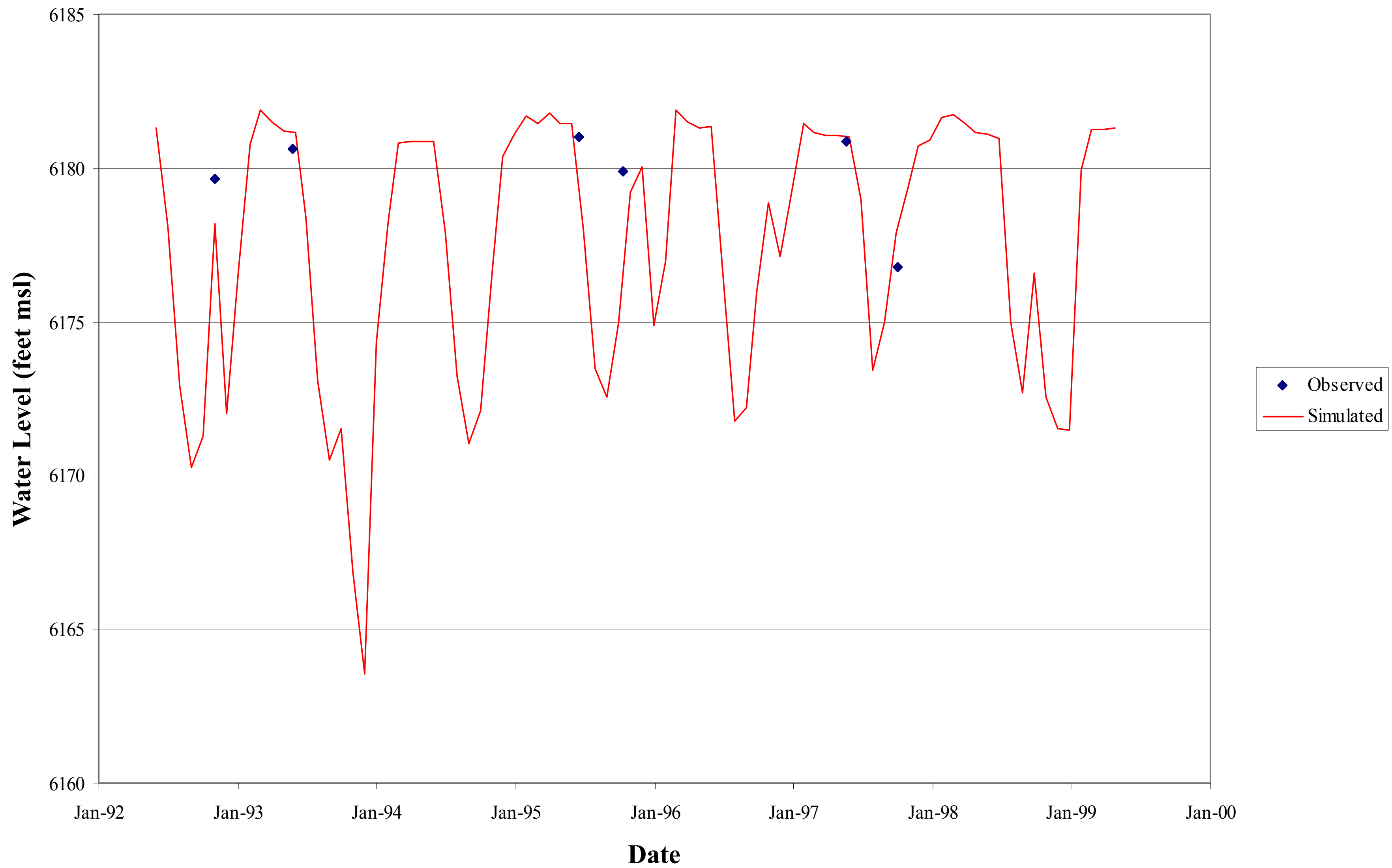

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Figure 4-25. Simulated Water Levels, Well MW-304

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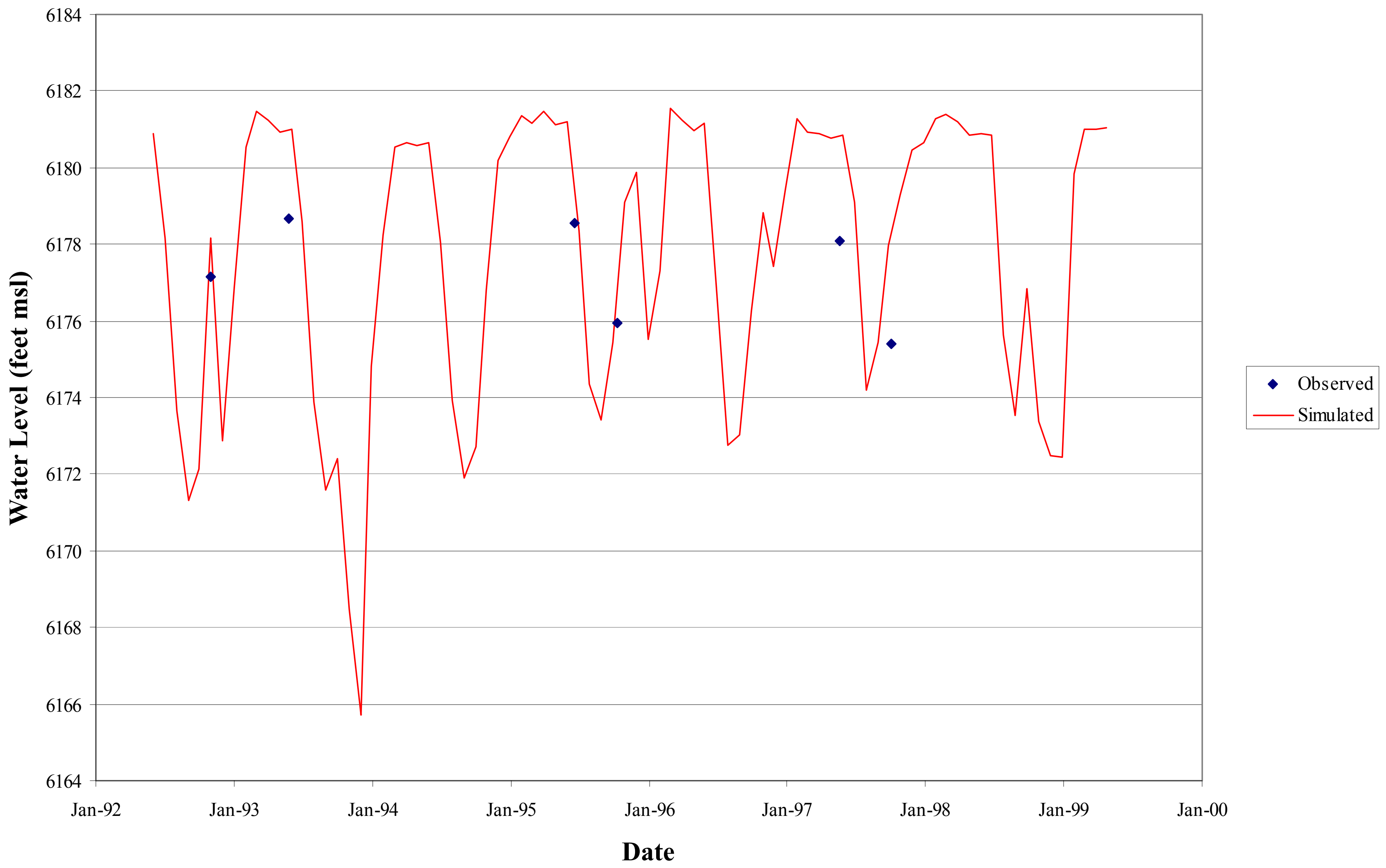

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Figure 4-26. Simulated Water Levels, Well MW-328

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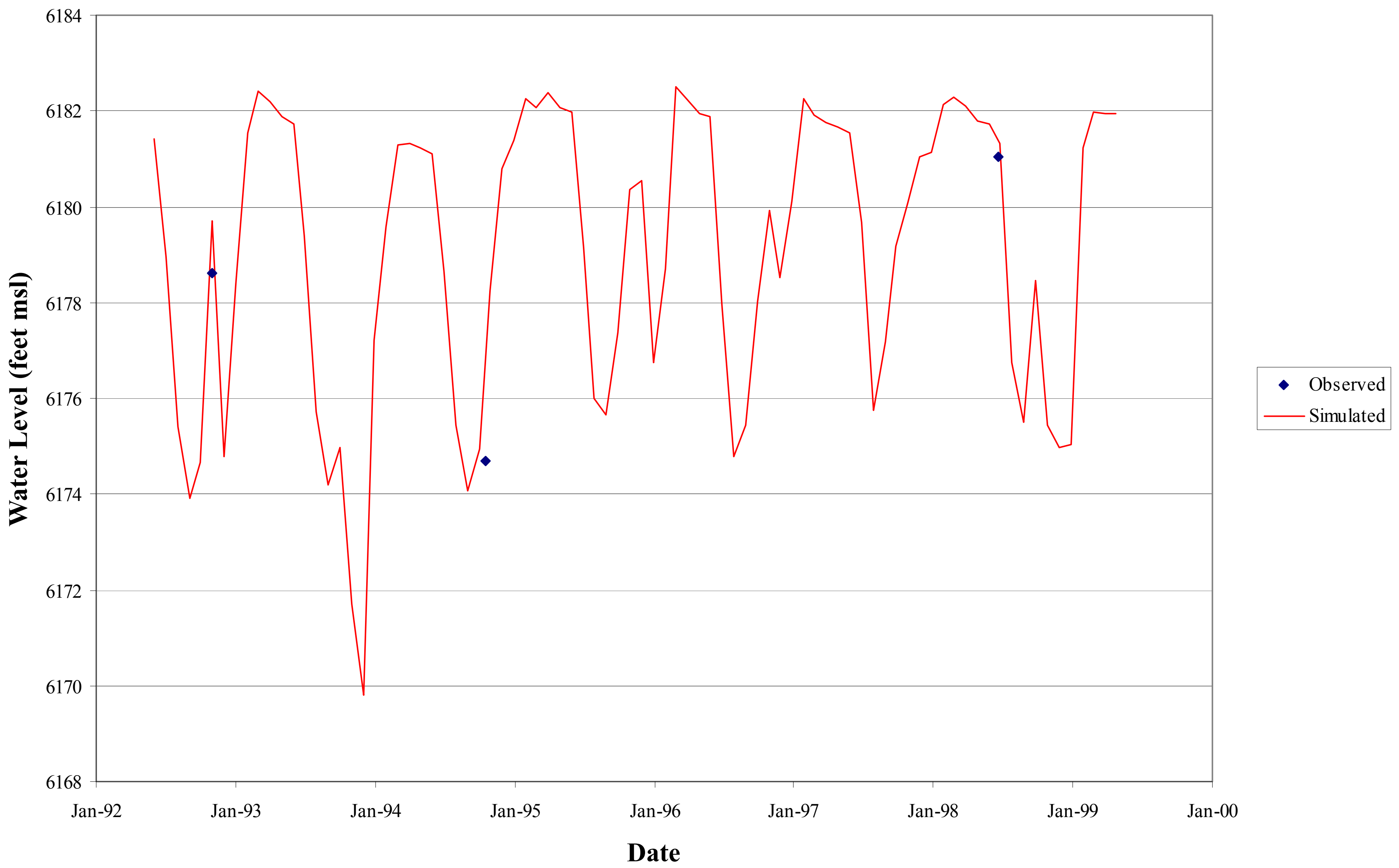

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Figure 4-27. Simulated Water Levels, Well MW-329

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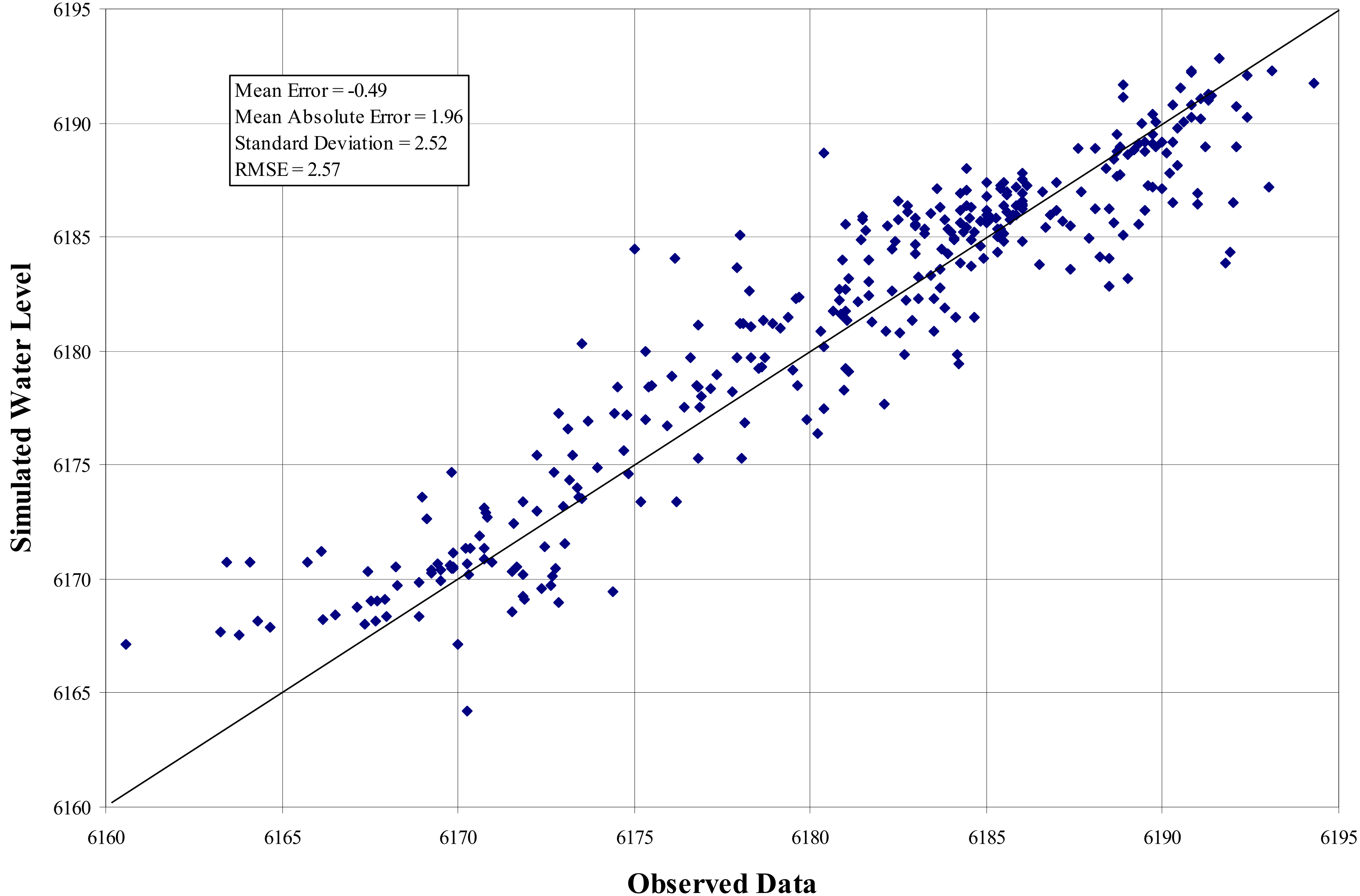

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Figure 4-28. Simulated Versus Observed Water Levels

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As a measure of successful model calibration, Anderson and Woessner (1992) state that the ratio of the spread of the errors to the total head range in the system should be small to ensure that the errors are only a small part of the overall model response. As a general rule, the standard deviation of errors should be less than 10 percent of the total head range in the model. The standard deviation of 2.5 is about 7.5 percent of the total head range of 33 feet. A second general rule that is occasionally used is that the mean error should be less than 5 percent of the total head range in the model. The mean error of 0.49 is about 1.5 percent of the total head range. Therefore, on average, the model errors are within an acceptable range.

A second graph used to evaluate bias in model results is shown on Figure 4-29. This figure is a graph of observed water levels versus model residual (model error for any given water level measurement). Results from a non-biased simulation will appear as a cloud of data points clustered around the zero model residual line. Results that do not cluster around the zero residual line show potential model bias. Results that display a trend instead of a random cloud of points may suggest additional model bias. The results plotted on Figure 4-29 show that the calibrated model results are generally unbiased, although the errors are a bit lower than the zero model residual line. This suggests that model results are a bit higher than the observed data. This is largely a result of comparing simulated and observed water level data from Mutual Well 2, and to a lesser degree Mutual Well 1.

Squaw Creek Stream Flows

Figure 4-30 compares the modeled flow in Squaw Creek with the estimated flow in Squaw Creek. The simulated flows represent the net simulated flow of Squaw Creek as it leaves the basin. The creek flows are estimated from gauged flow data of the Truckee River, along with statistical correlations of flows in nearby creeks. The estimated data represents our best estimate of the net monthly flow of Squaw Creek out of the basin.

Figure 4-30 shows that the simulated creek flows generally mimic the estimated creek flows. One of the most important differences between the simulated and estimated creek flows is that the estimated creek flows always assume Squaw Creek is a gaining creek. The simulated creek flows allow Squaw Creek to be a gaining or losing stream, and therefore the simulated flows are somewhat lower than the estimated flows.

Subsurface Flow Through the Terminal Moraine

Figure 4-31 shows the simulated monthly subsurface flow rates through the terminal moraine. The average annual subsurface flow rate through the terminal moraine is approximately 83 af per year. This is less than half the 220 af per year estimated by Sheahan (1990). These subsurface flow rates cannot be compared with measured data, and therefore cannot be used as definitive proof of adequate calibration. The subsurface flow rates, however, are reasonably small, and compare favorably with the estimates by Sheahan (1990).

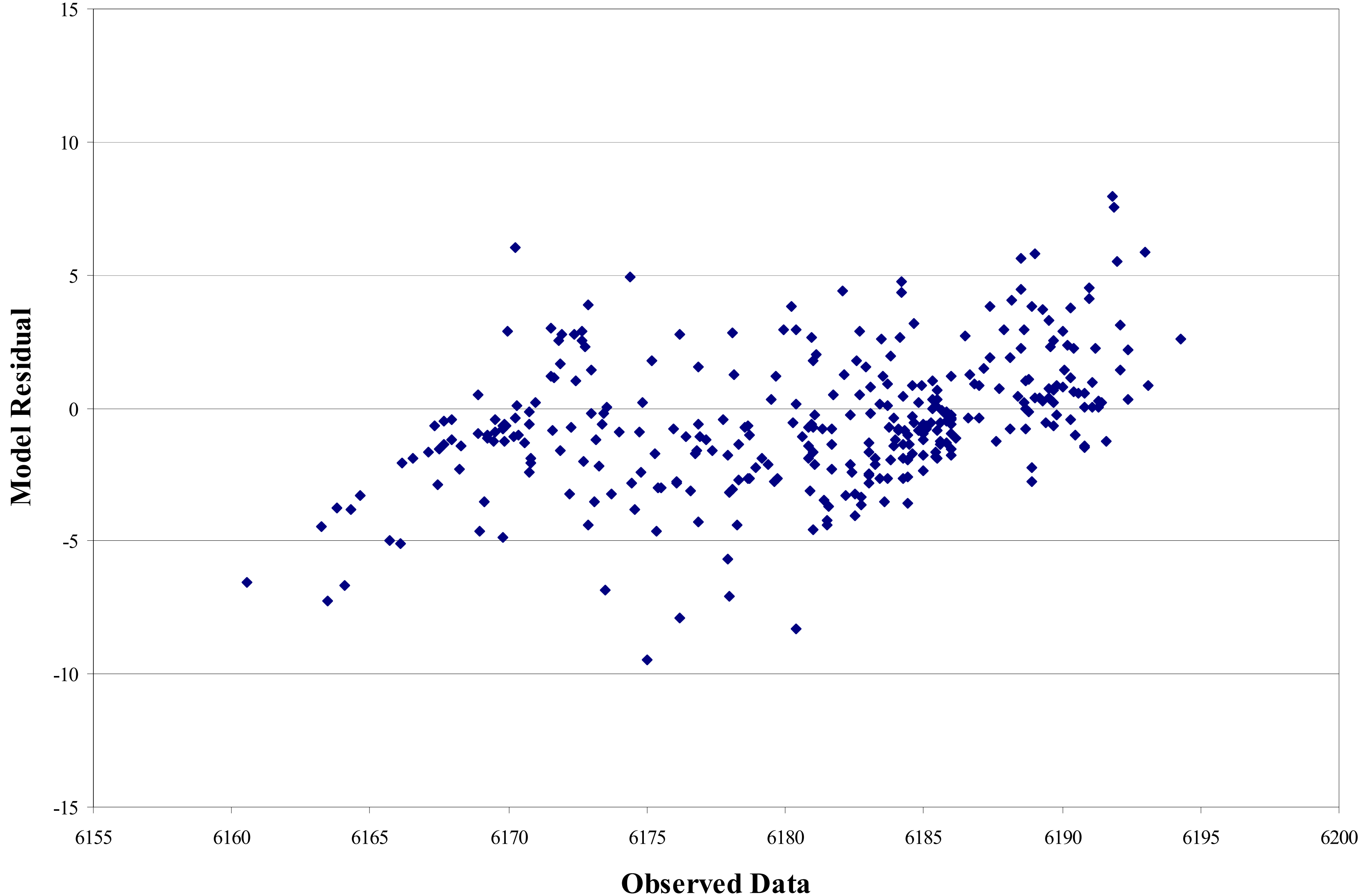

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Figure 4-29. Observed Water Levels Versus Residuals

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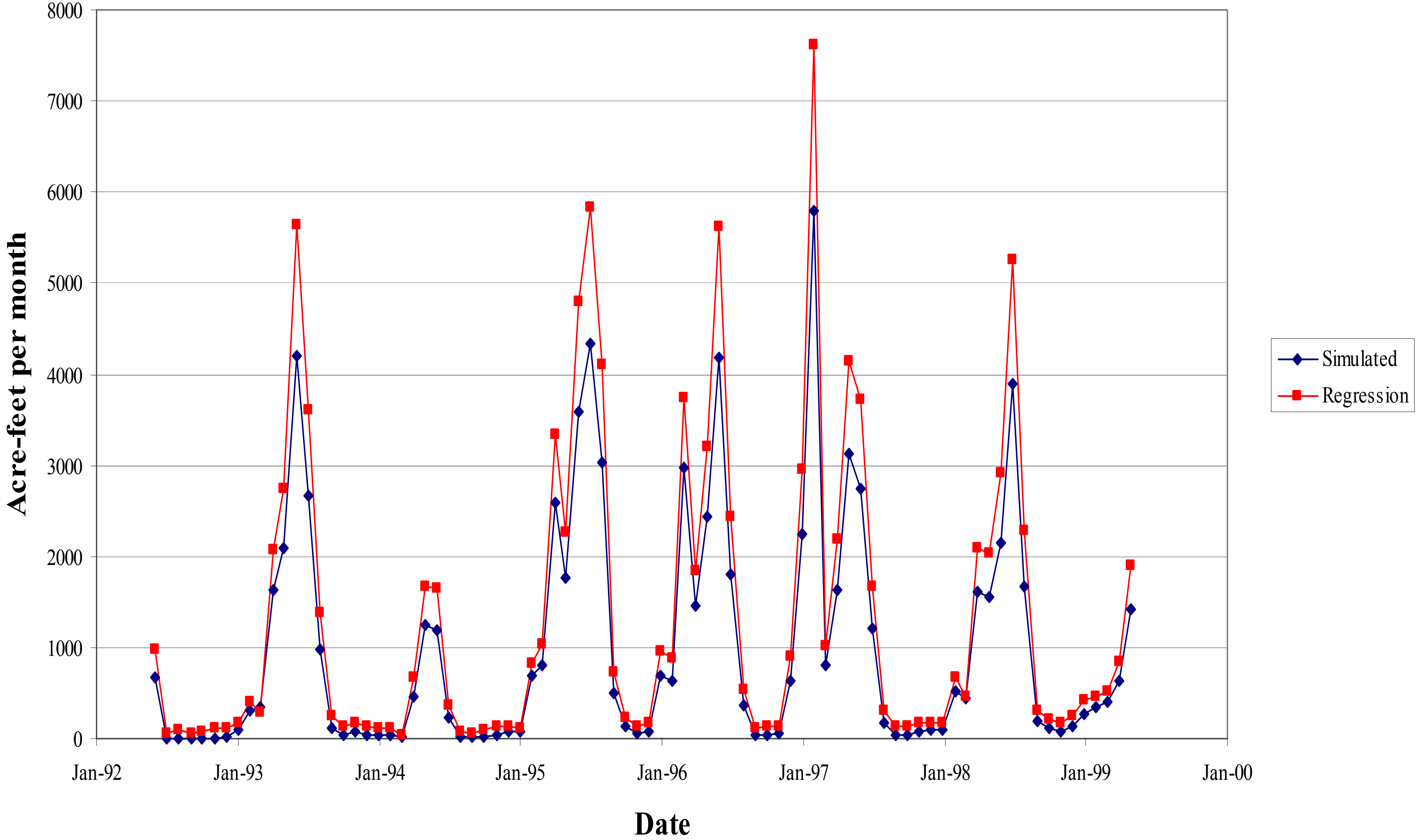

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Figure 4-30. Estimated and Simulated Stream Flow

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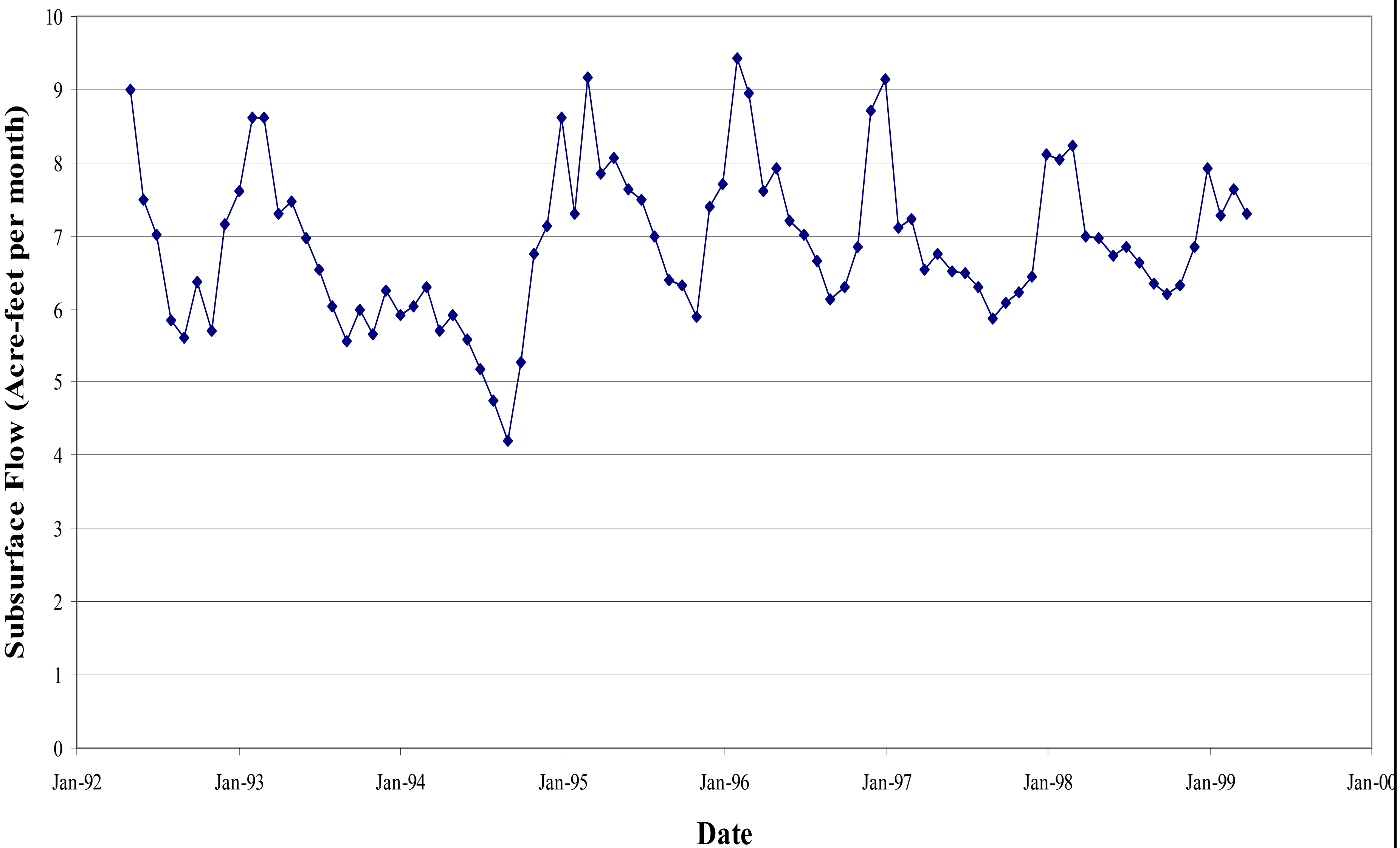

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Figure 4-31. Simulated Flow through the Terminal Moraine

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Sensitivity Analysis

A sensitivity analysis was performed on the major flow model input parameters. The input parameters analyzed in the sensitivity analysis included:

1. Horizontal hydraulic conductivity
2. Vertical hydraulic conductivity
3. Storativity

Results of these sensitivities are shown on Figures 4-32 through 4-34. The three charts in Figures 4-32 through 4-34 show the effect of changing a model parameter value on the standard deviation of the calibration residuals. The X-axis on all three charts is the multiplier on the parameter value. A multiplier of one represents the parameter value used in the groundwater model. The Y-axis on all three charts is the standard deviation statistic resulting from changing the model parameters. The values on the Y-axes all cover the same range (0 to 14) so that the results on all three charts can be directly compared.

Figures 4-32 through 4-34 show that the most sensitive parameters are the horizontal conductivities of Zones 5 and 6. These two zones are likely the most sensitive because the District and Mutual production wells are located in these zones. The District and Mutual wells have the most monitoring data, and therefore have the largest impact on the calibration statistics.

Figure 4-34 shows that changes to the storage coefficients have virtually no influence on the calibration statistics.

CONCLUSIONS

A groundwater flow model of Squaw Valley has been constructed and calibrated to available data. The model incorporates all known groundwater recharge and discharge mechanisms, as well as all available hydrogeologic data from the basin. The model successfully simulates water level fluctuations in both production wells and monitoring wells throughout the basin, and reasonably simulates flows in Squaw Creek. The combination of a solid technical model base and successful calibration has resulted in a valuable tool for future groundwater management studies.

The groundwater model is the best tool available for estimating effects of various pumping and recharge scenarios, and should be used for planning future groundwater management. Pumping rates from existing wells, placement of future wells, and effects of pumping on stream flows can all be studied with the existing model. The model will improve any future planning decisions, and can identify optimal groundwater management strategies.

As with all groundwater models, additional data will help validate the model, and direct modifications to uncertain model parameters. Data that may be particularly helpful includes measured stream flows entering and leaving Squaw Valley, and additional water level data from the western portion of the basin. Squaw Creek flow data will corroborate estimates of the amount of groundwater lost or gained by stream interaction. Additional stream data will furthermore allow accurate calibration of the impact on streamflow from groundwater pumping.

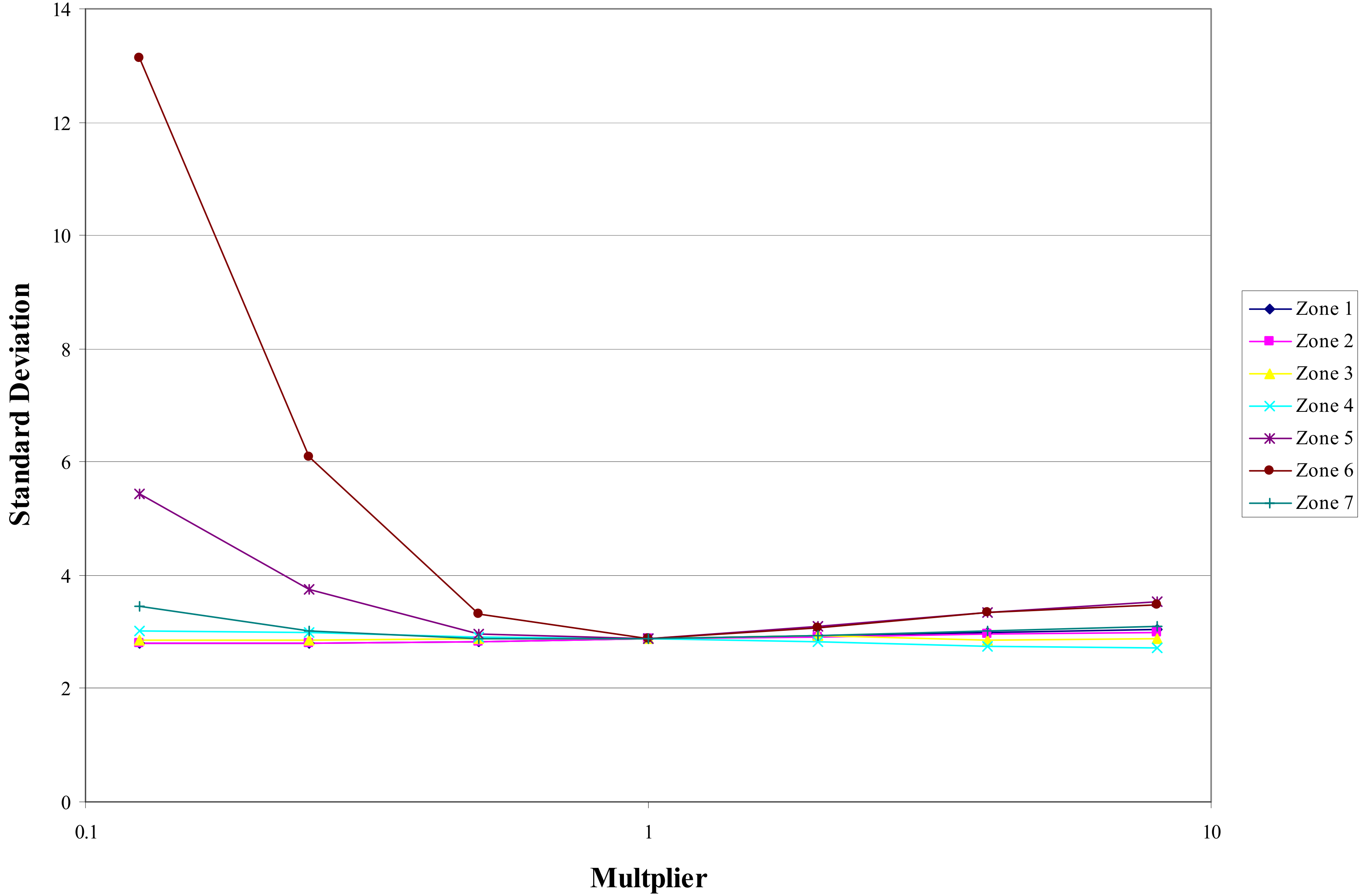

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Figure 4-32. Sensitivity of Horizontal Conductivities

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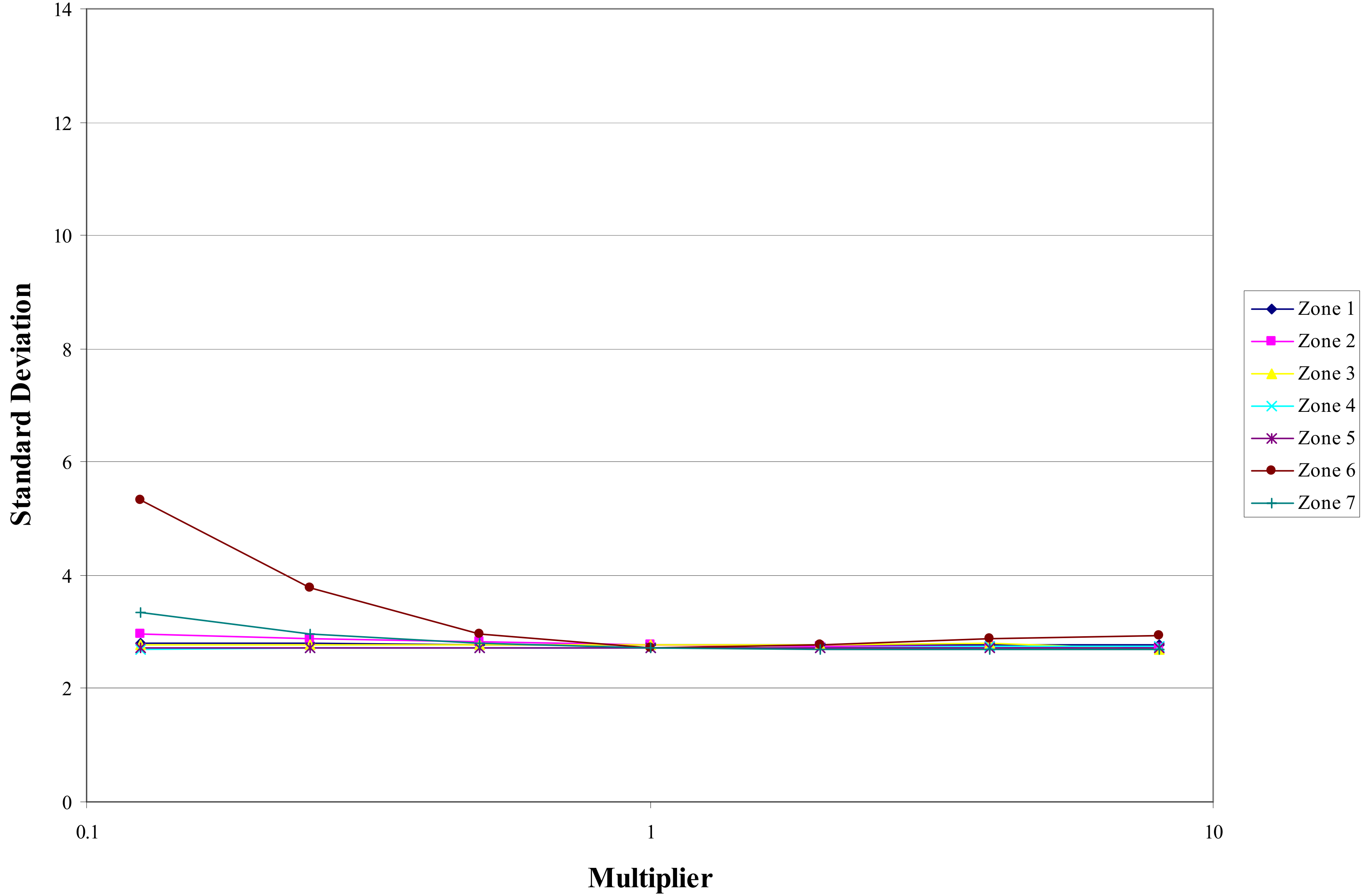

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Figure 4-33. Sensitivity of Vertical Conductivities

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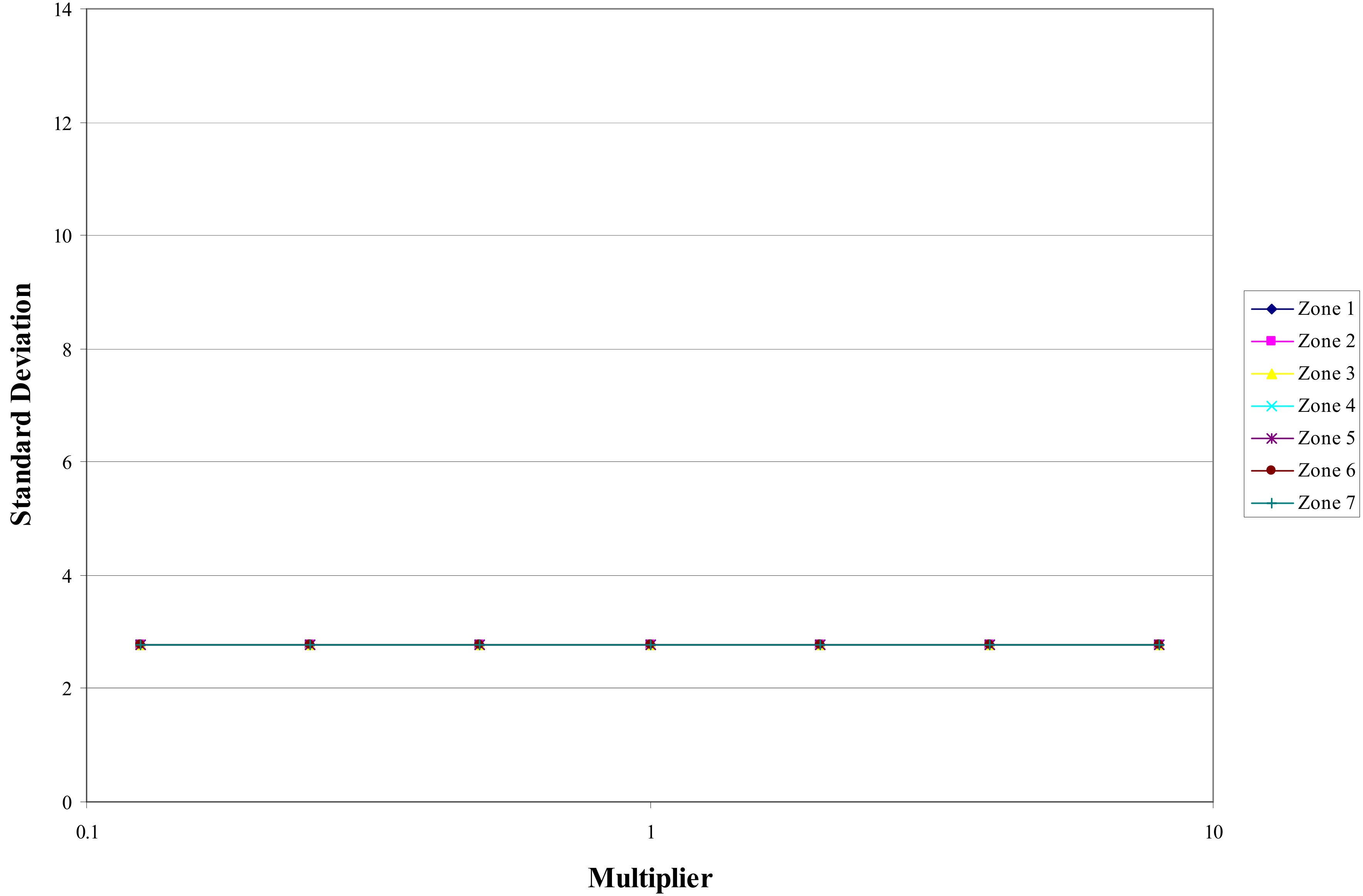


SVPSD
Groundwater
Model
Development

Figure 4-34. Sensitivity of Storage Coefficients

derrik williams

Water level and hydrologic parameter data from the western end of the Squaw Valley Basin will assist in future water management planning. The western portion of the basin has generally better producing wells, and the groundwater in the western basin generally does not require treatment before it is served. Additional data on the production capability of the western basin, along with information about the impact of Squaw Creek on water levels in the western basin, is crucial to future water planning efforts.

As with all groundwater models, the results are only as accurate as the data on which the model is based. Assumptions about the basin dynamics are based on the best available data at the time of model development. As new data becomes available, new interpretations of the basin hydrogeology may require re-structuring of parts of the model.

2003 GROUNDWATER MODEL UPDATE AND CALIBRATION

Data Incorporated into the Updated Model

Both numerical data and observational data were incorporated into the updated groundwater model. The new data included the following:

A map of the historical location of Squaw Creek. Mr. Carl Gustafson, a licensed Civil Engineer and Surveyor from Squaw Valley, California provided this hand-drawn map. The map shows the location of Squaw Creek in the western portion of the basin before the parking lot was built. Data from this map was used to delineate a high conductivity zone corresponding to the streambed deposits associated with the historical creek location.

Oblique photograph of Squaw Valley. Mr. Paul Arthur of Squaw Valley, California provided this photograph. The photograph shows the location of Squaw Creek prior to the development of Squaw Valley. This photograph was used to confirm the creek location information included on the map discussed above.

Detailed Mapping of Squaw Valley East of the Parking Lot. West Yost & Associates prepared a detailed map of ground surface elevations in the meadow and golf course area of Squaw Valley. Included in the mapping were a number of surveyed streambed elevations. These data were used to refine both the ground surface elevations and the creek bottom elevations in the model.

Groundwater levels observed during construction dewatering. Dewatering was necessary during construction of Intrawest's underground parking structures in 2002. The parking structure was included as part of the Phase II construction activities of Intrawest's new Squaw Valley Village. During dewatering, recorded drawdowns in observation wells were less than predicted by the previous model. The dewatering data, along with a preliminary model analysis of the data, are included in the *Intrawest Dewatering Impact Study Memorandum* (Williams, 2002).

Plumpjack well information. The Plumpjack well is located in the parking lot west of the Plumpjack development. The construction details of the Plumpjack well are unknown, however incidental information suggests that the basin sediments may be up to 130 feet thick at the Plumpjack well location.

Observations of the small reservoir in Olympic Lady Canyon. A small reservoir located in the Olympic Lady Canyon retains water throughout much of the summer. No data is available on potential leakage from this reservoir. The bottom of the reservoir appears to be unlined. It is reasonable to assume that some amount of water leaks from the reservoir throughout the year, providing a small source of water for the Squaw Valley Basin.

Groundwater level data from the Olympic House Loading Dock Underground Storage Tank (UST) site. Water level data from the Olympic House Loading Dock site appear to be referenced to Mean Sea Level (MSL). This allows a direct comparison of this water level data with groundwater model results. Groundwater level data for this study were derived from the *Groundwater Monitoring Report – 2nd Quarter 2002, Olympic House Loading Dock* (McGinley & Associates, 2002). Groundwater level data from four monitoring wells are plotted on Figure 4-35. This figure shows that most of the groundwater level data lie between 6190 and 6196 feet msl.

Groundwater Model Modifications

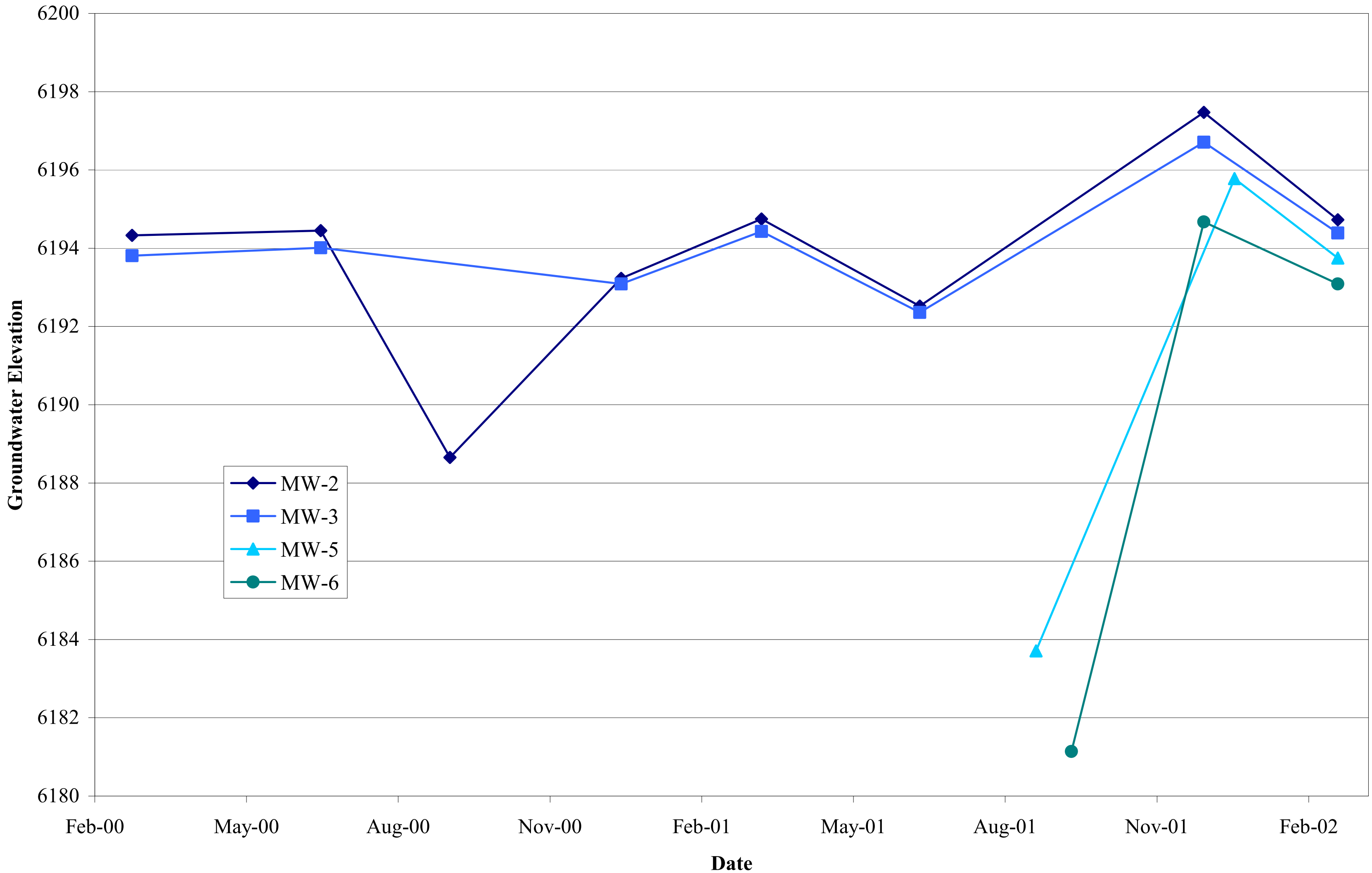
Based on the interpretations and analyses of the data discussed above, a number of modifications were incorporated into the existing groundwater model. The model modifications include the following.

Honored the historical location of Squaw Creek. A high hydraulic conductivity band was added to the model, representing the gravelly streambed deposits along the historical location of Squaw Creek. The high conductivity band was added to both of the top two model layers. The location of this band is shown on Figure 4-36. Wells that lie in these stream deposits are generally in closer hydraulic contact with the existing Squaw Creek than other wells.

Deepened the groundwater model around the Plumpjack well. Based on unverified information, the groundwater model was deepened to 130 feet around the Plumpjack well. The model was previously approximately 60 feet deep around the Plumpjack well, based on the depth of existing water supply wells and a reasonable interpretation of simple basin geometry.

Refined the ground surface and creek bottom elevations. The simulated ground elevation was modified based on the mapping provided by West Yost & Associates. Additionally, the thalweg of Squaw Creek in the meadow was modified based on the survey data. Generally, the creek at the western end of the meadow was lowered to honor the survey data.

Added a water source at the foot of the Olympic Lady Canyon. This water source represents leakage from the small reservoir in Olympic Lady Canyon. The water source was added as a general head boundary in the top layer of the groundwater model. Because there are no leakage measurements or estimates from the reservoir, an estimate of the amount of leakage was made during calibration. As discussed in the calibration section below, the amount of leakage from the Olympic Lady Reservoir is highly uncertain.



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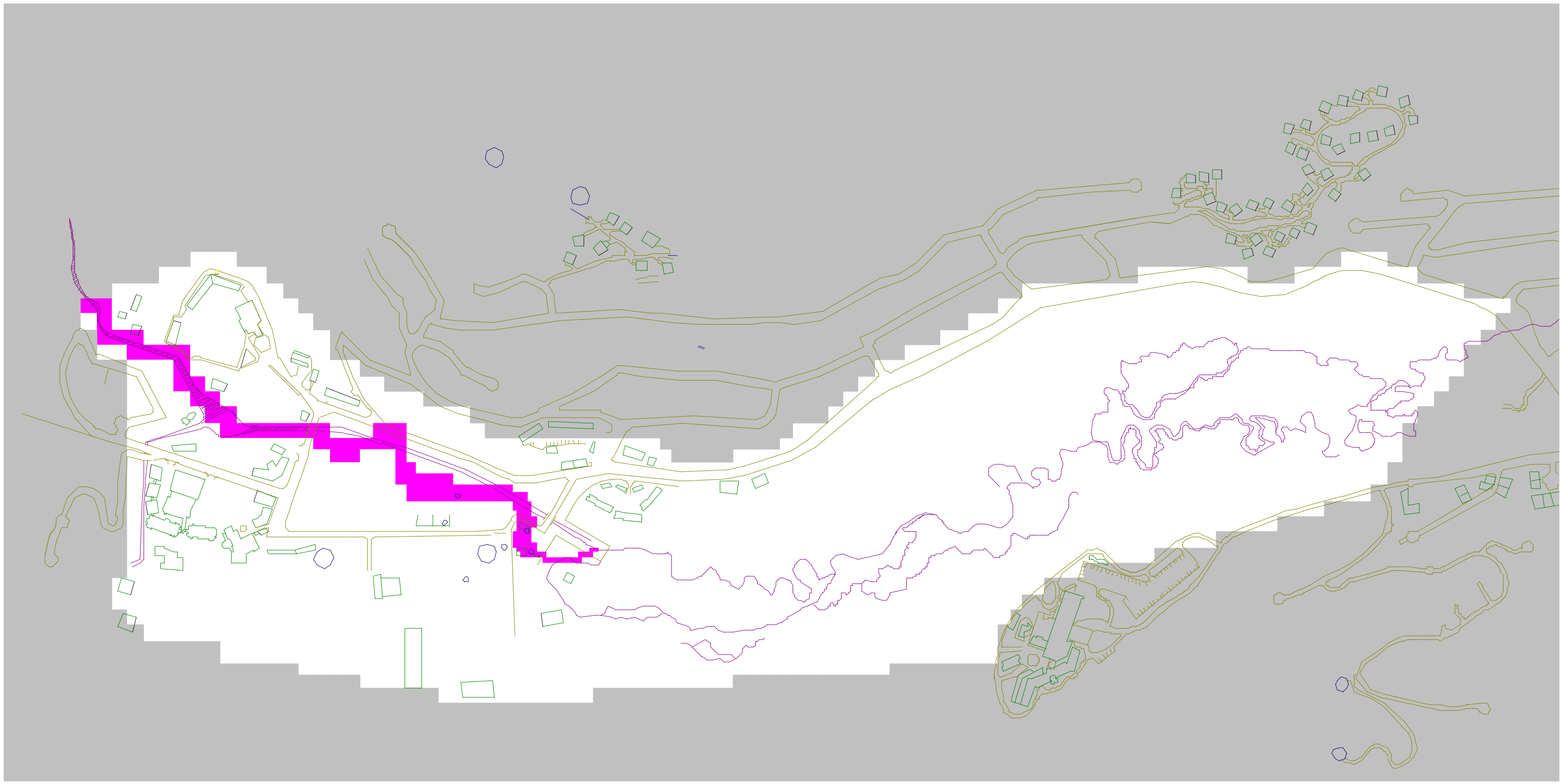
Observed Water Levels, Olympic House Loading Dock Site

Figure 4-35



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Location of Simulated Streambed Deposits

Figure 4-36

Groundwater Model Calibration

The groundwater model was calibrated to groundwater elevations measured between May 1992 and April 1999. This is the same period used for the original groundwater model calibration. The same period was used so that the current calibration statistics could be compared to the previous calibration statistics.

Calibration generally consisted of adjusting model parameters to fit the observed data. Very little effort was made to refine the distribution of model parameters. The obvious exception is the addition of the high hydraulic conductivity band that represents the historical stream deposits.

As part of the calibration, the leakance from Squaw Creek was redefined. The previous model divided Squaw Creek into three sections: a north fork, a south fork, and the entire reach below the confluence of the two forks. Squaw Creek, however, flows more rapidly west of the meadow than within the meadow. Streambed deposits are therefore likely coarser west of the meadow than within the meadow. This difference in streambed deposits likely affects the leakance from Squaw Creek. The reach of Squaw Creek below the confluence of the two forks was therefore divided into two sections: one section between the confluence and the meadow, and one section within the meadow. A separate stream leakance was assigned to each section.

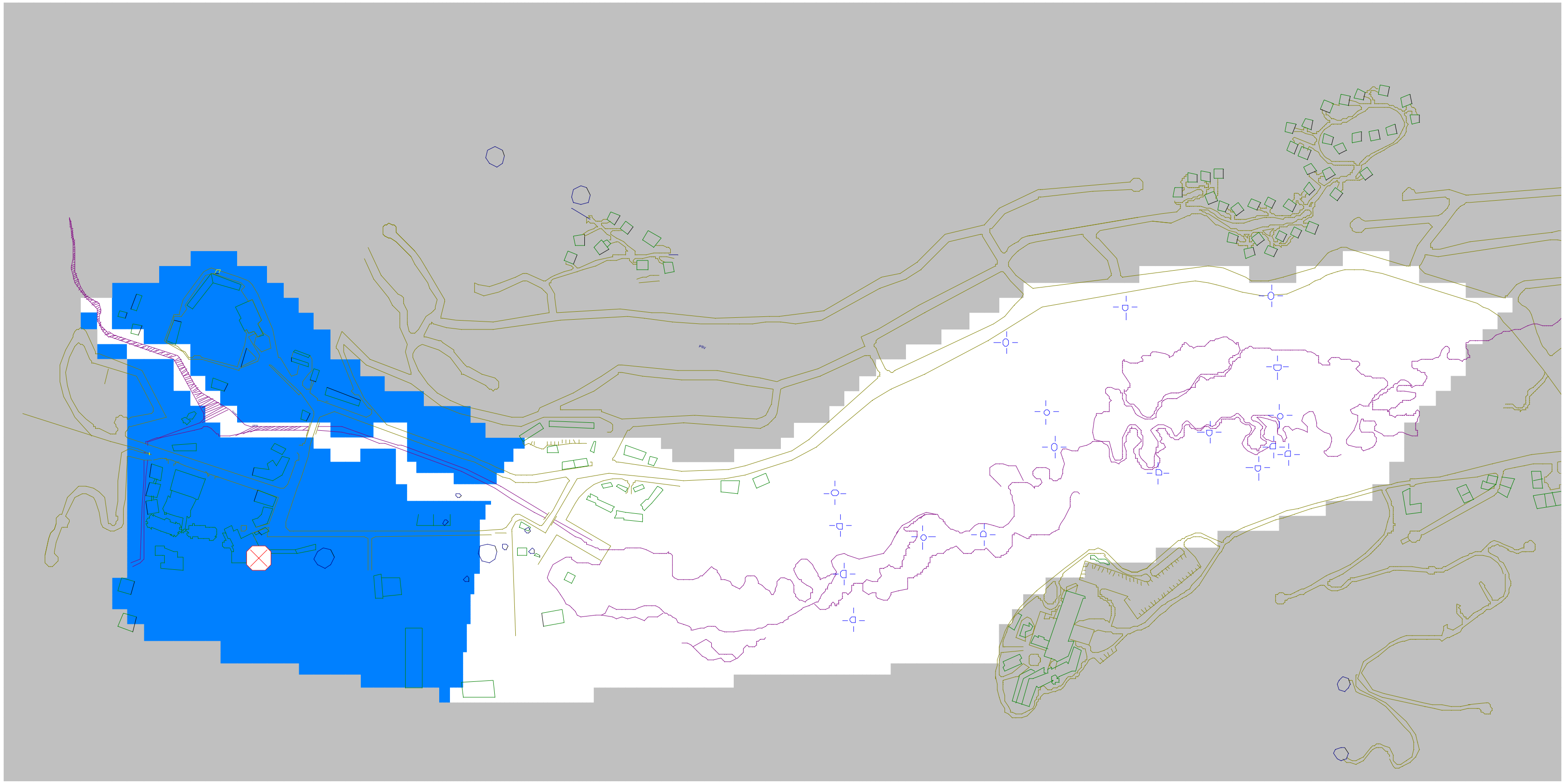
Calibrated Parameters

The model parameters that were adjusted during calibration included the following:

- Horizontal and vertical hydraulic conductivities of the western end of Squaw Valley (Figure 4-37). This parameter has an important impact on the simulated water levels in Squaw Valley Public Service District Well#2. The horizontal hydraulic conductivity was set to 60 feet per day in the original model, and was increased to 120 feet per day in the current model. The vertical hydraulic conductivity was set to 0.6 feet per day in the original model, and was increased to 3.6 feet per day in the current model.

- Horizontal and vertical hydraulic conductivities of the low conductivity band in the center of Squaw Valley (Figure 4-38). This low conductivity zone plays an important role in the water elevation drop between Squaw Valley Public Service District Well#2 and Squaw Valley Mutual Water Company Well#1. The horizontal hydraulic conductivity was set to 2 feet per day in the original model, and decreased to 0.15 feet per day in the current model. The vertical hydraulic conductivity was set to 0.007 feet per day in the original model, and was increased to 0.01 feet per day in the current model.

- Stream leakance. The stream leakance is a measure of the ability with which groundwater can flow into and out of Squaw Creek. The two stream leakances below the confluence of the two forks of Squaw Creek were modified during the calibration. As mentioned above, the model now assigns two separate leakances to the this stretch of Squaw Creek, one leakance for the stream west of the meadow and one leakance for the stream within the meadow. The hydraulic conductivity of the



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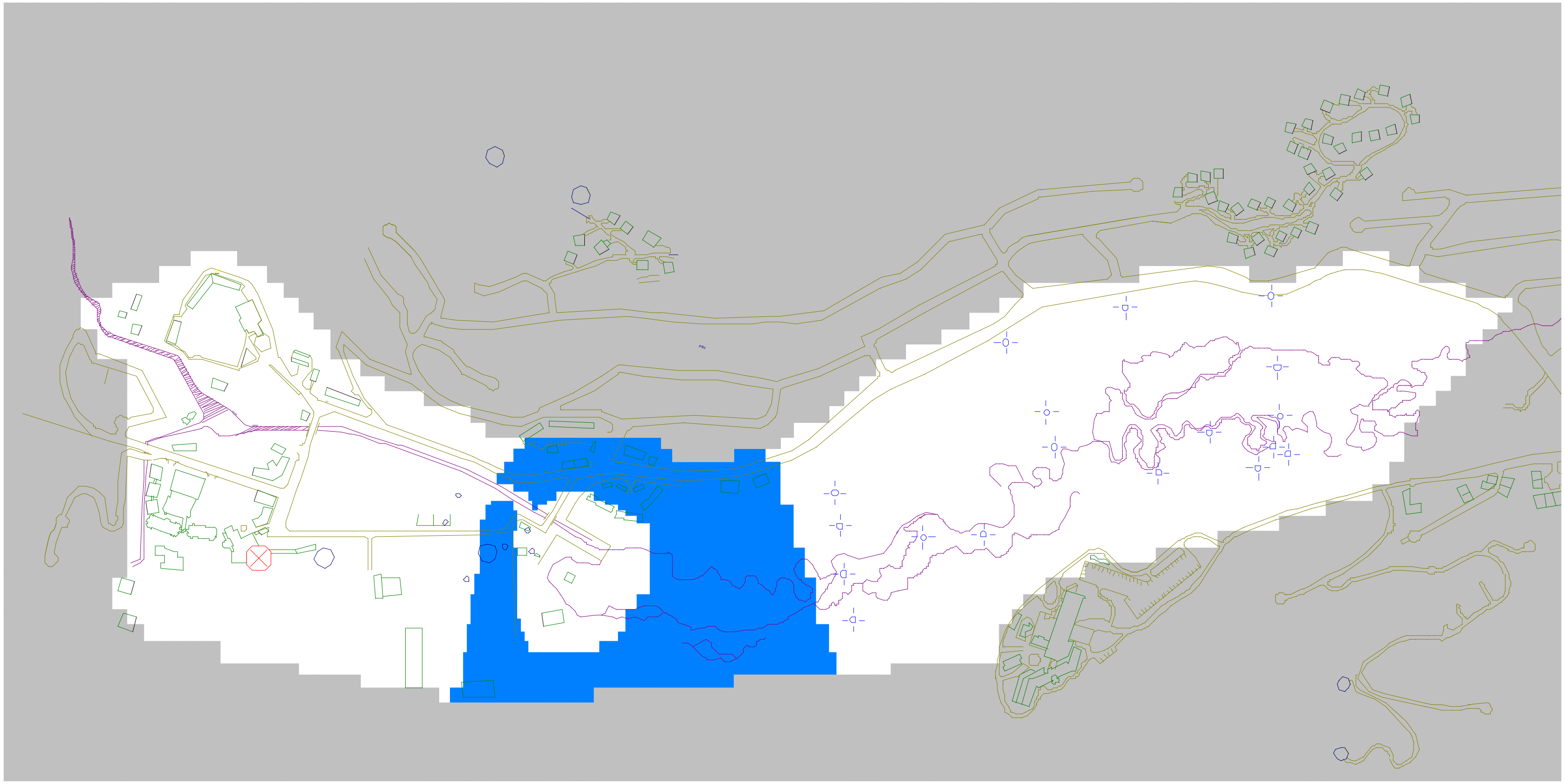
Hydraulic Conductivity Zone, Western End of Squaw Valley

Figure

4-37



Low Hydraulic Conductivity Zone, Middle of Squaw Valley

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Figure 4-38

streambed west of the meadow is set at 11.7 feet per day in the current model. The hydraulic conductivity of the streambed within the meadow is set at 0.24 feet per day in the current model.

- Specific yield of the western end of Squaw Valley (Figure 4-37). This parameter, in conjunction with the horizontal and vertical conductivities of this zone, play an important role in the simulated water levels in Squaw Valley Public Service District Well#2. The specific yield was set to 0.1 in the original model, and was increased to 0.2 in the current model.

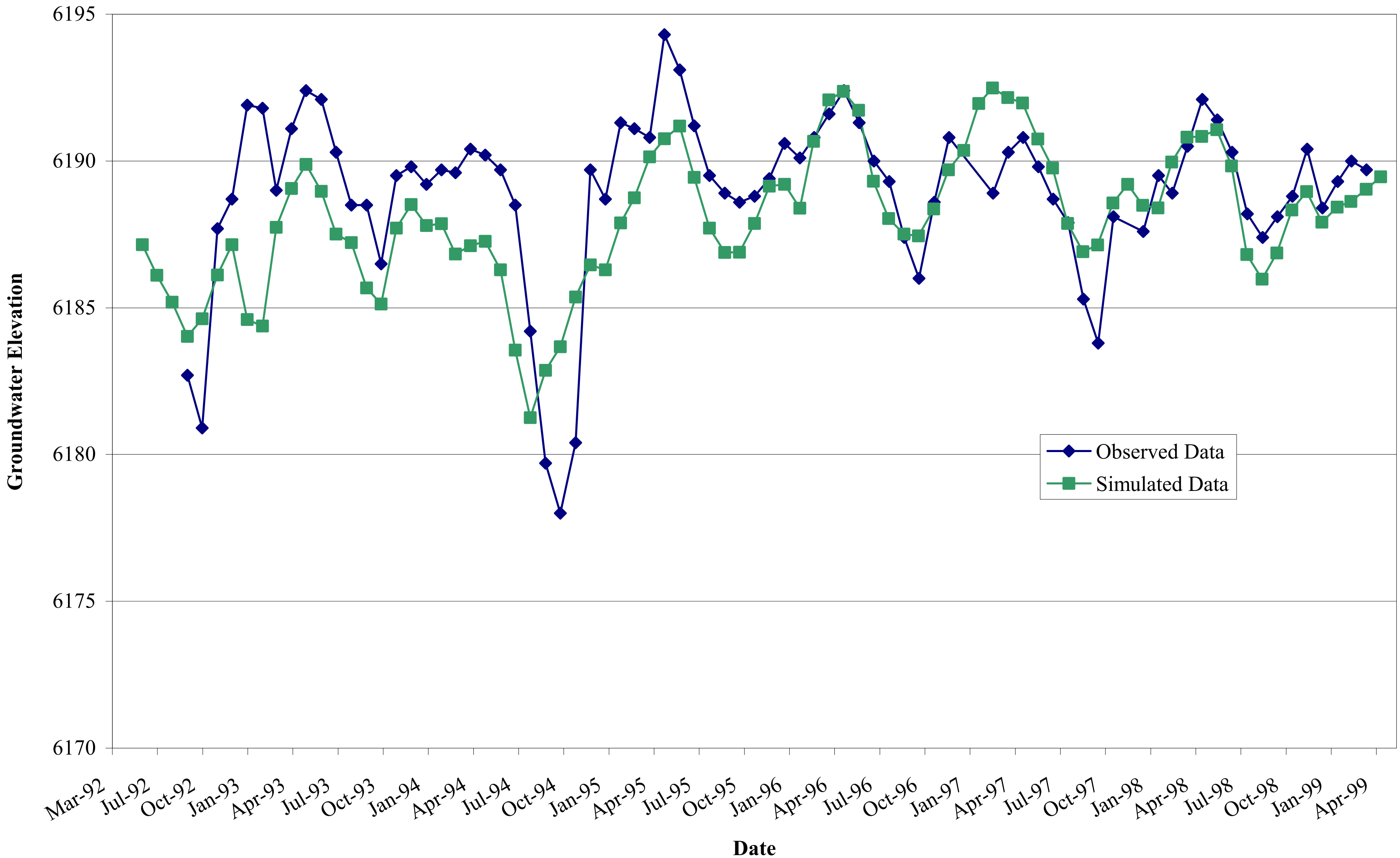
Calibration Results

Table 4-8 compares the calibration results from the initial model with the calibration results from the updated model. The calibration statistics are slightly better in the initial model, although the updated model is still considered well calibrated.

Table 4-8. Calibration Results

Parameter	Initial Model	Updated Model
Mean Error	-0.49	-0.40
Mean Absolute Error	1.96	2.13
Standard Deviation	2.52	2.75
RMSE	2.57	2.78

Figures 4-39, 4-40, and 4-41 show the measured and simulated water levels for District Well#2, Mutual Well#1, and Mutual Well#2, respectively. These figures show good correlation between the measured and simulated water levels. The greatest error appears to be associated with District Well#2 in the first two years of simulation. This may be associated with inappropriate initial conditions.



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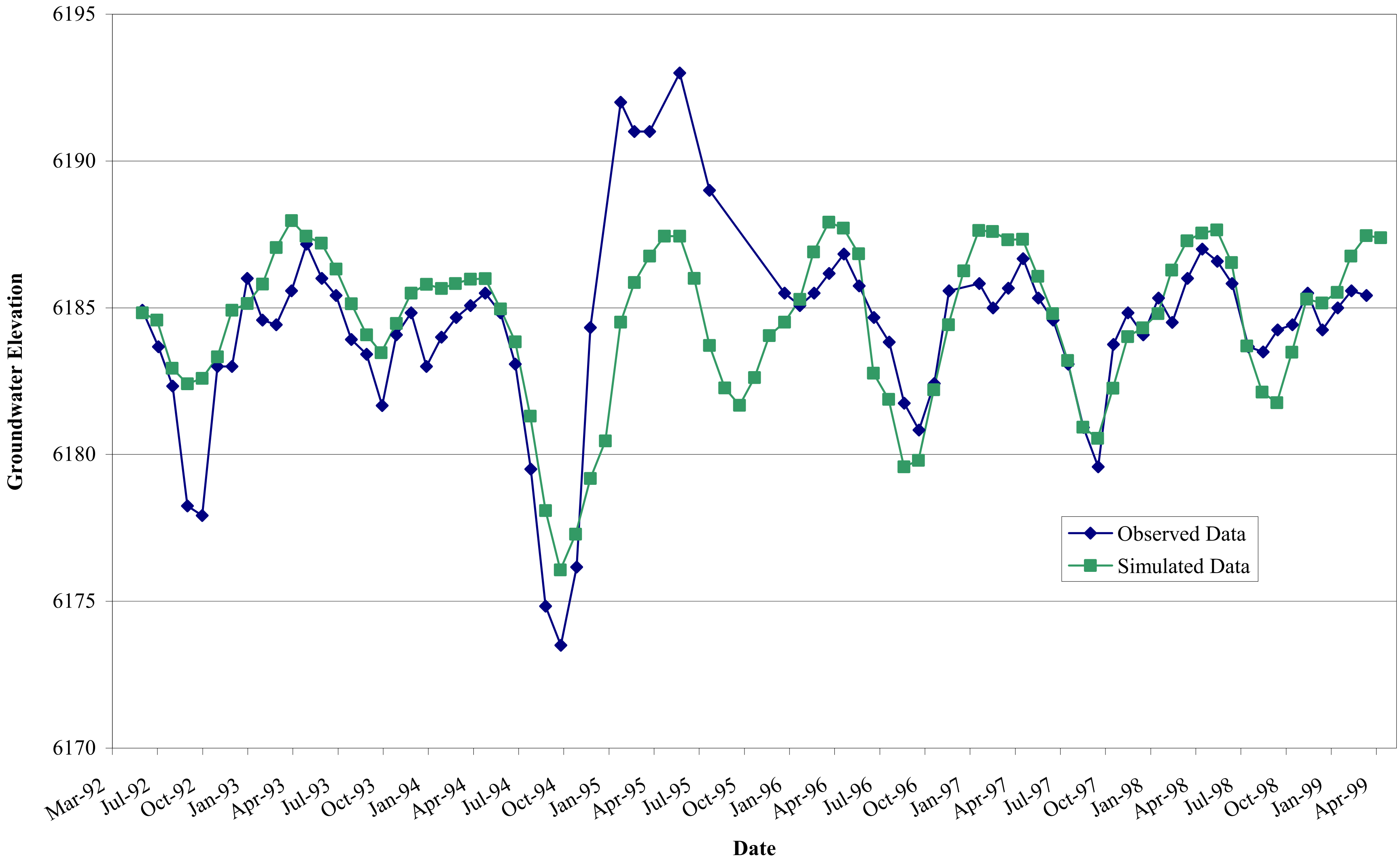
Observed and Simulated Water Levels, SVPSD Well#2

Figure

4-39



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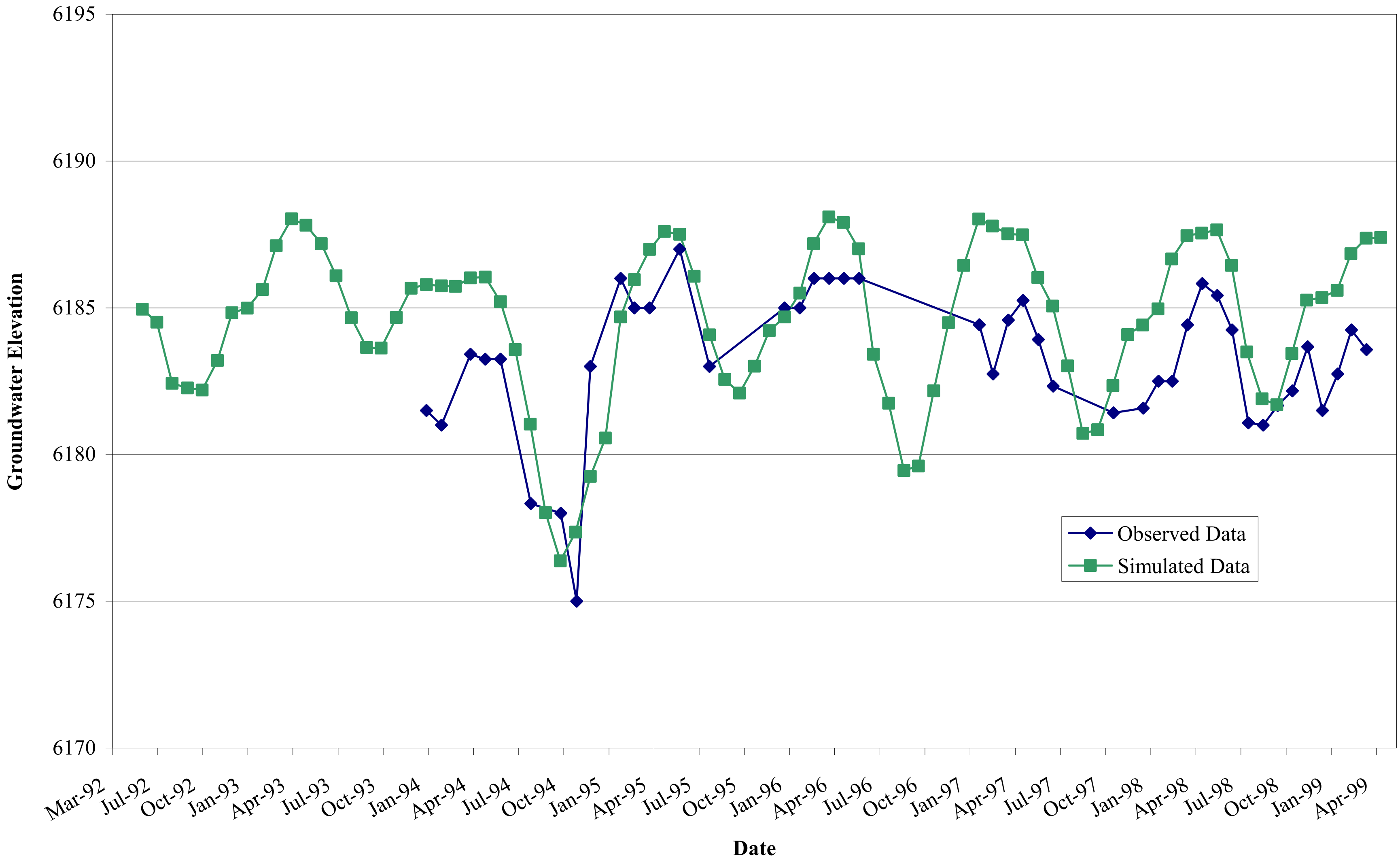
Observed and Simulated Water Levels, SVMWC Well#1

Figure 4-40



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Observed and Simulated Water Levels, SVMWC Well#2

Figure 4-41

SECTION 5. ESTIMATE OF ULTIMATE WATER PRODUCTION REQUIREMENT

Presented in this section is a projection of the ultimate buildout water demands in Squaw Valley. Projections are included for the demands served by the District and Mutual, and the Resort at Squaw Creek for golf course irrigation and snow making. The buildout water demand is based on recent estimates by the District of future development and current water use habits. Estimates of potential water savings from several water conservation measures are also provided. This is followed by the projection of ultimate water production requirements and a discussion of the need for additional water supply facilities.

DEMAND PROJECTION DATA SOURCES

Water demand projection data have been collected from several sources including:

1. Letter report about future water demands from ECO:LOGIC to Mr. Richard L. Lierman, dated February 8, 1999. This document presents a water demand projection at ultimate buildout for both the District and the Mutual.
2. Memorandum of Understanding Regarding Urban Water Conservation in California, by the Urban Water Conservation Council, as amended September 16, 1999.
3. Information provided by Aleta Drake and Jesse McGraw of the District, including:
   - Historical water demands by service types (residential, commercial, irrigation) and water production.
   - 1992 and current connection information
   - Anticipated ultimate buildout connections
   - Residential, commercial, and irrigation water rates
   - Information on leak detection/correction audits
   - Brief information on a water conservation plumbing retrofit program prepared in 1978.
4. Information provided by John Wilcox, office manager of the Mutual.
5. Information provided by Steve Bradley of the Resort at Squaw Creek.

DEMOGRAPHIC INFORMATION

The average annual water demand projection has been summarized from the ECO:LOGIC letter report and is presented as the 1999 Water Demand Projection (1999 Projection) in Table 5-1. The demographic information used in that water demand projection have been critically reviewed and updated as described below. The revised information has been used to develop the buildout water demand estimate. The buildout water demand assumes that the ultimate development in the valley will not exceed 80 percent of the development permitted in the 1983 Squaw Valley General Plan and Land Use Ordinance.

Table 5-1. 1999 Water Demand Projection[(a)]

Customer Category	Units Developed		Ave. Ann. Demand Factor, gpd/unit	Exist. Ave. Ann. Demand, gpd	Buildout Ave. Ann. Demand, gpd
	Existing	Buildout[(b)]			
Single Family Residential					
District Single Family Units—Metered	311	451	400	124,400	180,400
District Condo Units—Separately Metered	313	482	180	56,340	86,760
Mutual Single Family Units—Unmetered	278	306	500	139,000	153,000
Total Single Family Residential	902	1,239	1,080	319,740	420,160
Master Metered Residential—Condo and Hotel Units[(c)]	826	4,170	140	115,640	583,800
Commercial—Equivalent Dwelling Units[(d)]	149	852	207	30,843	176,364
Irrigation—Equivalent Dwelling Units	246	500	82.8	20,369	41,400
Total Demand				486,592	1,221,724
Unaccounted for Water at 12%				58,391	146,607
Total Production				544,983	1,368,331

(a) Data extracted from the ECO:LOGIC letter report to Richard Lierman, dated February 8, 1999.
(b) Ultimate buildout assumed to be 80 percent of development permitted in the 1983 Squaw Valley General Plan and Land Use Ordinance.
(c) Each condo and hotel unit assumed to have 1.5 bedrooms.
(d) Buildout commercial units are based on 1,330,000 square feet of new area using 40 gal/yr/ft^2.

District Single Family Connections

In the 1999 Projection, the District's single-family connections were projected to increase from 311 to 451 units. Based on recent information provided by the District, single-family connections are projected to increase from 292 to 454. The District's estimated buildout number of connections should be used in making future water demand projections.

District Individually Metered Condominiums

In the 1999 Projection, individually metered condominiums are expected to increase from 313 to 482 units, an increase of 169 units. Based on the conversion stated in the 1999 Projection (1.5 bedrooms per unit), this would equate to an increase of 253 bedrooms. The buildout projection recently provided by the District indicates that this increase will be about 159 bedrooms, which would be about 106 units. It is assumed that some of the projected 169 units were built over the last two years, and there is no reason to revise the total from the 1999 Projection.

Mutual Single Family Connections

In the 1999 Projection, Mutual single-family connections were estimated to increase from 278 units to 306 units. Based on information provided by John Wilcox, the Mutual provides water through 259 connections to 313 dwelling units (because 54 connections serve a primary house and an additional living or apartment unit located on the same parcel). There are about 20 empty lots in the Mutual service area. It is recommended that the Mutual build out projection be revised to include 333 units, rather than the 306 units used in the 1999 Projection.

District Master-Metered Residential Hotels and Condominiums

In the 1999 Projection, condominiums and hotel units served by the District were estimated to increase from 826 units (1,239 bedrooms at 1.5 bedrooms per unit) to 4,170 units (6,255 bedrooms) for an increase of 3,344 units (5,016 bedrooms). The buildout projection recently provided by the District indicates that master metered hotel and condo bedrooms will increase by 6,540 bedrooms, which equates to 4,360 units. It is recommended that the current projected increase (4,360 units) be used in the demand projection to bring the buildout number of units to 5,186.

District Commercial

In the 1999 Projection, the commercial demand is based on an increase of 1,330,000 square feet of commercial space. This increase compares well with the buildout projection provided by the District of an increase of 1,150,000 square feet, assuming some commercial space has been built in the last two years. No changes are recommended for the commercial demand projection.

District Landscape Irrigation

In the 1999 Projection, the District's irrigation demand was arbitrarily assumed to double from the current demand. For lack of other information this assumption appears reasonable, and no change to this assumption is recommended.

WATER USE ESTIMATES

The 1999 Projection buildout water demand, without implementation of any water conservation measures, was presented in Table 5-1. This demand projection is reviewed below.

District Individually Metered Residential

The District's average day demand for individually metered residential use as estimated in Table 5-1 is 124,400 gpd based on an average day demand per residence of 400 gpd. This use rate was based on estimates presented in the 1993 Water Master Plan. Based on more recent metered information provided by the District, the water used by the 311 single-family residential customers from May 1, 1999, to April 30, 2000 was about 51,541,000 gallons. The average day demand during the last year was 141,000 gpd or about 450 gpd per single-family residence. The estimated demand in Table 5-1 is about 12 percent less than the latest year's metered use. The existing use factor should be increased to match the 1999/2000-metered use rate of 450 gpd per residence.

A trend in the past couple of years has been observed where new houses and the rebuilding of existing houses results in substantially larger living units with permanent irrigated landscaping. A sample of recent water meter records of newly constructed houses shows the use is substantially higher for these houses. The average use for these new houses during 1999 and 2000 was about 750 gpd. District staff estimates that this trend will continue and probably about one half of the single-family lots will ultimately have substantially larger houses built upon them with the associated higher water demand. The buildout demand estimate will be based on half the single-family units using 450 gpd and the other half based on a use rate of 750 gpd.

District Master Metered Residential and Commercial

The District's combined master metered residential and commercial average annual demand from Table 5-1 is 146,000 gpd. Based on recent information provided by the District the individually metered water provided to this group of customers from May 1, 1999 to April 30, 2000 was about 60,092,000 gallons, or an average day demand of 163,000 gpd. These average day demand values are reasonably close, differing by about 12 percent. No changes to the master metered residential and commercial demand factors presented in the 1999 Projection are recommended.

District Landscape Irrigation

The landscape irrigation average day demand from Table 5-1 is 20,369 gpd. Based on information provided by the District, the metered annual average irrigation demand from May 1, 1999 to April 30, 2000 was about 22,270,100 gallons, or an average daily demand of 60,000 gpd. The current actual demand is about three times the irrigation demand estimated in Table 5-1. It is recommended that the irrigation demand factor be increased by a factor of 3.

Mutual Residential

The projected average day demand for the Mutual from Table 5-1 is 139,000 gpd. Assuming a similar unaccounted-for-loss rate to that of the District of 12 percent, this demand would require a production rate of 156,000. The annual production values for 1990 to 1999 are shown in Table 5-2. The average day production for the period was about 124,000 gpd. The maximum average daily production during this period was about 139,000, which occurred in 1994 and 1997. The projected water production for the Mutual is only about 12 percent higher than the average daily production rate during 1994 and 1997. These values are reasonably close, and consequently no change is recommended to the Mutual single-family connection demand factors for existing use presented in the 1999 Projection.

Similar to the increasing trend in water use of large new or rebuilt houses within the District, it is anticipated that one half of the single family residential lots in the Mutual service area will ultimately be rebuilt and use an average of 750 gpd. The buildout demand estimate will be based on half the single-family units using 500 gpd and the other half based on a use rate of 750 gpd.

Table 5-2. Squaw Valley Mutual Water Company's Annual Water Production

Year	Annual Production, gal/yr	Annual Production, af/yr	Average Daily Production, gpd
1990	38,221,800	117	104,717
1991	37,447,543	115	102,596
1992	43,683,800	134	119,682
1993	46,605,600	143	127,687
1994	50,963,660	156	139,626
1995	46,240,500	142	126,686
1996	49,359,720	151	135,232
1997	50,659,100	155	138,792
1998	40,730,030	125	111,589
1999	47,626,700	146	130,484
Average	45,153,845	139	123,709

Resort at Squaw Creek Golf Course Irrigation and Snow Making

Annual pumping by the Resort at Squaw Creek for golf course irrigation and snow making can vary significantly depending on climatic conditions in any particular year. Presented in Table 5-3 is a summary of 1990 to 1999 water production estimates for golf course irrigation and snow making.

Table 5-3. Golf Course Irrigation and Snow Making Water Demands

Year	Golf Course Irrigation		Snow Making	
	Annual Demand, gal/yr	Annual Demand, af/yr	Annual Demand, gal/yr	Annual Demand, af/yr
1990	27,500,000	84	40,000,000	123
1991	54,900,000	168	27,768,900	85
1992	54,900,000	168	22,887,500	70
1993	54,900,000	168	40,000,000	123
1994	53,745,239	165	16,250,000	50
1995	54,900,000	168	21,611,300	66
1996	55,241,377	170	34,742,400	107
1997	44,010,467	135	16,680,000	51
1998	42,236,008	130	40,000,000	123
1999	44,897,735	138	33,000,000	101
Average	48,723,083	150	29,294,010	90
Maximum	55,241,377	170	40,000,000	123

When the golf course superintendent was interviewed, he stated that the course has matured since opening in 1990, and irrigation amounts and scheduling are closely managed. Therefore, the last three years should be more representative of future golf course water use. Since 1997, golf course irrigation demands have averaged 44 million gallons per year. It is recommended that 45 million gallons be used as the average annual golf course irrigation demand, and the demand be distributed over 150 days (mid-May through mid-October) for calculation of the average day golf course irrigation demand. The average production would therefore be about 300,000 gpd during the irrigation season.

Since the winter of 1989-90, snow making demands have averaged 29 million gallons per year, with the highest annual demand which has occurred in three of the past ten years estimated to be 40 million gallons. Snow making typically occurs during the period of November through mid-January. For water supply planning purposes, it is recommended that 40 million gallons be used as the typical annual snow making demand, and the demand be distributed over 80 days for calculation of average day snow making demand, or 500,000 gpd. For the buildout projection, it is assumed that no additional snow making demands will be met by water pumped from the valley basin. Future increased snow making demands may be met by supplies from on the mountain, which could impact the recharge to the aquifer.

UPDATED BUILDOUT DEMAND PROJECTION

Presented in Table 5-4 is the updated annual demand projection at buildout for the District and the Mutual incorporating the latest information obtained from the District and the changes in water demand factors recommended above. These changes result in the current average day estimated demand increasing from 486,000 gpd calculated in the 1999 Projection to 543,000 gpd, and the buildout average day demand increasing from 1.22 million gallons per day (mgd) to 1.59 mgd.

The demands need to be increased by 12 percent to account for system losses to obtain the water production required to meet demands. The total production required at buildout is thus estimated to be 1.79 mgd. Presented in Table 5-5 are the annual required production estimates at buildout in million gallons/year and af/year for the District and the Mutual. Also included in Table 5-5 are the Resort at Squaw Creek's annual pumping requirements for golf course irrigation and snow making. The total water production requirements in the valley at buildout are estimated to be 2,262 af/yr. This amount is based on current water use practices.

WATER CONSERVATION

A recommended water conservation program for the District and Mutual customers has been developed. A series of water conservation measures including showerhead and toilet replacements, metering with a conservation rate schedule, system water audits, and a landscape conservation program are recommended to be included in the program. The estimated water savings are based on the California Urban Water Conservation Council's *Memorandum of Understanding Regarding Urban Water Conservation in California* (as amended September 16, 1999). The estimated maximum potential water conservation savings resulted from application of the recommended best management practices (BMPs) is summarized in Table 5-6, and discussed below.

Table 5-4. Updated Buildout Water Demand Estimates for SVPSD and SVMWC[a]

Customer Category	Units Developed		Average Day Demand Factor, gpd/unit		Average Day Demand, gpd	
	Existing	Buildout[b]	Existing	Ultimate	Existing	Buildout
Single Family Residential						
District Single Family Units—Metered	311	454	450	600[c]	139,950	272,400
District Condo Units—Separately Metered	313	482	180	180	56,340	86,760
Mutual Single Family Units—Unmetered	278	333	500	625[c]	139,000	208,125
Total Single Family Residential	902	1269			335,290	567,285
Master Metered Residential—Condo and Hotel Units[d]	826	5,186	140	140	115,640	726,040
Commercial—Equivalent Dwelling Units[e]	149	852	207	207	30,843	176,364
Irrigation—Equivalent Dwelling Units[f]	246	500	250	250	61,500	125,000
Total Demand					543,273	1,594,889
Unaccounted for Water at 12%[g]					65,193	191,363
Total Production Required					608,466	1,786,052

(a) Data as updated in this technical memorandum.
(b) Ultimate buildout assumed to be 80% of development permitted in Squaw Valley General Plan or as estimated by the District.
(c) Based on one half of units using 750 gpd.
(d) Each condo and hotel unit assumed to have 1.5 bedrooms.
(e) Buildout commercial units are based on 1,330,000 ft^2 of new area using 40 gal/yr/ft^2..
(f) Metered irrigation consumption is expected to double at buildout
(g) Based on 1993 Water Master Plan report.

Table 5-5. Summary of Updated Buildout Water Demand Estimates

Water Use Category	Annual Demand	
	million gallons/year	af/year
District	567	1,740
Mutual	85	261
Golf Course Irrigation	45	138
Snow Making	40	123
Total	737	2,262

Table 5-6. Summary of Estimated Water Conservation Savings

Recommended Conservation BMP	Current Potential Conservation		Buildout Potential Conservation	
	gpd	af/yr	gpd	af/yr
Low Flow Showerhead Replacement Program	6,800	8	6,800	8
Ultra-Low Flow Toilet Replacement Program	20,500	23	20,500	23
System Audits and Leak Detection/Repair	11,000	12	32,000	36
Metering with Commodity Rates	37,400	42	75,000	83
Large Landscape Conservation Program	9,000	10	18,750	21
Total Potential Conservation Savings	84,700	95	153,050	171

The conservation estimates represent the maximum potential conservation for each recommended BMP. That is, they represent the potential conservation if the BMP were complied with by 100 percent of the water customers. There are several ways that conservation programs can be implemented:

> Voluntary with incentives – This method includes providing the conservation BMPs to the water customers, and providing some sort of rebate (*e.g.* for toilet replacement) or incentive for the voluntary implementation of the program. This method would likely not achieve a 100 percent compliance rate, and the actual conservation would be less than estimated below.
>
> Mandatory compliance – This method includes enforcing the conservation BMPs through local ordinances with active review of compliance. This method of implementation would likely achieve nearly complete compliance.

Residential Plumbing Retrofit

This BMP includes providing high quality low flow showerheads, toilet displacement devices, toilet flappers, and faucet aerators to facilities constructed before 1992. Toilet retrofit devices have often provided customers with unsatisfactory performance. Implementation of a toilet retrofit device program is not recommended, but an ultra-low flow toilet (ULFT) toilet replacement program (see below) could be implemented. Showerhead replacement programs have been successful in other communities and have resulted in high customer satisfaction and significant water conservation.

The District implemented a plumbing retrofit program in 1978, in which it is believed that many low flow showerheads and toilet fillers were installed. For this analysis, it is assumed that no toilet fillers from this program are still in use. It is also assumed that in the District service area, all pre-1980 showerheads were replaced with post-1980 showerheads.

Low Flow Showerhead Replacements

Water savings using low flow showerheads are 7.2 gallons per capita per day (gpcd) for pre-1980 construction and 2.9 gpcd for 1980 to 1992 construction, based on a 100 percent occupancy rate. It is not appropriate to apply these standard conservation factors directly to houses in Squaw Valley because many Squaw Valley homes are occupied only part time. Conservation savings for this measure that are more appropriate for Squaw Valley are described below.

District Individually Metered Single Family Units

Assuming all pre-1980 houses were retrofit with low-flow showerheads during the 1978 retrofit program, all dwelling units constructed before 1992 would have low-flow showerheads. Assuming about three people per dwelling unit (DU), the conservation savings would be 8.7 gpd per DU. If an average water use for a 3-person house is about 450 gpd, then the percent reduction is about two percent.

This two percent reduction can only be applied to houses with post-1980 showerheads. Based on information provided by the District, there were about 240 single-family residences in 1992. The Water Conservation Memorandum of Understanding recommends an annual natural wear out and replacement rate of three to five percent. Assuming a natural wear out and replacement rate of four percent per year, only 72 percent of these DUs still have post-1980 showerheads, or 173 DUs. The potential water conservation savings from this measure is thus, (173 DUs / 311 DU current units) * (2% of 140,000 gpd) = 1,560 gpd.

District Individually Metered Condominium Units

Following the same logic applied to the single-family units, but assuming a conservation savings of 5.8 gpd per DU (2 people per unit) for a typical condominium using 180 gpd, the percentage savings is about 3.2 percent, or about 1,800 gpd.

Mutual Single Family Units

John Wilcox, Mutual manager, estimated that within the Mutual service area about 1/3 of the homes are occupied full time, and the rest are occupied from three to four weeks per year to full time. Assuming 2/3 of the homes are occupied half time, results in an average occupancy rate of about 67 percent.

Assuming there are about three people per house in the Mutual service area, then the pre-1980 showerhead replacement conservation savings is (7.2 gpd/person) * (3 people/DU) * (67% occupancy) = 14.5 gpd/DU. For post-1980 showerheads the replacement conservation savings is (2.9 gpd/person) * (3 people/DU) * (67% occupancy) = 5.8 gpd/DU.

Based on information provided by the District, Mutual, and the Squaw Valley General Plan and Land Use Ordinance, it appears that the Mutual housing stock has the following showerhead distribution, which results in the following potential water savings:

Pre-1980 – about 98 DU * 14.5 gpd/DU =	1,421 gpd
Post-1980 – about 120 DU * 5.8 gpd/DU =	696 gpd

Post-1992 – about 92 DU = (no potential conservation available)
Total Potential conservation = 2,117 gpd

Resort at Squaw Creek

The following analysis is based on information provided by Steve Bradley of the Resort at Squaw Creek. There are 405 total rooms (roughly 380 single rooms and 25 suites), which have about 440 showerheads. Post-1980 type showerheads were installed originally, and about 40 have been replaced with post 1992 showerheads, leaving about 400 post 1980 showerheads. Assuming two guests per room on the average, the conservation factor is (2.9 gallons per guest) * (2 guests per room) * (65% occupancy) = 3.8 gpd/shower. The potential water savings is (3.8 gpd/shower) * (405 showers) = 1,540 gpd.

The estimated potential water conservation for a showerhead replacement program is summarized in Table 5-7. The conservation from this measure applies to only houses built before 1992, so the estimated conservation is the same at buildout as it is today.

Table 5-7. Estimate of Potential Water Conservation from a Shower Head Replacement Program

Item	Potential Conservation, gpd	Potential Conservation, af/yr
District Individually Metered Single Family Units	1,560	1.7
District Individually Metered Condo Units	1,628	1.8
Mutual Single Family Units	2,117	2.4
Resort at Squaw Creek	1,540	1.7
Total Potential Conservation	6,805	7.6

ULFT Replacement Programs

This BMP includes replacing non-ULFTs with ULFTs. It requires an implementation program that is at least as effective as requiring replacement of non-ULFTs at time of property resale. Often a rebate program for replacement of non-ULFTs with ULFTs is also implemented.

In 1978 a plumbing retrofit program was implemented in which toilet fillers were provided to the District customers. It is assumed that none of the toilet fillers are still in use today.

Pre-1980 Toilets. The potential water conservation from a toilet replacement program is based on the estimated housing stock for the various toilet types (see Table 5-8). Assuming an average of about three people and two toilets per DU, the conservation factor would be 42 gpd per DU for pre-1980 toilets. If an average water use for a 3-person house is about 400 gpd, then the percent reduction is about 10 percent.

Table 5-8. Estimate of Potential Water Conservation from a Toilet Replacement Program

Item	Quantity, DUs
Pre-1980 Toilets—Total	640
DU with toilets replaced through retrofit program	0
Quantity replaced through normal failure (at 4% per year)	357
DU with pre-1980 toilets remaining that could be replaced in a retrofit program	283
1980 to 1992 Toilets—Total	89
DU with toilets replaced through retrofit program	0
Quantity replaced through normal failure (at 4% per year)	25
DU with 1980-1992 toilets remaining that could be replaced in a retrofit program	64

This 10 percent reduction can only be applied to houses with pre-1980 toilets. Based on information in Table 5-8, there are about 283 DUs with pre 1980 toilets. The potential water conservation from this measure is thus, (283 DUs/902 DU current units) * (10% of 320,000 gpd) = 10,000 gpd.

1980-1992 Toilets. Assuming an average of about three people and two toilets per DU, the conservation factor for 1980-1992 toilets would be 33 gpd per DU for post-1980 toilets. If an average water use for a 3-person house is about 400 gpd, then the percent reduction is about eight percent.

This eight percent reduction can only be applied to houses with 1980-1992 toilets. Based on information in Table 5-8, in there are about 64 DUs with 1980-1992 toilets. The potential water conservation from this measure is thus, (64 DUs / 902 DU current units) * (8% of 320,000 gpd) = 1,800 gpd.

Resort at Squaw Creek

The following analysis is based on information provided by Steve Bradley of the Resort at Squaw Creek. There are 405 total rooms (roughly 380 single rooms and 25 suites), which have about 440 toilets. The toilets are all of the 1980-1992 type, except for the three or four that have been replaced, leaving about 436, 1980-1992 type toilets. Assuming 2 guests per toilet on the average, the conservation factor is (31 gpd/toilet) * (65% occupancy) = 20 gpd/toilet. The potential conservation is (20 gpd/toilet) * (436 toilets) = 8,700 gpd.

The combined potential water conservation from a toilet replacement program by the District and the Mutual is about 20,500 gpd, or about 23 af/yr.

System Water Audits, Leak Detection, and Repair

This BMP involves estimating unaccounted-for water losses annually and if losses are greater than 10 percent of production, performing a detailed investigation to identify and correct leaks as described in the AWWA's *Water Audit and Leak Detection Guidebook*. Conservation is accounted for by limiting unaccounted-for losses to 10 percent.

The District performed this type of audit in 1995 and 1996. As a result of these audits, several leaks were found and repaired, resulting in a decrease in the estimated unaccounted-for losses from about 20 percent to about 12 percent. The District annually looks for and repairs leaks, but has not done a formal audit since 1996.

The Mutual has performed two leak detection surveys in the last 3.5 years and found and repaired several leaks. Assuming the Mutual was as successful as the District, the unaccounted-for losses for the Mutual are probably about 12 percent.

Implementation of annual formal audits with a goal of reducing unaccounted-for losses to 10 percent would likely reduce current and buildout production by about two percent, or 11,000 gpd currently (12 af/yr) and 32,000 gpd at buildout (36 af/yr). At buildout, the District would save about 31 af/yr and the Mutual would save about 5 af/yr. If, on the other hand, audits are not ever performed and leaks are not located and fixed, unaccounted-for losses could be expected to increase back to 20 percent, increasing the current average day demand by 44,000 gpd and by 128,000 gpd at buildout.

Metering with Commodity Rates for All New Connections and Retrofit of Existing Connections

This BMP includes requiring meters on all new connections, retrofitting all existing unmetered connections, and billing by volume of water used. It also includes identifying inter- and intra-agency barriers to retrofitting commercial mixed-use (domestic and irrigation uses) with dedicated landscape meters, and conducting a feasibility study of providing incentives to switch mixed use accounts to dedicated landscape meters. The estimated water savings from this BMP is 20 percent.

All District connections are already metered, and the District charges for water using a declining block rate structure for residential meters, an inclining rate structure for commercial meters, and a neutral structure for landscape meters. The District has already implemented much of this BMP. Implementing an inclining block rate billing structure for residential meters would not achieve the 20 percent reduction estimated for this BMP, but might reduce demand by a small percentage beyond the current and projected demands. For the District, implementation of an inclining block rate billing structure is assumed to reduce residential demand by three percent, or 9,400 gpd under current conditions (11 af/yr) and 33,000 gpd (36 af/yr) at buildout (exclusive of the Mutual).

None of the Mutual connections are metered. Metering all Mutual's connections, and implementing an inclining block rate billing structure would likely reduce the Mutual's demands by 20 percent, or 28,000 gpd (31 af/yr) currently and 42,000 gpd (47 af/yr) at buildout.

Large Landscape Conservation Programs and Incentives

This BMP includes providing nonresidential customers with support and incentives to improve their landscape water use efficiency, including:

- Assigning landscape metered accounts a water budget based on irrigated area, landscape type and local evapotranspiration and providing notification each billing cycle if the budget amount was exceeded.
- For unmetered or mixed-use metered accounts, offer landscape water use surveys/analysis, voluntary water budgets, installation of dedicated landscape meters, training in landscape maintenance, training in irrigation system design and maintenance, and financial incentives for installation of water efficient irrigation systems.

The estimated water savings for this BMP for each account for which it is implemented is 15 percent of landscape water use. For the District, if this BMP were implemented, the irrigation demand could be reduced by 15 percent, or 9,000 gpd (10 af/yr) currently and by 18,750 gpd (21 af/yr) at buildout. For the Mutual no significant water demand reduction is likely through this BMP.

Review of Table 5-6 shows that if the recommended water conservation measures by the District and the Mutual are fully implemented, the annual savings in water production can total 171 af at buildout. The buildout water demand estimates shown in Table 5-5 would therefore be reduced to the totals shown in Table 5-9.

Table 5-9. District and Mutual Buildout Production Requirements with Recommended Conservation Program (af/yr)

Supplier	Production Requirement w/o Conservation	Estimated Savings	Production Requirement w/ Conservation	Percent Savings
District	1,740	112	1,628	6.4
Mutual	261	59	202	22.6
Total	2,001	171	1,830	8.6

Squaw Valley Buildout Water Production Requirements with Conservation

The buildout water production requirements in the valley with full implementation by the District and the Mutual of the recommended conservation program described above and pumping by the Resort at Squaw Creek for golf course irrigation and snow making is summarized in Table 5-10. The total annual production estimated at full build-out is 2,091 af per year, or 681 million gallons.

Table 5-10. Required Annual Water Production with Conservation In Squaw Valley at Buildout (af)

Supplier/Use	Required Production
Squaw Valley Public Service District	1,628
Squaw Valley Mutual Water Company	202
Resort at Squaw Creek	
Golf Course Irrigation	138
Snowmaking	123
Total	2,091

For use in water supply planning and for input to the groundwater model, the annual production has been transformed into monthly water production by supplier as shown in Table 5-11.

Also the average day and maximum day production requirements for the District and SVMWC have been estimated and are shown in Table 5-12. The recommended minimum water supply facilities production capability for municipal water purveyors is to be able to meet the maximum day production requirements with the largest supply source out of service. The maximum day production requirements will be used in future work to identify the needed number and size of wells to be in service at buildout.

Table 5-11. Monthly Water Production Requirement in Squaw Valley at Buildout (af)

Month	District	SVMWC	Golf Course Irrigation	Snow Making	Total Production
January	126	12	0	31	169
February	114	10	0	0	124
March	119	11	0	0	130
April	106	10	0	0	116
May	109	15	14	0	138
June	159	22	28	0	209
July	213	31	28	0	272
August	218	30	28	0	276
September	166	24	27	0	217
October	112	16	13	0	141
November	76	9	0	46	131
December	112	11	0	46	169
Totals	1,628	202	138	123	2,091

Table 5-12. Average Day and Maximum Day Production Requirements for District and Mutual at Buildout

Purveyor	Average Day Production Requirement		Maximum Day Production Requirement[a]	
	gpm	mgd	gpm	mgd
Squaw Valley Public Service District	1,010	1.45	2,525	3.64
Squaw Valley Mutual Water Company	125	0.18	315	0.45
Total Municipal Production Requirement	1,135	1.63	2,840	4.09

(a) Maximum Day Production Requirement = 2.5 times the Average Day Production Requirement

WATER PRODUCTION REQUIREMENTS

The water supply and production for the District and the Mutual are identified in Tables 5-10 and 5-12. The annual supply should be available in all years, except in drought emergency years when demand management should be implemented to reduce demands to equal the supply available. The District has recently enacted a water conservation ordinance to assist in managing the demands and groundwater resource. Section 3.33 “Critical Water Supply Shortage, Emergency Water Conservation Restrictions” of the District’s Water Code, sets forth requirements for all District customers to implement mandatory reduction in average base water consumption by 20 percent or more during a critical water supply shortage. A 20 percent reduction in the District’s buildout demand shown on Table 5-10 would reduce the annual water supply requirement from 1,628 af to about 1,300 af. A 30 percent reduction would lower the District’s annual water supply requirement to about 1,140 af.

The District’s water supply production facilities should be capable of supplying the maximum day demand with the largest well out of service. The pumping capacities of the existing wells are shown in Table 5-13. The pumping capacities range from 120 gpm for Well 3 to 400 gpm for Well 5. The District’s total pumping capacity is 1,250 gpm. In addition, the District has two horizontal wells that are capable of producing up to 40 gpm, and with a dry year yield of about 23 af. Therefore, the total water supply capacity of all sources is 1,290 gpm. With the largest well out of service, the total capacity is 890 gpm.

Table 5-13. Existing District Water Supply Capacity, (gpm)

Existing Supply Facility	Pumping Capacity
Well 1	390
Well 2	340
Well 3	120
Well 5	400
Horizontal Wells	40
Total Supply Capacity	1,290

In average or wet years, the District's buildout demands were estimated to increase to the values shown in Table 5-12. The water production facilities must produce the maximum day demand with the largest production well out of service. The estimated maximum day demand is 2,525 gpm. Assuming the largest well is out of service, the water supply capacity should be increased by 1,635 gpm. To meet this requirement, the District will need to construct four to six new wells that produce in the range of 250 to 400 gpm each.

SECTION 6. GROUNDWATER MODEL SIMULATIONS TO ESTIMATE SUSTAINABLE YIELD

To develop a reasonable estimate of the dependable water supply that can be developed for use within Squaw Valley, a series of runs were completed using the updated groundwater model. The model runs estimated the basin's sustainable yield during drought years. The definition of sustainable yield was first developed and then a series of iterative model runs were undertaken to develop estimates of the maximum pumping that can be sustained during a critically dry year. This section presents a summary of the process followed to develop the basin's estimated sustainable yield and the results of the analyses using the updated model.

DEFINITION OF SUSTAINABLE YIELD

For this study, sustainable yield has been defined as the maximum amount of reasonable quality water that can be pumped from the groundwater basin during a critically dry year without significantly impacting the pumping water levels of existing wells. Additional water could be pumped from the valley east of the parking lot, but studies show that the water will probably be of marginal quality for drinking purposes and require substantially more treatment than would be economically justified. The sustainable yield analyses of the basin assumed the recharge in a critically dry year is represented by that experienced in 1994. Pumping of existing wells was increased and proposed new wells added in the basin to identify the maximum annual pumping amount that can be sustained without lowering the pumping water levels below the top of the existing wells' perforations. This criterion is a conservative approach for defining sustainable yield. The District could and has operated its wells at lower water levels for short periods of time. Lowering water levels below the perforations can lead to operational problems including cascading water, entrained air and increased biofouling. As detailed in the Appendix, a series of analyses were performed to first identify maximum pumping using only the existing District and Mutual wells, and then using the existing wells and proposed new wells, to estimate the sustainable yield of the basin.

In addition, the sustainable yield analysis was completed after relaxing two important assumptions contained in the previous analyses. This final analysis again looked at the sustainable yield available from both existing and new wells. The significant assumptions changed during this analysis included:

1. The perforations in District Well#2 were assumed to be lowered by 15 feet. This could be accomplished by sleeving the existing well or reconstructing the well since it is over 40 years old. This assumption will lower the acceptable simulated water level by 15 feet. The previously assumed minimum water level in District Well#2 was 6177 feet msl. The new assumption results in a minimum water level of 6162 feet msl for District Well#2.
2. The wells could provide different percentages of water throughout the year. In the previous analysis, the pumping rates for all wells were increased and decreased by the same amount each month. A 20% increase in total pumping was attained by all wells increasing production by 20%. In the current analysis, all wells can be increased or decreased independently.

As with the previous analyses described below, two consecutive critically dry years were simulated in each run. The 1994 hydrology was used to simulate the hydrology of a critically dry year.

IDENTIFICATION OF CRITICALLY DRY YEAR

The groundwater model was developed and calibrated for the period of 1992 through 1999. It was determined from the calibration efforts that the average annual recharge from direct precipitation and streamflow from snowmelt typically provides an abundant supply that can meet existing and future demands. However, it was also noted that during low precipitation years, the groundwater levels could be significantly lowered during the late summer and early fall months when there is very little recharge. Review of precipitation records from the Squaw Valley Fire Station revealed that 1993-94 was a critically dry year, with total precipitation at the second lowest level ever recorded in the station's 36-year history. Only 1976-77 was drier than 1993-94. During the summer and fall of 1994, the District was very concerned about falling groundwater levels and was considering enacting measures outlined in the Squaw Valley Water Management Action Plan to maintain adequate water levels and water quality in the basin. The hydrology during 1994 was thus used to characterize drought year supply recharge conditions in the model analyses to estimate sustainable yield. The calendar year hydrology of January through December 1994 was used for the sustainable yield analysis; having the basin essentially full at the beginning and end of the analysis period with minimal inflow from precipitation and stream recharge.

GROUNDWATER MODEL SIMULATIONS

Three factors were combined to define each simulation. The three factors include precipitation (recharge), acceptable minimum groundwater levels, and number of active wells and monthly pumping amounts. Each of these factors is discussed below.

Precipitation

Annual precipitation conditions determine the amount of water available for recharge to the groundwater basin. Precipitation recorded at the Squaw Valley Fire Station from July 1964 through June 2000 has averaged 55.2 inches per year. Rainfall in 1994 was 66 percent of the average rainfall, which was the second driest year during the 36 years of record at the Fire Station. This year is a reasonable representation of drought conditions expected in the Squaw Valley basin. The only year with lower rainfall was 1976, when precipitation was only 40 percent of average. The use of water years (October through September) was investigated to see if they should be analyzed rather than calendar years. The October 1993 through September 1994 water year represents 46 percent of average annual rainfall. The 1993-94 water year is again the second lowest precipitation year with 1976-77 being the only year with lower precipitation.

Because the groundwater model has been calibrated for the years 1992 to 1999, there is a level of confidence that the model reasonably represents the groundwater response to recharge in those years. It was therefore advantageous to use recharge data from these years to represent recharge conditions for the sustainable yield analysis simulations.

An additional factor considered in developing sustainable yield simulations is the number of years to be simulated. One drought year may not accurately simulate a worst-case drought, however multiple years of recharge less than 50 percent of average may overestimate the impacts of drought. After review of the hydrologic record and model calibration results, it was determined that the simulation period be limited to one drought year cycle that would begin in January with a recharged groundwater basin, and end in December after the storage in the basin has been used and water levels have recovered from recharge created by fall precipitation. The model was run for two consecutive years using the 1994 hydrology to better define the sustainability of the groundwater resource during the selected dry period.

Minimum Groundwater Levels

In the sustainable yield analysis, the amount of pumping is limited by requiring the groundwater pumping levels to remain above the top of the perforations in the existing wells. The elevations of the top of the perforations of the existing District and Mutual wells were defined in a letter from Jesse McGraw, District Operations Manager, dated January 22, 2001, and are summarized in Table 6-1. For the 2003 Update, the top of the perforations in District Well #2 are assumed to be lowered by 15 feet for estimating sustainable yield at buildout.

The updated groundwater model was used to estimate the response of groundwater levels to increased pumping. The model is a tool used for basin-wide studies and uses a monthly time step. The model-simulated water level in a pumping well will be higher than the observed pumping level for three reasons:

- The model does not account for well efficiencies
- The model calculates an average water level for an area in the vicinity of the pumping well
- The model calculates an average level for the month

Table 6-1. Minimum Acceptable Simulation Water Levels

Well Owner and Number	Ground Elevation, ft	Elevation of Top of Perforations, ft	Minimum Acceptable Simulation Water Level Elevation, ft	
			50% Efficiency	70% Efficiency
District #1	6202.6	6125.6	6155.6	6143.6
District #2[(a)]	6202.2	6167.0[(a)]	6177.0[(a)]	6173.0[(a)]
District #3	6201.9	6125.0	6149.5	6139.7
District #5	6199.3	6133.3	6157.0	6147.4
Mutual #1	6190.5	6130.5	6158.8	6149.3
Mutual #2	6195.0	6160.0	6169.5	6163.7

(a) The 2003 Update assumed the top of the perforations are lowered 15 feet. The resulting elevations are 6152.0, 6162.0, and 6158.0, respectively.

To assure that the water level in a well remains above the top of the well perforations, the minimum acceptable simulated water levels need to be adjusted to account for the three factors. The minimum acceptable simulated water levels were estimated by applying an assumed well efficiency to the maximum drawdown to the top of the well's perforations. The following formula was used to relate the maximum allowable drawdown to the required minimum simulated water level.

(Maximum allowable drawdown) x (well efficiency) = (Maximum acceptable simulated drawdown)

The maximum allowable drawdown is the difference between the fully recharged water level and the top of the perforations shown in Table 6-1. The maximum acceptable simulated drawdown is the difference between the fully recharged water level and the minimum acceptable simulated water level. The fully recharged water level for each well was estimated from plots of historical water levels.

Well efficiencies of 50 percent and 70 percent were used to estimate the minimum acceptable simulated water levels for each production well. The actual efficiency for the production wells is not known, however, these efficiencies are considered to be conservative estimates for most wells in the valley. The efficiency of a well is dependent on the initial construction and the age of the well. Well efficiencies in Squaw Valley are the highest just after construction or rehabilitation and then decline with age due to clogging of the screen/perforations and gravel pack due primarily to the growth of iron bacteria. The resulting minimum acceptable simulated water levels are shown in Table 6-1 for both 50 percent and 70 percent efficiencies. Review of recent pumping level data collected by the District shows that the typical drawdown experienced in the wells is reasonably represented by the assumptions made in using well efficiencies in the range of 50 percent to 70 percent. In analyzing the model results resulting from increased pumping, the simulated minimum water levels were compared to the minimum acceptable levels to identify the maximum annual pumping that could be achieved without violating the criteria for maintaining pumping levels above the top of the perforations.

Pumping Scenarios

Pumping scenarios were developed for two groups of wells: 1) only existing well; and 2) adding new wells to meet ultimate buildout demands. The typical annual demand pattern was identified from pumping records from the District, the Mutual and the Resort at Squaw Creek. The monthly distribution of the annual demands was based on the percentages shown in Table 6-2. The annual pumping amounts and monthly pattern of use for golf course irrigation and snow making are assumed to remain constant in the future. The annual demand for the District and the Mutual were then increased to represent future conditions, and the monthly pumping requirements by entity determined.

Table 6-2. Monthly Pumping Based on Percentage of Annual Supply Requirement

Month	District	Mutual	Golf Course Irrigation	Snowmaking
January	7.8	6.0	0	25.2
February	6.9	5.0	0	0
March	7.3	5.4	0	0
April	6.5	5.4	0	0
May	6.7	7.4	10.1	0
June	9.8	10.9	20.3	0
July	13.1	15.3	20.3	0
August	13.4	14.9	20.3	0
September	10.2	11.9	19.6	0
October	6.8	7.9	9.4	0
November	4.7	4.5	0	37.4
December	6.8	5.4	0	37.4
Totals	100	100	100	100

The monthly pumping estimated for each well was determined by using the ratio equal to the well's pumping capacity divided by the total amount of pumping capacity of the entity. The current pumping capacities of the existing wells are shown in Table 6-3.

Table 6-3. Existing Well Pumping Capacities

Existing Well	Pumping Capacity, gpm
District #1	390
District #2	340
District #3	120
District #5	400
Mutual #1	200
Mutual #2	200
Resort 18-1	20
Resort 18-2	150
Resort 18-3	266

Two sets of pumping scenarios were developed:

- A set of monthly pumping rates was developed to identify the maximum pumping rates from existing wells that can be maintained during critically dry years. If the maximum pumping rates can not be sustained, pumping was reduced to estimate the maximum pumping rate that can be sustained during critically dry years using only existing wells.
- Sets of monthly pumping rates using existing and new wells were developed to estimate the maximum pumping rate that can be sustained during critically dry years. If the increased pumping cannot be sustained, then the pumping rate was reduced to estimate the maximum pumping rate that can be sustained during critically dry years.

The monthly pumping rates incorporated into the base simulation for using only the existing wells were based on the estimated current maximum pumping rates for each well and are shown in Table 6-4. The wells pumping in the base simulation include District Wells 1, 2, 3 and 5 and Mutual Wells 1 and 2. Total annual pumping was increased to 807 af for the District and to 202 af for the Mutual. Pumping by the Resort at Squaw Creek is assumed to remain constant at 261 af per year. The total monthly pumping rates are plotted on Figure 6-1.

Table 6-4. Monthly Pumping Rate by Well for Maximum Pumping of Existing Wells Base Case Simulation (af)

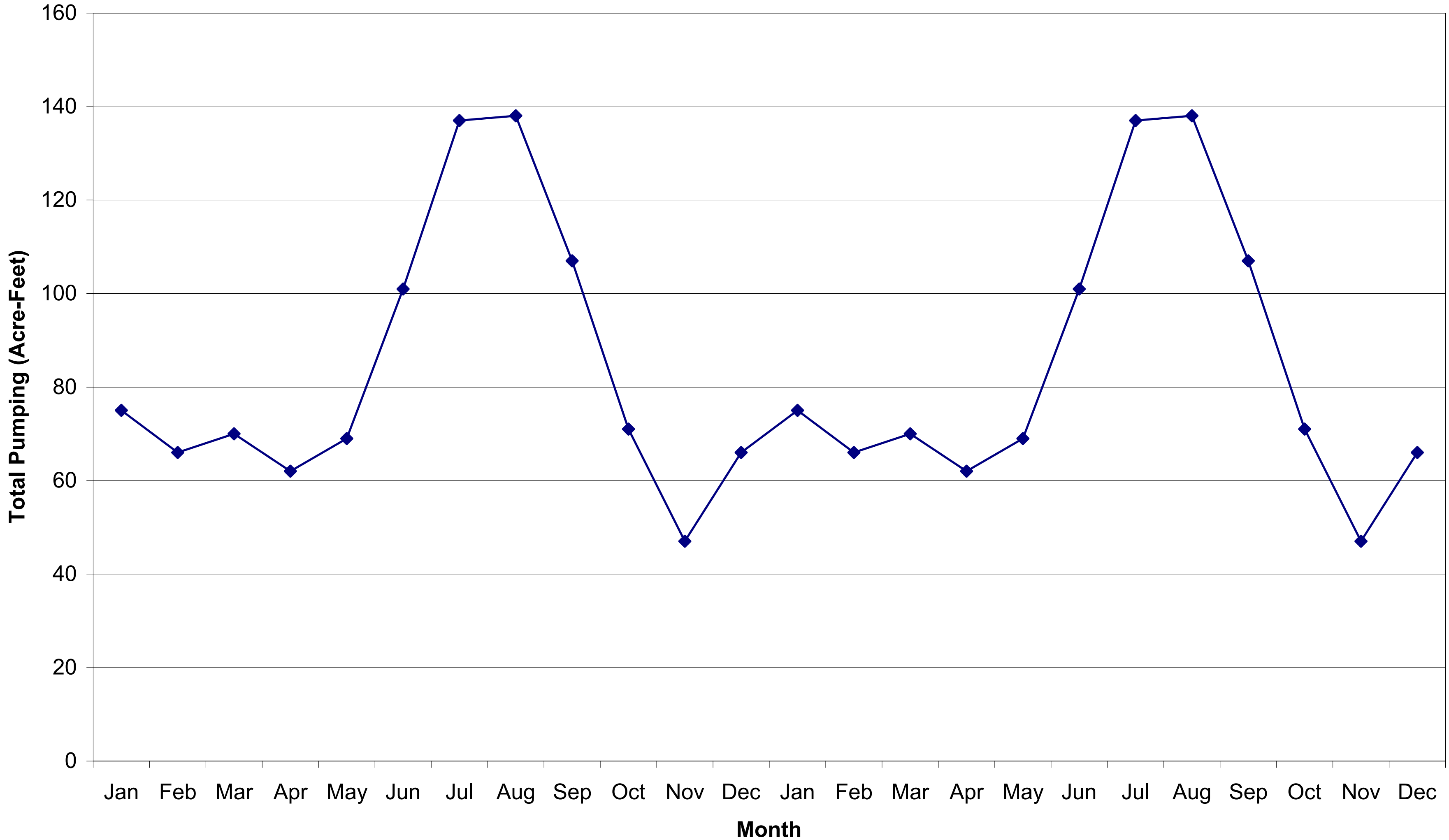
Well	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
District #1	19.66	17.47	18.41	16.22	16.85	24.65	33.07	33.70	25.58	17.16	11.86	17.16	251.78
District #2	17.14	15.23	16.05	14.14	14.69	21.49	28.83	29.38	22.30	14.96	10.34	14.96	219.50
District #3	6.05	5.38	5.66	4.99	5.18	7.58	10.18	10.37	7.87	5.28	3.65	5.28	77.47
District #5	20.16	17.92	18.88	16.64	17.28	25.28	33.92	34.56	26.24	17.60	12.16	17.60	258.24
Mutual #1	6.00	5.00	5.50	5.00	7.50	11.00	15.50	15.00	12.50	8.00	4.50	5.50	101.00
Mutual #2	6.00	5.00	5.50	5.00	7.50	11.00	15.50	15.00	12.50	8.00	4.50	5.50	101.00

The target monthly pumping rates for the initial simulation for determining the sustainable yield using existing and new wells are shown in Table 6-5. These monthly pumping rates represent the estimated demands at ultimate buildout of Squaw Valley, including the projected buildout demands of the District and the Mutual of 1,605 af and 202 af, respectively. Pumping for golf course irrigation and snow making by the Resort at Squaw Creek was assumed to remain constant at 261 af per year.

Table 6-5. Target Monthly Pumping for Buildout Simulation (af)

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
172	126	133	117	134	204	265	270	211	136	131	169	2,068

Figure 6-1. Maximum Monthly Pumping Rates
Existing District and Mutual Wells - Base Case Simulation



UPDATED SUSTAINABLE YIELD ANALYSIS

Monthly pumping rates were determined for each simulation and tables created as input data. The updated model was then run. The resulting water levels in each of the municipal wells were compared to the minimum acceptable water levels. If the levels were found to be above the minimum acceptable elevations for all wells, the pumping rates were increased. Conversely, if the water levels were found to be below the acceptable elevations, the pumping rates were decreased. The sustainable yield was then determined as the maximum annual quantity of pumping that can occur under the critical hydrologic conditions.

Maximum Pumping of Existing Wells

A series of groundwater model simulations using the updated model were conducted to estimate the basin's sustainable yield using just the existing production wells. The simulations used the hydrology of 1994 to simulate critically dry conditions. Two consecutive critically dry years were simulated in each run. The initial estimates of monthly pumping rates using only existing wells reflected the maximum monthly yield of each existing production well. These rates were shown on Table 6-4.

As with the original groundwater model, the maximum pumping rates shown on Table 6-4 could not be sustained during two consecutive critically dry year. A series of simulations were then performed to determine the maximum acceptable pumping rates. Each simulation maintained the relative pumping distribution used in the base simulation, but the total amount of water pumped was reduced.

The simulation that maintained water levels near the minimum acceptable levels established on Table 6-1 for two consecutive critically dry years simulated pumping at 70% of the base case simulation. This corresponds to an annual yield of approximately 706 af. Although this number is lower than previous estimates of sustainable yield, it is supported by recent conversations with District staff. During the late summer of 2002, the District lowered the pumping rate on District Well#2 to prevent the water level from falling below the minimum acceptable level. This happened during a relatively dry year, while the District was on course to pump between 600 and 700 af.

It is worth noting that the minimum acceptable water level of District Well#2 is the controlling factor on the amount of water that can be extracted during a critically dry year. District Well#2 has the highest minimum acceptable water level of all the existing wells, and it is the first well to be impacted by a lowered water table. If the minimum acceptable water level of District Well#2 could be lowered, more water could be pumped.

Maximum Pumping of Both Existing and New Wells Under Previously Used Assumptions

A series of groundwater model simulations were conducted to estimate the basin's sustainable yield using both existing and new wells. As with the previous simulations, two consecutive critically dry years were simulated in each run. The 1994 hydrology was used to simulate the hydrology of a critically dry year.

The initial estimate of additional wells that could be added to the existing wells was derived from the Draft Squaw Valley Groundwater Development & Utilization Feasibility Study Report (West

Yost & Associates, 2001). Based on work conducted for that report, the following modifications were initially made to the basin pumping:

- Pumping from the Resort at Squaw Creek Wells 18-2 and 18-3 was increased to take advantage of the unused capacity of these wells.
- Pumping from the Mutual Well#1 was transferred to the Condo Well
- Additional pumping was added at the 4th Fairway well
- Pumping from District Well#3 was transferred to a portion of the unused capacity in Well 18-3
- Two new wells were added at the western end of the Valley

Initial simulations showed that the basin could not sustain all of the pumping. One new well was eliminated from the western end of the basin, and the pumping rate of the other new well was adjusted to minimize impacts to existing wells. The final pumping rates for the sustainable yield using both new and existing wells is shown in Table 6-6.

Table 6-6. Maximum Pumping Rates for Simulation with Both Existing and New Wells (af)

Well	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Totals
Condo	5.10	4.25	4.68	4.25	6.37	9.35	13.18	12.75	10.63	6.80	3.83	4.68	85.85
Mutual #2	4.20	3.50	3.85	3.50	5.25	7.70	10.85	10.50	8.75	5.60	3.15	3.85	70.70
18-1	1.70	0.00	0.00	0.00	1.70	1.70	1.70	1.70	1.70	1.70	1.70	1.70	15.30
18-2	17.04	4.57	4.81	4.24	14.61	18.35	20.55	20.71	18.59	13.84	15.00	16.39	168.70
18-3	35.25	18.12	19.19	16.52	17.06	37.05	45.58	46.65	37.12	17.59	42.26	47.59	379.97
District #1	13.76	12.23	12.89	11.36	11.79	17.25	23.15	23.59	17.91	12.01	8.30	12.01	176.25
District #2	12.00	10.66	11.23	9.90	10.28	15.04	20.18	20.56	15.61	10.47	7.24	10.47	153.65
District #3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
District #5	14.11	12.54	13.22	11.65	12.10	17.70	23.74	24.19	18.37	12.32	8.51	12.32	180.77
4th Fairway	5.70	3.78	4.36	3.71	4.45	6.55	8.73	8.91	6.72	4.45	4.22	5.61	67.19
WELL 4R	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
New-1	4.18	3.71	3.91	3.45	3.58	5.24	7.03	7.16	5.44	3.65	2.52	3.65	53.50
Totals	113.04	73.37	78.14	68.58	87.19	135.93	174.69	176.72	140.83	88.43	96.72	118.26	1,351.89

As shown on Table 6-6, the sustainable yield of the basin using both new and existing wells is approximately 1352 af annually. Of this, 261 af are assumed to be used by the Resort at Squaw Creek for golf course irrigation and snow making, leaving 1,091 af annually available for potable use by the District and the Mutual.

Maximum Pumping of Both Existing and New Wells Under New Assumptions

As stated at the beginning of this section, we conducted an additional sustainable yield analysis by relaxing two important assumptions contained in the previous analysis. This third analysis again looked at the sustainable yield available from both existing and new wells. The significant assumptions changed during the third analysis included:

1. The perforations in District Well#2 lowered by 15 feet. This could be accomplished by sleeving the existing well or reconstructing the well since it is over 40 years old. This will lower the acceptable simulated water level by 15 feet. As shown in Table 6-1, the previously assumed minimum water level in District Well#2 was 6177 feet msl. The new assumption yields a minimum water level of 6162 feet msl for District Well#2.

2. Wells can provide different percentages of water throughout the year. In the previous analysis, the pumping rates for all wells were increased and decreased by the same amount each month. A 20% increase in total pumping was attained by all wells increasing production by 20%. In the current analysis, all wells can be increased or decreased independently.

As with the second analysis described above, two consecutive critically dry years were simulated in each run. The 1994 hydrology was used to simulate the hydrology of a critically dry year.

Following a series of iterative simulations, the following changes were made to the pumping distribution:

- Two new wells were installed at the western end of the basin. One well is approximately at the Plumpjack well location. The second well is located between the first well and District Well#2. We assumed these two wells are hydraulically similar to District Well#2. The maximum pumping rate for each of these wells in the buildout simulation is 112 gallons per minute.
- Pumping from the Resort at Squaw Creek Wells 18-2 and 18-3 was increased to take advantage of the idle capacity of these wells.
- Pumping from the Mutual Well#1 was transferred to the Condo Well
- Additional pumping was added at the 4th Fairway well
- Pumping from District Well#3 was transferred to a portion of the idle capacity in Well 18-3
- Pumping from District Well#1 was increased relative to other District wells.
- Pumping from District Well#5 was reduced relative to other District wells.
- District Well#3 was been taken out of service.

The final pumping rates for the sustainable yield using both new and existing wells under the relaxed assumptions is shown in Table 6-7.

Table 6-7. Maximum Pumping Rates for Simulation with Both Existing and New Wells (Relaxed Assumptions) (af)

Well	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Totals
District #1	16.51	14.68	15.46	13.63	14.15	20.70	27.78	28.30	21.49	14.41	9.96	14.41	211.48
District #2	12.00	10.66	11.24	9.90	10.28	15.04	20.18	20.55	15.62	10.48	7.24	10.48	153.67
District #5	10.58	9.41	9.91	8.74	9.07	13.27	17.81	18.14	13.78	9.24	6.38	9.24	135.57
Mutual #1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Mutual #2	4.20	3.50	3.85	3.50	5.25	7.70	10.85	10.50	8.75	5.60	3.15	3.85	70.70
18-1	1.70	0.00	0.00	0.00	1.70	1.70	1.70	1.70	1.70	1.70	1.70	1.70	15.30
18-2	17.04	4.57	4.81	4.24	14.61	18.35	20.55	20.71	18.59	13.84	15.00	16.39	168.70
18-3	35.25	18.12	19.19	16.52	17.06	37.05	45.58	46.65	37.12	17.59	42.26	47.59	379.98
Condo	5.10	4.25	4.68	4.25	6.37	9.35	13.18	12.75	10.63	6.80	3.83	4.68	85.87
4th Fairway	5.70	3.78	4.36	3.71	4.45	6.55	8.73	8.91	6.72	4.45	4.22	5.61	67.19
New1	9.19	8.17	8.61	7.58	7.88	11.52	15.46	15.75	11.96	8.02	5.54	8.02	117.70
New2	9.19	8.17	8.61	7.58	7.88	11.52	15.46	15.75	11.96	8.02	5.54	8.02	117.70
Totals	126.46	85.31	90.72	79.65	98.70	152.75	197.28	199.71	158.32	100.15	104.82	129.99	1523.86

As shown on Table 6-7, the sustainable yield of the basin using both new and existing wells is approximately 1,524 af annually. Of this, 261 af are assumed to be used by the Resort at Squaw Creek for golf course irrigation and snow making, leaving 1,263 af annually available for potable use by the District and the Mutual.

Conclusions

The maximum pumping rates of existing wells were applied to all six existing municipal production wells in the Squaw Valley Basin. The monthly pattern of pumping was determined based on the historical monthly demand in Squaw Valley. The maximum amount of sustained production from existing wells during a drought is 706 af per year. This is 167 af greater than the combined pumping by the District and the Mutual of 539 af during the year 2000

The simulations suggest that it will be difficult for the District to meet the estimated buildout demands during critically dry years. This is primarily because of the lack of summertime recharge from Squaw Creek during drought years. Groundwater model simulations suggest that approximately 73 percent of the buildout demand can be supplied by groundwater during a critically dry year. This is equivalent to an annual sustainable supply of 1,524 af per year. These results assume lowering the perforations in District Well #2 and a well efficiency of 70 percent; lower well efficiencies may result in less available supply. Of this total sustainable yield, pumping by each valley entity based on the assumptions in the simulations is shown in Table 6-8. As an alternative, the yield could be distributed based on the assumption that the supply would be equally shared based on the ratio of sustainable yield (1,524) divided by the buildout demands (2,068). This distribution of the sustainable yield is shown in Table 6-9.

Table 6-8. Sustainable Yield Analysis – Assumed Pumping Rates, (af)

Pumping Entity	Annual Pumping Amount
Squaw Valley Public Service District	1,091
Squaw Valley Mutual Water Company	172
Resort at Squaw Creek	261
Total Sustainable Yield	1,524

Table 6-9. Sustainable Yield Analysis – Equally Reduced Pumping Rates, (af)

Pumping Entity	Annual Pumping Amount
Squaw Valley Public Service District	1,183
Squaw Valley Mutual Water Company	149
Resort at Squaw Creek	192
Total Sustainable Yield	1,524

SECTION 7. ALTERNATIVE WATER SUPPLIES

Additional water production capacity needs were identified in Section 5 – Estimate of Ultimate Water Production Requirement. The supplemental production capacity required to meet District maximum day demands at buildout is about 1,600 gallons per minute.

The District's estimated annual demand from the groundwater basin at buildout is 1,605 af. The sustainable yield of the groundwater basin was estimated to be about 1,524 af per year in Section 6 - Groundwater Model Simulations to Estimate Sustainable Yield. The District's portion of the sustainable yield has been assumed to be about 1,100 af as was shown in Table 6-8. The District's horizontal wells in an average year produce about 30 to 45 af. They will probably produce about half that amount during a drought year. Therefore, the total supply available to the District is about 1,120 af. A supply of 1,120 af will provide about 69 percent of the District's buildout demand during a critically dry year. The remainder must be supplied from other sources, or the demand during a critically dry year be reduced by about 31 percent through conservation or by limiting development.

The District has recently passed a water conservation ordinance that includes the curtailment of demands during a critically dry year. The ordinance requires that normal demands be reduced by at least 20 percent during a drought emergency. Increased demand management needs to be part of the final solution for meeting future demands in the valley.

Facilities must be identified to supply both the projected annual and maximum day demands. Alternative water supplies using sources inside the valley and outside the valley were evaluated. Alternative water supplies identified and evaluated as part of this study are:

- Additional Squaw Valley Wells
- Springs East of the Truckee River
- Truckee River Wells
- Alpine Springs County Water District

Each of these alternative supplies have been investigated and evaluated in terms of their feasibility of meeting some if not all of the increased water supply needs. These alternative water supplies are described below.

ADDITIONAL SQUAW VALLEY WELLS

Four to six wells located within Squaw Valley are required to supplement the production from existing wells to meet projected maximum day demands at buildout. It is assumed that each well will have a production capacity of between 100 to 400 gpm. Table 7–1 provides a list of test holes and wells that have been considered as a potential location for a new well. The table also includes a brief summary of the information known about the potential to complete a well at that location. Included is information on the casing size, drilling depth, estimated production potential, water quality, and comments regarding whether to continue to consider construction of a well at that site. The locations of the wells/test holes are shown on Figure 7-1.

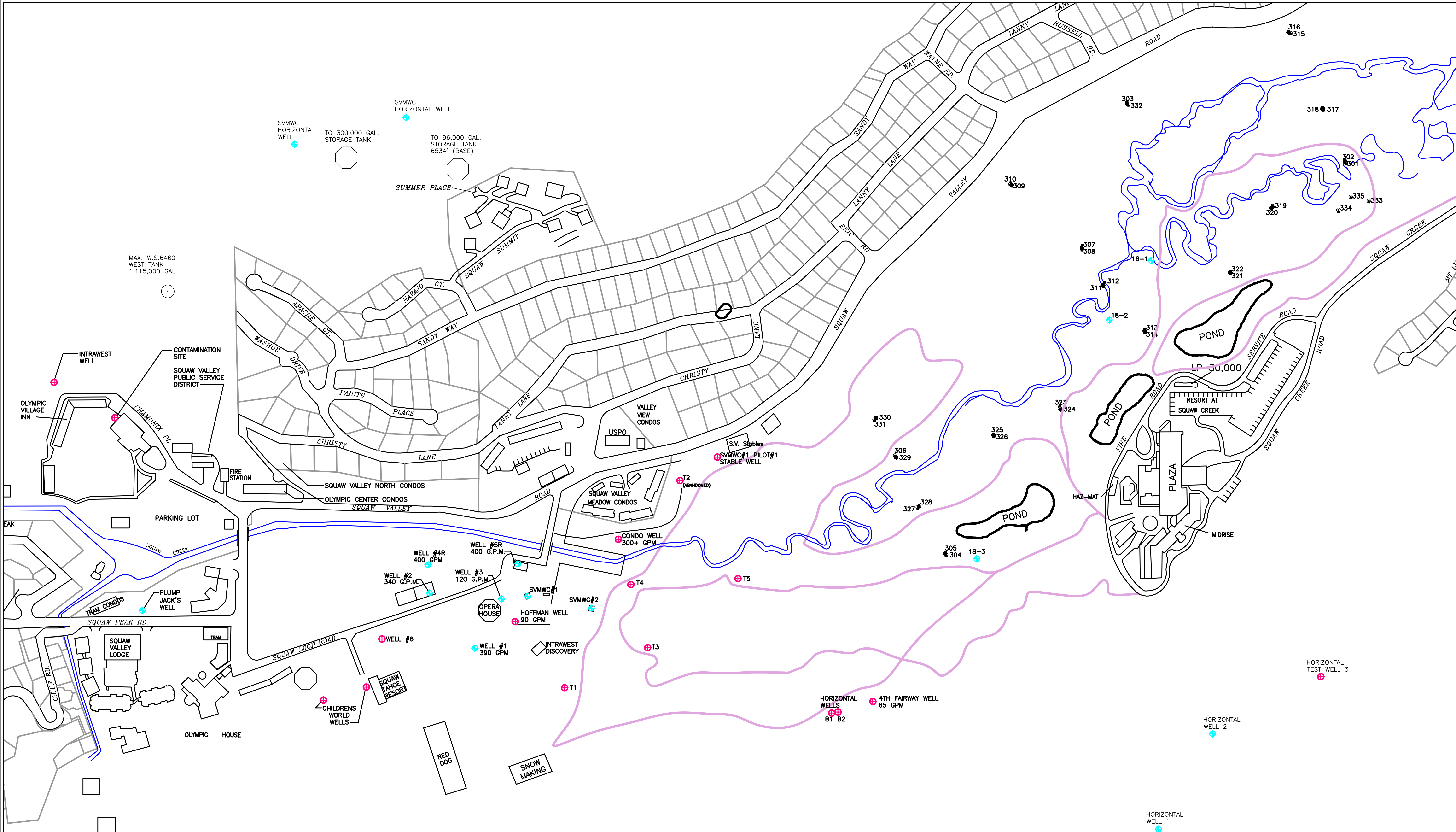


Figure 7-1

Squaw Valley Public Service District

TEST HOLE & WELL LOCATIONS

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The test holes and wells identified in Table 7-1 provide the best information on the potential for developing additional water supply in the valley. These test holes were sited after review of previous drilling and geologic investigations in an attempt to find the best location for new supplies. All of these test holes are in the western end of the valley, because this area has proven to be the have the best water quality and production capability. There have been several wells drilled in the eastern half of the valley that have proven to be small producers with lower quality water, such as the Poulsen Well at Russel Road and the Resort Well 18-1. The water producing aquifers in the eastern half of the valley are typically comprised of finer sediment materials and the produced water is of lower quality than from the western valley wells. In addition recent exploration of alluvial and bedrock targets in the northwest and southeast ends of Squaw Valley have not located sufficient production capacity to justify construction of production wells in these areas.

This generalization of where the better producing wells with better water quality have been located was taken into account in evaluating where to recommend new well construction. Also, the information obtained from the previous test drilling and pumping also influenced the recommendations on where the next wells should be drilled. There is a need to pump water from areas further to the east to better utilize the overall basin. However, the priority for drilling new wells should be to rely on proven locations in the west central area of the valley floor first, and if these locations prove to be unsuccessful, look to other areas to perform new test drilling and pump testing. In addition, the groundwater development supply alternative is flexible in terms of timing and locations for new wells. If some of the identified well locations are not successful in providing the needed supply, other locations will then need to be investigated and test holes constructed.

Table 7-1. Potential New Well Sites

Well Name	Casing Size and Drilling Depth	Estimated Production Capacity, gpm	Water Quality	Comments on Site Desirability
District Well 4RII	12”, 71’	500+/-	Meets standards	Continue to pursue – assume no treatment needed
District Well 6	12”, 61’	250+/-	High in Fe, Mn	Too shallow to be permitted
Condo Well	14”, 120’	300+/-	High in Fe, Mn	Continue to pursue – treatment needed
4th Fairway Well	6”, 86’	65	High in Fe, Mn	Continue to pursue – treatment needed
Hoffman Well	4”, 157’	90	Very High in Fe, Mn	Poor quality and too close to other wells
Stables Well	4”, 72’	200+/-	High in Fe, Mn	Poor quality and too shallow
Test Hole 1	4”, 110’	Low-Medium	Meets standards	Too close to existing wells – could be a replacement well site
Test Hole 2	4”, 32’	Not pumped	Not sampled	Too shallow – drilling abandoned

Table 7-1. Potential New Well Sites, Cont…

Well Name	Casing Size and Drilling Depth	Estimated Production Capacity, gpm	Water Quality	Comments on Site Desirability
Test Hole 3	4", 143'	Low	High in Fe, Mn	Too close to MWC Well 2
Test Hole 4	4", 159'	High	Very High in TDS, Fe, Mn, As	Very poor quality-should be tested for more information on poor water quality source
Test Hole 5	4", 166'	High	Very High in TDS, Fe, Mn	Very poor quality-should be tested for more information on poor water quality source

In addition to these wells and test holes, two other sites were identified in the development of the groundwater model and estimate of the sustainable yield. These two additional sites are in the western end of the parking lot in an area that will probably have good production capability and is somewhat removed from the cluster of existing wells in the eastern portion of the parking lot. Their locations are only approximate. Additional research to define property purchase requirements for their well sites, and exploratory drilling is needed to finalize a decision to locate a well at these or nearby sites. It has been assumed that wells drilled at these locations will be good producers with pumping capacities similar to the District's Wells 4R and 6. For this study, it is assumed that the water produced from these wells will be high in iron and manganese, similar to Well 6, and will need to be treated.

There have been a number of groundwater contamination sites in the western parking lot area and there is a concern of future groundwater contamination in this area. The historical contamination sites have been identified and clean up of the contamination has been completed or is in progress on all known sites. The production well capture zone analysis performed for the Source Water Assessment shows that there is relatively fast movement of groundwater in the western basin. Figures 3-3, 3-4 and 3-5 show the capture zone of the existing production wells and it should be noted that the area supplying the wells does not include the basin to the east underlying the golf course. Most of the historical contamination has been eliminated from the western basin area through treatment and travel through the basin. It is anticipated that there will be a diligent groundwater protection program undertaken by the District and Mutual in the future including review of potential contamination activities and ongoing water quality monitoring. This program coupled with having a treatment facility that can be augmented for hydrocarbon removal provide safeguards for the use of additional wells in the western parking lot area.

Additional horizontal wells are not considered in this supply alternative other than incorporating the District's existing horizontal wells near the 4th Fairway Well into the supply system. The horizontal wells on the valley edges have proven to supply very little water for the District and Mutual, tens of gpm versus hundreds of gpm in the valley. The close proximity of the District's two horizontal wells to the 4th Fairway Well site would require only a very little amount of plumbing to include the production of 30 to 40 gpm into the District's supply system when the 4th Fairway Well is developed.

The sustainable yield analysis also identified the need for additional pumping in the golf course area to increase the capture of the sustainable yield of the basin. For this study it has been assumed that the idle capacity of the Resort at Squaw Creek Wells 18-2 and 18-3 could be used to help meet future District needs. Well 18-2 is reported to produce about 150 gpm, and Well 18-3 produces 265 gpm. The water from these wells is high in iron and manganese and will need to be treated. These wells could continue to meet Resort golf course irrigation and snow making demands, but idle capacity could also be used by the District. An agreement for the use of these wells will need to be developed between the District and the Resort. It is assumed that about one-half, or 200 gpm, of the pumping capacity of the two wells could be made available for the District to use. In the future, the construction of new wells in the vicinity of Wells 18-2 and 18-3 could also be explored as replacement wells for 18-2 and 18-3 or new wells for the District. Exploration in this area for new wells would only be undertaken after additional operational information on the sustained pumping capacity from Wells 18-2 and 18-3 is obtained and found to be adequate. It is assumed that all water pumped in this area of the valley will need treatment for high levels of iron and manganese. Increased pumping from wells in the golf course/meadow area may create opportunity for increased iron and manganese concentrations in Wells 18-2 and 18-3. It is unlikely that the additional pumping will result in higher iron and manganese concentrations in the other production wells to the west.

The pumping capacity from the wells that will continue to be considered as elements of the Squaw Valley Groundwater Alternative are summarized in Table 7-2. Their total pumping capacity is 1,600 gpm, which equals the amount of additional supply required at buildout. It has been shown in Section 6 that these wells in combination with the existing District wells, can produce the sustainable yield of the basin without adversely impacting water levels.

Table 7-2. Wells Included in Squaw Valley Groundwater Supply Alternative

Well Name	Assumed Pumping Capacity, gpm	Treatment Required
Well 4R II	400	No
Condo	300	Yes
4th Fairway Well	100	Yes
New Well 1	300	Yes
New Well 2	300	Yes
Wells 18-2 and 18-3	200	Yes
Total Pumping Capacity	1600	

Wells to be included in the Squaw Valley Groundwater Supply Alternative are shown on Figure 7-2. The water from all the new wells, except Well 4R II, must be treated to remove iron and manganese. It is assumed that supply from the horizontal wells located near the 4th Fairway well will be incorporated into pipe systems to bring the supply to the treatment plant.

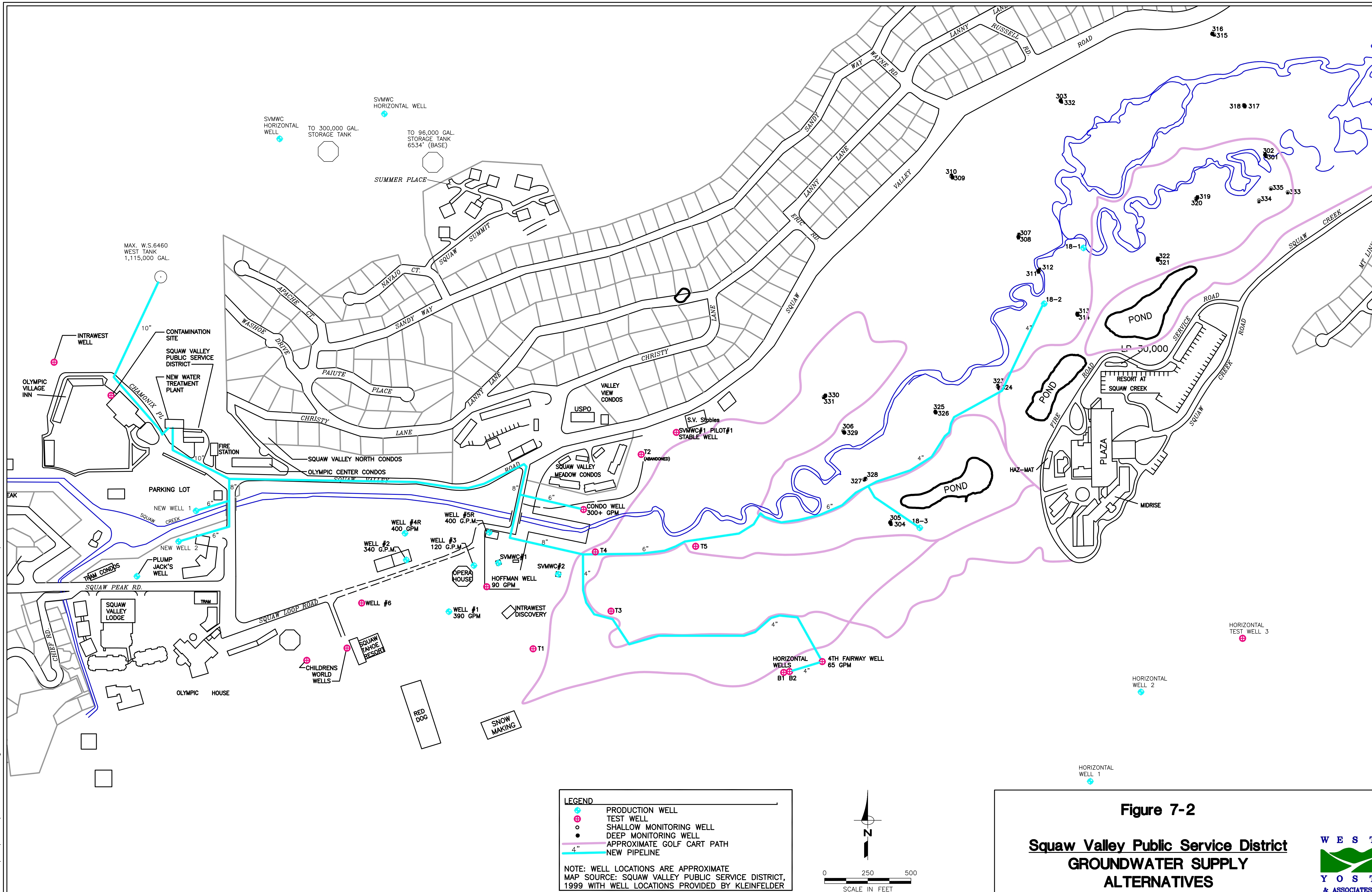

Figure 7-2

Squaw Valley Public Service District
GROUNDWATER SUPPLY ALTERNATIVES

Pipelines to deliver water to the proposed treatment plant are also shown on Figure 7-2. An iron and manganese removal treatment plant has been evaluated and the preferred site for the plant based on a study performed by the District is at the existing District office site, just behind the existing office building. Section 8 provides the details of the evaluation and recommendations on the proposed treatment plant. The treatment plant would have a nominal capacity of 1,400 gpm or 2 mgd.

Routes for new pipelines to bring the water to the treatment plant from Wells 18-2 and 18-3 have been identified from review of the locations of golf cart paths, existing District pipeline easements, and other course features to minimize disruption to the course during their construction and for access for maintenance. The connecting pipeline routes for the rest of the wells have been selected to stay within public rights-of-way as much as possible. The pipe sizes have been selected to maintain a maximum velocity of five feet per second in the pipeline. A minimum pipe size of four inches was used. Table 7-3 presents a summary of the pipelines required for this alternative.

Table 7-3. Pipelines for Squaw Valley Groundwater Supply Alternative

Pipe Size, inches	Length, feet
4	3,300
6	3,500
8	2,100
10	1,700

The length of 10-inch pipeline shown in Table 7-3 includes 1,200 feet of pipe from the water treatment plant to the 1.1 MG storage tank.

SPRINGS EAST OF THE TRUCKEE RIVER

A local supply could be developed using the springs located about 4,500 feet southeast of the intersection of Squaw Valley Road and Highway 89. There are several springs located in this area at the contact between the quaternary volcanic latite and cinders and the underlying tertiary, andesitic rocks in Section 34 as shown in Figure 7-3. Water from these springs has been used to serve some properties along the river, but that stopped because of supply reliability and water quality concerns. These properties are currently being served by the Tahoe City Public Utility District (TCPUD) with water supplied by the District. The spring supply was investigated by TCPUD in 1981 and the results presented in the report, “Master Plan for the Tahoe City Public Utility District” by Culp, Wesner and Culp, 1981. It was found that there is limited recharge in this area because of the impaired vertical permeability within the overlying volcanic rock. In addition, TCPUD used the area above the springs for many years to dispose of their primary treated wastewater effluent. DOHS has reviewed the use of this supply and has serious concerns about the water quality from these springs. District staff have recently visited the old spring collection boxes and found them to be in disrepair and producing a very low flow. It did not appear that these springs would provide a significant supply of water to the District, and their ability to be permitted by the DOHS without treatment is unlikely. This supply alternative has been dropped from further consideration for these reasons.

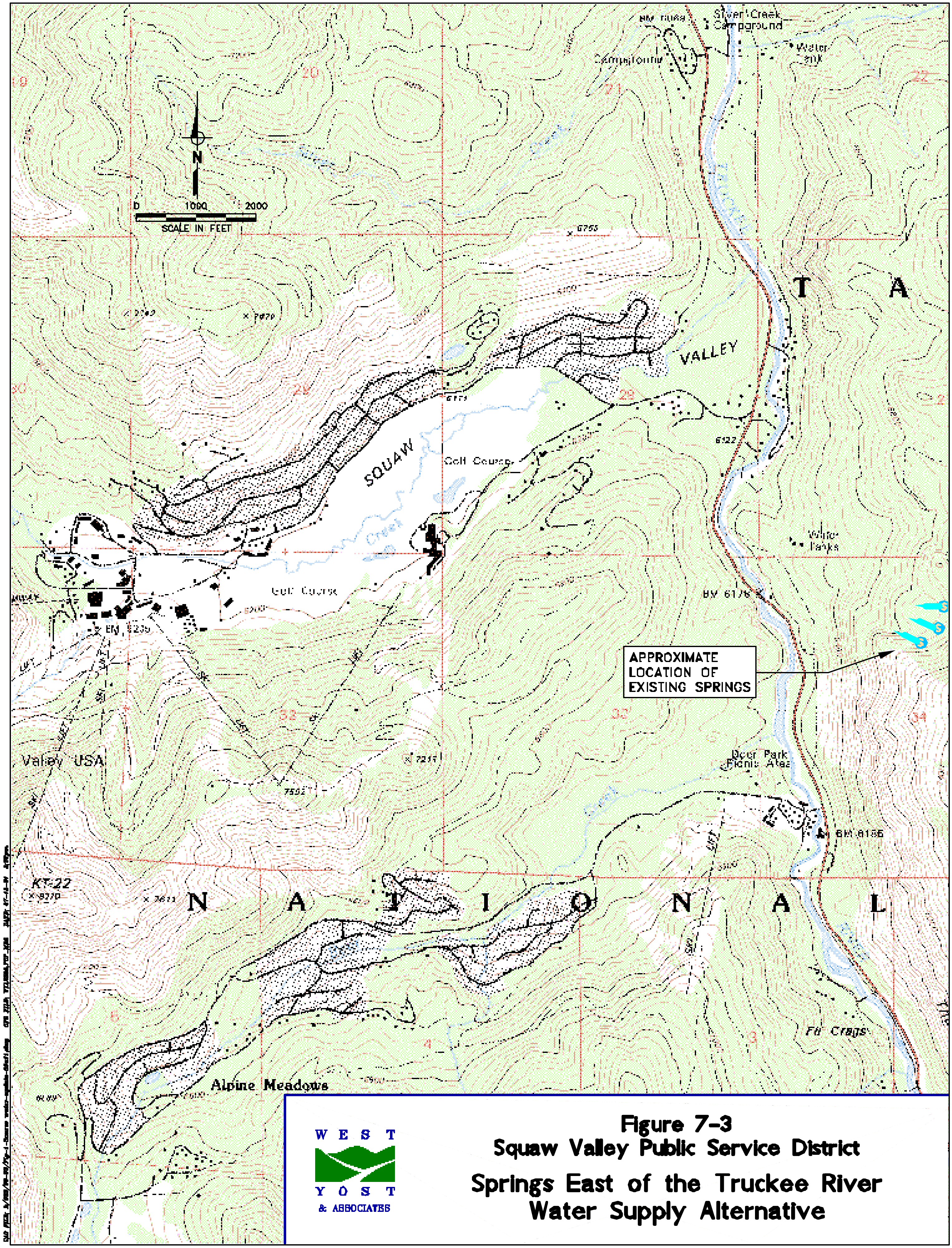

Figure 7-3
Squaw Valley Public Service District
Springs East of the Truckee River Water Supply Alternative

TRUCKEE RIVER WELLS

The District may be able to divert surface water from the Truckee River. The Truckee-Carson-Pyramid Lake Water Rights Settlement Act of 1990, Public Law 101-618, includes the settlement of water rights claims on the Truckee River between the State of California, the State of Nevada and the Fallon Paiute-Shoshone Indian Tribe.

The settlement act provides for up to 32,000 af per year to be used in the Truckee River Basin in California. Of that amount, up to 10,000 af per year of surface water can be diverted in California. To date there is a substantial amount of the 10,000 af of surface water diversion from the river that is still available for use. Only about 3,000 to 4,000 af of the surface water rights in California are currently being used.

The Truckee River Operating Agreement (TROA) is currently being negotiated and will ultimately establish the operating rules for the streamflow diversion and reservoir operation in the Truckee River basin to satisfy the exercise of agricultural and municipal and industrial water rights. The TROA should be finalized by late 2001, and then an EIS/EIR will be prepared on the proposed operating agreement. The EIS/EIR process is expected to take about two years to complete. To use surface water from the Truckee River, the District must file a water rights application with the State Water Resources Control Board (SWRCB), Division of Water Rights. The SWRCB has a policy that they are currently accepting applications, but they will not review or act upon any applications prior to the finalization of the TROA and the adoption of the EIS/EIR.

In drought conditions spanning several years, releases from Lake Tahoe could be eliminated in the summer when the lake level drops below the natural rim elevation. Hydrologic studies for the TROA show that surface water could be available for diversion by the District during over 90 percent of the time. However, diversion of water during droughts that are sustained for several years, such as the 1988 to 1992 drought, would probably be curtailed. Use of this source of supply would have to be coordinated with other water rights holders during extended dry periods.

For this alternative it is expected that the application would be for a maximum diversion rate of 1,600 gpm and an annual total diversion of about 920 af. The annual total is the difference between the ultimate buildout demand of 1,605 af, and the Squaw Valley basin sustainable yield of 685 af based on use of the maximum pumping rate of the District's existing wells as determined in Section 6. The application would be for surface water, but it is envisioned that the water would be diverted through the use of wells or Ranney collector system located near the Truckee River. Wells would minimize the impact to the river, as compared to a surface diversion, and provide a more consistent quality of water, filtered through the streambed sands and gravels.

Two wells or a caisson with radial collector pipes and two pumps located adjacent to the Truckee River would be needed to supplement projected maximum day demands, assuming each well delivers 600 to 800 gallons per minute. The wells would likely be considered by DOHS to be under the influence of surface water and require water treatment. This water supply alternative would include wells or collector system, pipeline conveyance to a water treatment plant, and pumping into the District water distribution system. Project components are listed in Table 7-4.

Table 7-4. Truckee River Diversion Alternative – Project Components

Item	Number/Size
Wells	
Number	2
Capacity, gpm each	600 – 800
Conveyance Pipeline	
Diameter, inches	10
Length, feet	12,000
Water Treatment Plant	
Pressure filters with UV Disinfection	1
Capacity	2mgd

The location of the proposed facilities comprising this alternative is shown on Figure 7-4. The wells would be located between the Truckee River and State Highway 89, near its intersection with Squaw Valley Road. The alignment for the 10-inch diameter raw water pipeline would follow Squaw Valley Road to the treatment plant site located at the existing District office parcel at the west end of the valley. The water treatment plant would be sized for two mgd and is described in Section 8. Treated water would be boosted to the 1.1 MG storage tank that is located on the mountainside above the District office to provide chlorine contact time. As an alternative, the treatment plant could be built on land at the east end of the valley near the District's park site, and the cost of the long raw water pipeline would be reduced. The treated water would then be pumped to the 0.5 MG tank located above the Resort at Squaw Creek to provide chlorine contact time before entering the distribution system.

ALPINE SPRINGS COUNTY WATER DISTRICT

Alpine Springs County Water District (ASCWD) is located about 1.5 miles south of Squaw Valley as shown on Figure 7-5. ASCWD's water supply facilities include four springs, two wells, and two snow production wells within the Bear Creek Valley. The combined capacity of the springs and wells is estimated to be about 1,100 gallons per minute according to a 1998 Water Audit prepared by ECO:LOGIC Engineering with all facilities operating. The actual capacity varies slightly between summer and winter conditions. The maximum day summer water demand of ASCWD is estimated to be about 400 gallons per minute, leaving as much as 700 gallons per minute of possible idle supply capacity. The bulk of this capacity, if not all, is produced by the snow production wells. It should be noted that ASCWD has made no indication that any water supply is available or that they are willing to sell any water on a long-term basis.

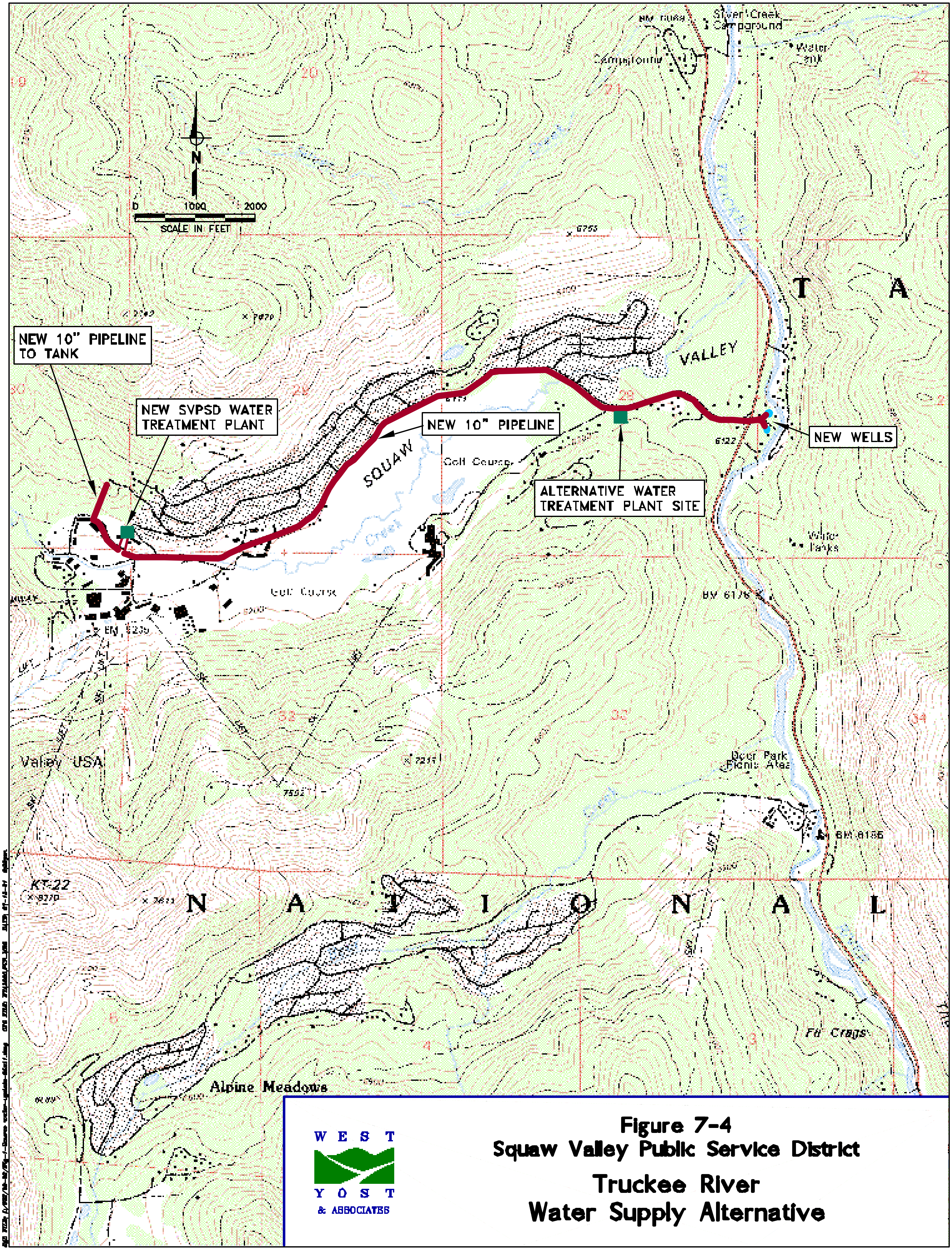

Figure 7-4
Squaw Valley Public Service District
Truckee River Water Supply Alternative

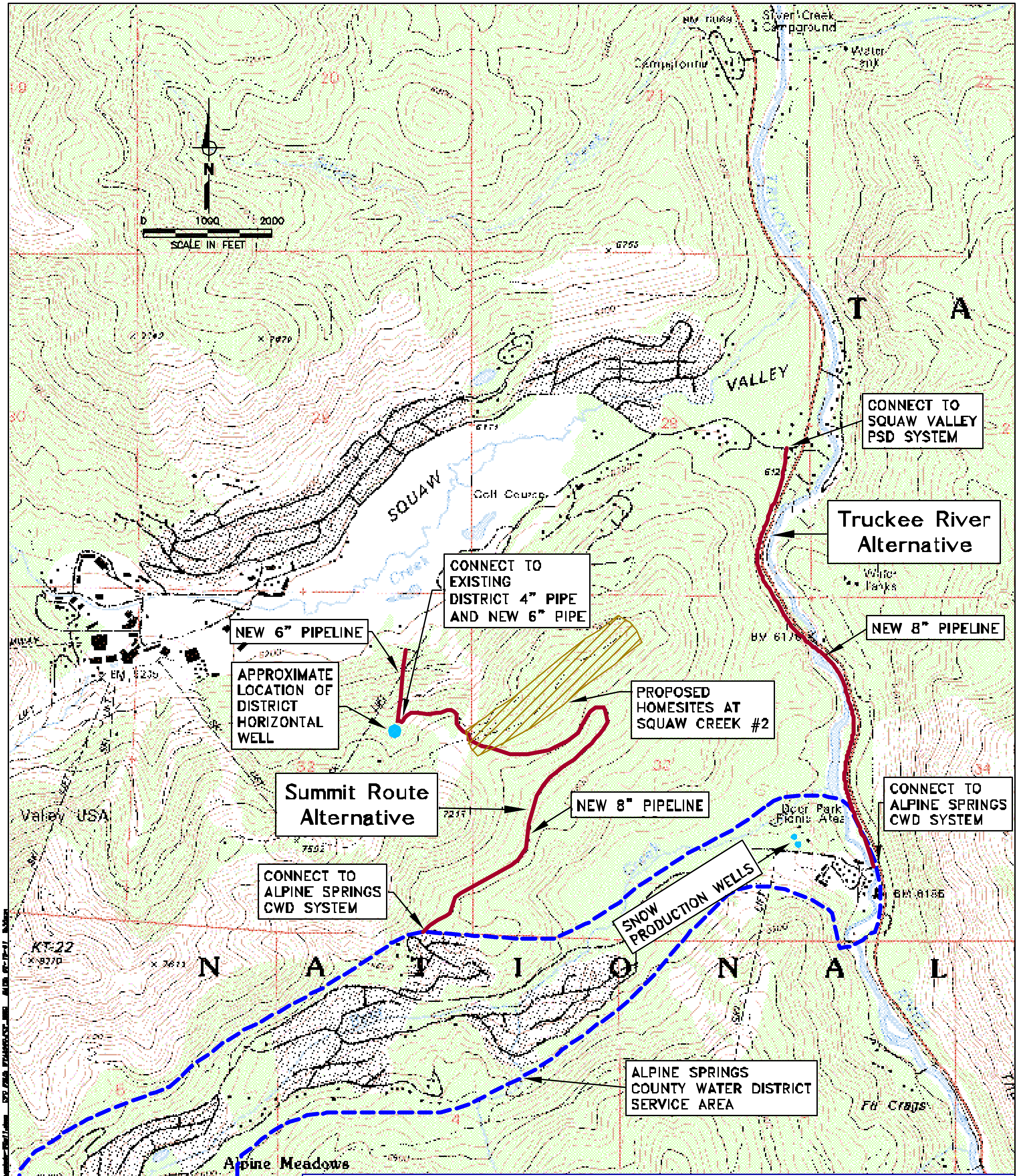

Figure 7-5
Squaw Valley Public Service District

Alpine Springs CWD
Water Supply Alternatives

Snow Production Wells. The snow production wells are used in the late fall and early winter months for snow making by the POWDR Corporation for the Alpine Meadows Ski Area. The snow production wells are not currently used for municipal purposes within ASCWD. The capacity of the two snow production wells, operating at one time, is estimated at between 800 and 1,000 gallons per minute. ECO:LOGIC Engineering estimates the reliable capacity of these wells operating together at 850 gallons per minute, during a 24-hour pumping period, after which a minimum 8-hour recovery period is provided to allow the fractured rock aquifer to recover sufficiently to maintain this pumping capacity. The most economical use of these wells is to operate them during off peak electric demand periods. If the wells are used any time during the peak electric demand period, the peak electric rate applies for the entire month of water pumping.

Water use agreements between the POWDR Corporation and ASCWD allow ASCWD to use water from these wells anytime during the summer (June 1 to September 30) and during any 5 consecutive day periods in the winter (October 1 to May 31). Therefore, it may be feasible for the District to contract with ASCWD to use surplus capacity from the snow production wells during the summer or fall months. The available idle capacity has been estimated by ECO:LOGIC Engineering to be about 700 gallons per minute during maximum day demand conditions. The actual supply depends on how the wells are operated. Given the need for supply recovery of at least 8 hours, after 24 hours of pumping, the available capacity on average would be about 525 gallons per minute. If the wells were operated 12 hours per day during off peak electric use periods, the available capacity would produce about 500,000 gallons per day, or an average supply rate over a 24-hour period of only about 350 gallons per minute. Operation of the wells would need to be coordinated with operation of the Squaw Valley wells and storage facilities to maximize yield of the ASCWD supply when it is available.

Water Use Agreement. Supplemental water is needed by the District in late summer and fall months. In a dry year there could be a conflict with domestic needs of the District and snowmaking needs of the Alpine Meadows ski resort, if snowmaking began before November. Initially, the District may only need supplemental water for a few days each fall, but water needs would increase in the long-term. The District could also use water from the snow production wells during the summer to reduce pumping from the Squaw Valley basin, and extend the available water supply through the year. Water from ASCWD should be considered a supplemental water supply and not a primary supply because any long-term agreements between the districts would most likely allow ASCWD to terminate water deliveries to the District, if the water was needed first for the ASCWD service area customers.

Water Quality. Use of the wells may be restricted by water quality considerations and DOHS permitting requirements. One snow production well has a manganese level of 122 mg/L. This is not a problem for ASCWD if the water is blended with other supplies. However, the snow production wells are not currently permitted for domestic use. Generally, new water supplies may not be blended with existing supplies for purposes of meeting water quality standards. The snow production wells may be considered a new water supply, and blending of the wells into the Squaw Valley water distribution system may not be permitted without treatment. This is an issue that would have to be addressed before more detailed plans are made for using water from ASCWD. The wells will be tested again this winter by ASCWD. The manganese concentration seems to be decreasing with time according to the ASCWD.

Spring Water. At times ASCWD does not use all water from its springs. Excess water flows from these springs into Bear Creek during low water demand periods within the ASCWD service area. It may be possible to capture this excess water in the evening and deliver it to Squaw Valley. However, the amount of supply available from the springs is limited to less than 200 gpm. Development of this water supply for the District could be investigated in the future in the event a permanent water connection is completed between the two districts, and the water is not committed to other uses by ASCWD.

Alpine Springs Water Delivery

Supplemental water from ASCWD snow production wells could be delivered to the District through two possible pipeline alternatives. One alternative would be to extend the water distribution system across the summit between the two districts. Construction of the distribution system to serve proposed development called Homesites at Squaw Creek #2 would extend the ASCWD system to within about 1,000 feet of a 4-inch pipeline that provides water from the District's horizontal well that is located on the mountainside above the Resort at Squaw Creek. This 4-inch pipeline is connected to the District's water distribution system. Another alternative would be to construct a new water line between the two districts along the Truckee River/Highway 89 corridor. The estimated length of the pipeline would be 7,500 to 8,000 feet, depending on the actual alignment selected.

Homesites at Squaw Creek #2 Connection. ASCWD has a network of 6-inch water lines that may be extended next year to serve the proposed subdivision, Homesites at Squaw Creek #2. This proposed project includes 30 single-family residential homesites on about 90 acres of land located on the summit between the two districts as shown on Figure 7-5. The project is within both the Squaw Valley General Plan and the Alpine Meadows General Plan. If Squaw Creek #2 project develops, it would be possible to connect the District and ASCWD water distribution systems by construction of about 1,000 feet of pipe. A pressure reducing station would be required to reduce pressures from the ASCWD system at the connection to the District pipeline. To provide up to 700 gpm of idle production capacity to the District, an 8-inch pipeline is required from existing ASCWD facilities to the Squaw Valley horizontal well site. It is unknown what size pipe and booster pump capacity would be constructed for the Squaw Creek #2 development. It probably would be less than the 700-gpm capacity needed for this alternative. If this subdivision proceeds, the facilities could be oversized to accommodate the 700-gpm delivery to the District.

The new 8-inch water line could connect to an existing 4-inch water line, which currently delivers from 20 to 50 gpm to Squaw Valley. The capacity through the existing 4-inch pipeline is limited to about 300 gpm. Therefore, the 4-inch waterline would only be able to deliver an additional 250 to 280 gpm to Squaw Valley from ASCWD. This capacity is insufficient to capture the available reserve supply capacity of 700 gpm from ASCWD. A parallel 6-inch water line 1,200 feet long would be constructed from the connection point to deliver up to 700 gpm to the District distribution system.

The Squaw Creek #2 project would extend the ASCWD water distribution system about 5,000 feet with the addition of a new 6-inch water line and pump station that lifts the water an estimated 300 feet in elevation. It is assumed that a new water storage tank would be provided above the Squaw Creek #2 service area for fire and emergency supply.

Project components for a connection to ASCWD that would supply up to 700 gpm are listed below. This is based on development of Squaw Creek #2 and oversizing the pump station and pipeline to provide a new 8-inch water line to connect to the District system. If Squaw Creek #2 is not constructed, an additional 5,000 feet of 8-inch pipe, and a booster pump station, would need to be constructed for this alternative.

Table 7-5. Squaw Creek #2 Connection- Project Components

Item	Number/Size
Conveyance Pipeline	
6 inches	1,200
8 inches	6,000
Booster Pump Station	
Additional Capacity	700 gpm
Meter and PRV Station	
Diameter, inches	4

In addition to the above listed facilities, the cost of water supply from the summit connection would include ASCWD normal operating costs, and pumping costs from the snow production wells to the connection between the two districts. The lift is approximately 600 feet. There may also be costs for improving the existing water distribution system to the connection to Squaw Creek #2.

Truckee River/Highway 89 Corridor Alternative Route. Supplemental water from ASCWD snow production wells could be delivered to the District through a new water line between the two districts along the Truckee River/Highway 89 corridor. The estimated length of the pipeline is 7,500 to 8,000 feet depending on the actual alignment selected. Conditions for construction of a new waterline are not ideal because of steep side slopes above sections of Highway 89 and the Truckee River that parallels the highway. One possible alternative is to follow the Truckee River Interceptor alignment. The 24-inch interceptor was constructed by the Tahoe-Truckee Sanitation Agency (T-TSA) in the mid-1970's and crosses the Truckee River five times between Alpine Springs and Squaw Valley. A second alternative is to construct the water line in the shoulder of Highway 89. A third alternative is to construct the waterline west of the highway and river across Forest Service property. A specific alignment study is needed to confirm the appropriate alignment in this area.

Supplemental storage may be required within ASCWD to maximize yield of the snow making wells. Project components for connection to ASCWD are listed below. This is based on following the alignment of the Truckee River Interceptor and installing a new 8-inch water line to Squaw Valley.

Table 7-6. Highway Corridor Connection- Project Components

Item	Number/Size
Conveyance Pipeline	
Diameter, inches	8
Length, feet	8,000
Meter	
Size, inches	4

One advantage to the Truckee River/Highway 89 corridor connection is that no supplemental pumping is required. The existing snow production well booster pumps are sufficient to deliver water directly into the District water distribution system.

EVALUATION OF ALTERNATIVE WATER SUPPLIES

Each of the water supply alternatives have been evaluated in terms of their feasibility of meeting some, if not all, of the increased water supply needs. Included in the evaluation are estimated construction costs, ability to meet the supplemental water supply needs of the District, time to implement, environmental constraints, water rights and legal issues.

Estimated Construction Costs

Unit Costs. Unit costs for estimating construction costs are based on information obtained from other water supply master plans developed by West Yost & Associates, recent bid results, and recent experience in the Squaw Valley area. All costs are based on current conditions and referenced to a June 2001 Engineering News Record 20 Cities Cost Index of 6318. The estimates of probable construction costs include a 25 percent construction contingency to account for the preliminary nature of the planning estimates. To estimate the total project cost, an additional 20 percent has been added to the estimated construction cost to account for engineering, administrative and legal costs. The costs of the improvements have been estimated for planning and comparison purposes only (*i.e.* with an accuracy in the range of +50% to –30%). These are appropriate costs for budgeting, but should be updated for specific project cost budgeting to match construction cost conditions at that time, and to integrate any more refined information on the project and site conditions.

Table 7-7 provides a summary of unit costs for the construction of water pipelines complete in place with surface restoration and/or paving. The unit costs include a 25 percent contingency. Actual costs will vary for a number of reasons depending on the size and timing of the construction bidding package.

Table 7-7. Unit Costs for Water Pipelines

Pipe Size, diameter	Cost per Linear Foot, dollars
4	40
6	60
8	80
10	100
12	120

The above costs assume that the pipe material will be ductile iron, class 200.

Other cost items include new wells and storage tanks. The recent cost breakdown for the reconstruction of Well 5 totaled about $250,000. This project did not include the construction of a new building. The estimated construction cost for a new well producing between 300 and 400 gpm with a new building and including a 25 percent contingency is $425,000. For the larger wells along the Truckee River, the estimated cost for a 600 to 800 gpm well including a building is $500,000

The estimated project costs for each alternative are shown in Tables 7-8 through 7-10. The construction cost includes a 25 percent contingency and the project capital cost also includes 20 percent for engineering, legal and administration of the project.

Table 7-8. Estimated Capital Costs for Squaw Valley Groundwater Supply Alternative [(a)]

Item	Unit Cost, Dollars	Estimated Construction Cost, Dollars
2 mgd Water Treatment Plant for Iron and Manganese Removal	L.S.	2,875,000
5 New Wells	425,000	2,125,000
3,300 feet of 4” pipe	40	132,000
3,500 feet of 6” pipe	60	210,000
2,100 feet of 8” pipe	80	168,000
1,700 feet of 10” pipe	100	170,000
Total Construction Cost		5,680,000
Engineering, Legal and Administrative Costs	20%	1,136,000
Total Project Cost		6,816,000

(a) Costs are in 2001 dollars

Table 7-9. Estimated Capital Costs for Truckee River Wells Supply Alternative[(a)]

Item	Unit Cost, Dollars	Estimated Construction Cost, Dollars
2 mgd Water Treatment Plant with UV	L.S.	3,000,000
2 New Wells	500,000	1,000,000
12,000 feet of 10” pipe	100	1,200,000
Total Construction Cost		5,200,000
Engineering, Legal and Administrative Costs	20%	1,040,000
Total Project Cost		6,240,000

(a) Costs are in 2001 dollars

Ability to Meet District’s Supplemental Water Needs

The District needs supplemental water supply to meet projected buildout demands. Supply must be provided to meet both maximum day demands and sustainable annual demands. The District will need an additional 1,600 gpm of maximum day production capacity and 1,100 af annually of sustainable supply capacity, to meet the estimated demands at buildout. The ability of each of the three alternative water supplies to meet these needs is discussed below.

Squaw Valley Wells. This alternative provides the needed maximum day demand production capacity, but can only supply an additional 557 af during a critically dry year. For the demands and supply to match additional supply during the critically dry years would be needed, the recently enacted water conservation ordinance must be enforced to reduce demands by about 31 percent or the amount of development limited. This alternative can easily be phased, with initial construction of 2 or 3 wells, and half the treatment plant capacity.

Truckee River Wells. This alternative will provide the needed maximum day demand production capacity, but relies on releases from Lake Tahoe for the water supply. Studies for the Truckee River Operating Agreement have shown that in critically dry years, releases from Lake Tahoe would probably be curtailed. The District would not likely receive supply from the Truckee River during multiple drought years when the water level in Lake Tahoe drops below the natural rim elevation. This situation last happened during the drought of 1988 to 1992.

Table 7-10. Estimated Capital Costs for Alpine Springs CWD Supply Alternatives[a]

Item	Unit Cost, dollars	Estimated Construction Cost, dollars
1. Summit Connection with Squaw Creek #2		
6,000 feet of 8" pipe oversizing	20	120,000
1,200 feet of 6" pipe	60	72,000
Meter and PRV	LS	30,000
Pump Station Oversizing	LS	60,000
Total Construction Cost		282,000
Engineering, Legal and Administrative Costs	20%	58,000
Total Alternative 1		340,000
2. Summit Connection without Squaw Creek #2		
1,200 feet of 6" pipe	60	72,000
6,000 feet of 8" pipe	80	480,000
Pump Station	LS	100,000
Meter and PRV	LS	30,000
Total Construction Cost		682,000
Engineering, Legal and Administrative Costs		138,000
Total Alternative 2		820,000
3. Highway 89 Connection		
8,000 feet of 8" pipe	80	640,000
Meter	LS	5,000
Total Construction Cost		645,000
Engineering, Legal and Administrative Costs	20%	130,000
Total Alternative 3		775,000

(a) Costs are in 2001 dollars

Alpine Springs County Water District. The Alpine Springs County Water District appears to have idle water supply production capacity of up to 700 gpm. This is the maximum capacity available and is less than half the buildout supplemental need of the District. However, it is not known at this time how much water would be available from ASCWD during any given year. The snow making wells are not used in the summer months, but the long-term sustainable yield from the wells, constructed in fractured rock has not been proven. The wells experience significant drawdown while pumping, and must be shut down at least eight hours in any given day of pumping to allow water levels to recover in the wells. The wells were installed in late 1992 at the end of the five year drought and have been operated only in the late fall and winter months. They have not been operated over a complete year or during a severe drought period.

Summary. The only alternative that provides the needed maximum day demand production and has a significant sustainable annual yield is the addition of new wells and an iron and manganese treatment plant in Squaw Valley. The Squaw Valley Wells alternative cannot provide sufficient annual supply during a drought year to meet full buildout demands, without implementation of demand reduction measures. However, this alternative, coupled with a 31 percent reduction in demands, could meet the District's supply needs in a critically dry year. A supply from Alpine Springs could also assist in meeting demands during a drought emergency.

Capital Cost Comparison

The estimated costs for the three alternatives are shown in Tables 7-8 through 7-10. The Squaw Valley Wells and the Truckee River Wells alternatives are sized to supply the needed maximum day demand requirements of 1,600 gpm. Review of the cost tables show that the development of wells in Squaw Valley will cost about $3.3 million for the wells and pipelines to deliver the water to a treatment plant that is estimated to cost about $3.5 million for a total project cost of $6.8 million. The cost of this alternative is essentially equal to the cost of developing wells near the Truckee River, and constructing a treatment plant capable of treating groundwater under the influence of surface water.

The third alternative, supply from Alpine Springs County Water District, has the capability of supplying only about 40 percent of the maximum day demand supply requirement if all the idle capacity of ASCWD system can be used. The estimated construction costs are significantly lower because existing production wells that are assumed not to need treatment can be used. The cost for connecting the two systems is relatively minor, if the Homesites at Squaw Creek No. 2 subdivision is constructed, and the amount of water provided by ASCWD is limited to the capacity of a 6-inch pipe of about 300 gpm.

Other Evaluation Issues

Other issues on which to evaluate the alternative water supplies include time to implement, environmental constraints, water rights and legal issues. Discussion of each of these issues for the three alternatives is presented below.

Squaw Valley Wells. Development of additional wells in Squaw Valley can proceed immediately with the construction of the replacement for Well 4R and negotiations for the acquisition of new well sites. The project could be staged, developing the first increment of water treatment after well sites and pipeline easements are secured and the first phase of the water treatment plant has been designed. Environmental documentation for this alternative should be relatively straightforward. It is anticipated that any unavoidable impacts from the construction of the facilities will be effectively mitigated. It is not anticipated that any water rights issues will need to be dealt with for this alternative, and the legal issues should be minor.

Truckee River Wells. This alternative would start with the filing of a water right application to the SWRCB to establish the right to divert surface flow from the Truckee River. This application can be filed at any time, but it will not be evaluated or result in a permit until after the TROA and its EIS/EIR is adopted. This process will take at least another two years. The amount of water that would be available in critically dry years would have to be worked out with other water rights holders. This may entail negotiation of an exchange for water stored in another reservoir in the area. The legal and environmental issues will center on obtaining the water rights, and land acquisition for the wells near the river. Environmental documentation should be relatively straightforward, with any unavoidable impacts from the construction of the facilities effectively mitigated.

Alpine Springs CWD Supply. The supply available from Alpine Springs County Water District must be further quantified to better define available production capacity and annual yield that can be supplied to Squaw Valley during average and critically dry years. Charges for the water supplied by the ASCWD also must be determined. It would likely take a couple of years to implement this alternative because of the probable need to incorporate construction of the pump and pipeline with development of the Homesites at Squaw Creek #2 Subdivision if the summit pipeline route is chosen. To construct the pipeline along the Truckee River corridor will also take a similar length of time to complete environmental and permitting processes to allow this construction along the highway and river. Legal issues for this alternative will focus on developing an agreement between the districts, and obtaining permits from the affected agencies, including Cal Trans, Department of Fish and Game, and T-TSA. Water rights should not be an issue with this alternative.

SECTION 8. WATER TREATMENT PLANT ALTERNATIVES

The groundwater quality data obtained from sampling production and test wells in Squaw Valley was reviewed to determine the general water quality characteristics of various groundwater sources, and establish specific treatment requirements to remove contaminants of concern. This evaluation was a preparatory step to identify the alternative treatment process that meets the goals of the study. The goal is to identify those treatment processes that can provide water that meets or exceeds current drinking water standards, provides flexibility for expansion and future treatment needs, and is cost effective. The recommended processes were further defined and conceptual treatment plant layouts and cost estimates prepared and the recommended treatment alternative was identified.

WATER QUALITY DATA REVIEW

Water quality data accumulated over several years from the existing wells supplying users within Squaw Valley are presented in Appendix L of the Squaw Valley Groundwater Background Data Technical Memorandum (Kleinfelder, 2000) described in Section 2. Table 4 of that TM, included as Table 8-1, is a summary of the water quality data from existing wells and test holes. Water quality data from Test Wells T-3, T-4 and T-5 (Table 1, Test Well Development Report, Kleinfelder, 2000), which were constructed in March 2000 was also reviewed. These test wells are located in the general proximity of the existing production wells. Test Wells T-4 and T-5 showed good production potential but both wells contained unusually high levels of iron and manganese. Additionally, Test Well T-4 contains arsenic at a concentration of 0.026 mg/l, which is currently below the MCL of 0.05 mg/l MCL, but would be above the proposed future MCL of 0.010 mg/l. The water quality in Test Wells T-4 and T-5 is also extremely high in total dissolved solids and has relatively high levels of sulfate. Although the well report indicates that these test wells were pumped long enough to establish stability during draw down tests, it appears that prolonged pumping could result in a reduction in the iron, manganese and arsenic concentrations in these two wells.

Existing production wells number 1, 2, 3 and 5 are the major producers for the District. Groundwater quality data reviewed in this study revealed that all four of these wells (with the exception of Well No. 5) have background iron concentrations that exceed the MCL. However, manganese does not appear to exceed the present standard of 0.05 mg/l in any of the current District operating wells, (Table 4 “Squaw Valley Groundwater Quality Test Data”, Kleinfelder, 2000 report).

Well No. 5 (5R) is approximately 100 feet south of Squaw Creek. During heavy pumping, this well may be operating under the influence of surface water inflow from the nearby creek. In May 2000 several wells were sampled to perform a Microscopic Particle Analysis (MPA) to confirm either the presence or absence of organisms of surface water origin. The results of these tests indicate that all wells sampled appear to be free of any organisms or particulate material of surface water origin. The MPA analysis of Well 5R, however, revealed the presence of 90 pollen grains.

Table 8-1. Squaw Valley Groundwater Quality Test Data

Well No.	Data Source	TDS (mg/L)	pH	Fe (mg/L)	Mn (mg/L)	Nitrate Nitrogen (mg/L)	Sulfate (mg/L)	Lead (ug/L)	Arsenic (ug/L)	Zinc (ug/L)	Toluene (ug/L)	Total Xylenes (ug/L)	Chloro-form (ug/L)	Sample Date
EPA MCL:		500	6.5-8.5	0.30	0.05	10	250	15	50	5000	1000	1750		
Resort at Squaw Creek #18-2	6	220	7.6	**.96**	**0.17**	<1	85	<1	<4	**5700**	<0.50	0.57	<0.50	1/2/90
SVMWC #1: Pilot 1 "Stable Well"	10	130	–	**5.9**	**0.12**	4	46	<5	<4	–	1	ND	–	7/17/89
	10	–	–	<0.05	<0.015	–	–	–	–	–	1.4	–	–	7/25/89
SVMWC #1:Pilot 2 "Hoffman Well"	6	180	–	**4.2**	**0.16**	3	16	7	<4	**6500**	20	<0.50	0.68	1/11/90
	11	–	–	**9.8**	**0.42**	–	–	–	8	40	–	–	–	11/8/99
SVMWC #1: Pilot 3 "Condo Well"	9	110	–	**1.1**	**0.25**	<1	18	<1	<4	40	–	–	–	12/12/91
	11	–	–	**1.1**	**0.2**	–	–	–	11	–	–	–	–	11/5/99
SVPSD #1	14	82	**6.4**	0.05	<0.005	0.15	11.3	–	–	–	–	–	–	12/58
	14	133	7.7	**0.6**	–	2.2	47.8	–	–	–	–	–	–	12/75
SVPSD #2	14	72	7.2	**0.3**	0.0	0.3	8.4	–	–	–	–	–	–	9/59
	14	111	7.8	**0.7**	–	0.4	9.6	–	–	–	–	–	–	12/59
SVPSD #3	14	81	6.7	0.05	0.0	0.2	5.6	–	–	–	–	–	–	12/58
	14	356	7.1	**0.6**	–	2.3	7.4	–	–	–	–	–	–	2/60
SVPSD #4 (4R; Ice Rink)	6	68	–	0.18	0.026	2	13.0	<1						
	14	39	**6.4**	0.05	<0.005	0.57	20.1	–	–	–	–	–	–	12/75
SVPSD #5 "Papoose"	14	106	7.0	0.18	0.05	3.8	12.7	–	–	–	–	–	–	6/74
SVPSD #6 (MW)	6	170	–	<0.030	<0.010	6	76	<1	–	–	–	–	–	
SVPSD #6 (Munic.)	6	85	–	**2.4**	**0.17**	4	13	<1	<4		2.4	0.57	–	
4th Fairway	11	–	–	**0.66**	**0.17**	–	–	–	10	–	–	–	–	11/9/99
SVPSD Test Well 1 (WD)	11	–	–	0.14	0.025	–	–	–	2	–	–	–	–	11/15/99
Horizontal B1	15	162	7.37	0.14	0.000	ND	–	1.6	2	–	–	–	–	5/13/98

Note: Bold-faced values exceed the EPA Maximum Concentration Level

Source: Kleinfelder 2000

– = not analyzed for this compound

After review of the sampling period and methods, the District concluded that the pollen was from air-borne contamination and not from the inflow of surface water. Any future MPA samplings should be carefully supervised and the results critically reviewed to assure that the analysis is not compromised in any way.

Other water quality data from Well 5R indicate that iron and manganese concentrations do not exceed the present MCL. Water samples analyzed in 1999 indicated that the arsenic concentration was less than 2 µg/l, which would place it below the anticipated future MCL for arsenic of 5 µg/l. Presuming that future pumping will result in no water quality degradation in Well 5R, this well can continue to be used to serve District customers. Since the MPA test, Well 5R has been permitted for use by the DOHS.

Wells No. 1, 2, 3 and 5 are presently District production wells. Well No. 4R, which has a reported tested pumping capacity of 600 gpm, apparently has not been in production since before 1990. The limited analyses on samples of water from Well 4R indicate that it is of relatively good mineralogical quality. The last sample collected in September of 1989 indicated that both iron and manganese concentrations were below the secondary standard 0.3 mg/l for iron, and 0.05 mg/l for manganese. The arsenic concentration was less than 4 µg/l, which is below than the proposed standard. The relatively low alkalinity and hardness compared to water in wells located at a greater distance from Squaw Creek would also suggest that this particular well could be under the influence of surface water. Well 4R should definitely be developed based upon good production capability and suitable water quality.

The present focus of new well development in the District is in an area downstream of the existing well field along Squaw Creek. Test Wells T-3, T-4 and T-5 have been constructed in this area. The most productive of these Test Wells, T-4, contains high levels of iron, manganese and arsenic, and the water produced by this well must be treated to meet water quality standards.

There is another issue associated with the water quality in the test wells, which is also of concern. Test Wells T-4 and T-5, in addition to having high concentrations of iron, manganese and arsenic, also have high concentrations of both sulfate and total dissolved solids, which may occasionally exceed the secondary standard. This highly mineralized groundwater can be treated effectively to remove the contaminants of concern (iron, manganese and arsenic), but would have to be diluted with less mineralized sources to reduce sulfates and total dissolved solids to compliance levels. Prolonged pumping of the well may reduce the background concentrations of all of the above-mentioned mineral constituents.

Developing wells closer to Squaw Creek to avoid highly mineralized groundwater in this area may be a more desirable alternative. Such wells may require filtration because of the possibility that they may be surface water influenced, but they definitely would not exceed sulfate and total dissolved solids standards.

TREATMENT PROCESS ALTERNATIVES

Three treatment processes were evaluated, and are described below.

- Pressure greensand filtration
- Ozone oxidation/gravity filtration

- Membrane filtration

The conventional method for removal of iron and manganese from groundwater involves oxidation followed by filtration. Filtration is usually accomplished by the use of pressure filters or conventional gravity beds. Oxidants include chlorine, chlorine dioxide, potassium permanganate and ozone. All of these oxidants have been used successfully to convert the reduced forms of manganese and iron to an insoluble precipitate, which can be filtered from the water. Oxidation of the reduced form of arsenic to arsenate, coupled with adsorption on the floc produced by the oxidation of manganese-to-manganese dioxide and the oxidation of iron to ferric hydroxide, generally removes much of the arsenic present in the water. Occasionally, a primary coagulant like aluminum sulfate (alum) is needed to reduce arsenate to acceptably low concentrations.

The oxidation of both iron and manganese is temperature and pH dependent and requires a certain amount of detention time for the oxidized forms to develop. The reaction of chlorine with iron is very rapid, so detention time in a treatment system is relatively short. On the other hand, the reaction of potassium permanganate with manganese is more pH dependent and requires a longer detention time to form the filterable manganese dioxide. Elevating the pH of the water is often required to obtain effective manganese removal in reasonable detention times. In systems designed to simultaneously remove iron and manganese, chlorine is often fed initially to oxidize iron and other reduced compounds because it is less expensive than potassium permanganate that must be used to remove manganese. For example, it requires 0.63 mg/l of chlorine to react with 1 mg/l of ferrous iron, whereas to oxidize a similar 1 mg/l of manganese, 1.92-mg/l potassium permanganate is needed.

Often it is difficult to obtain effective removal of both iron and manganese because of the differing oxidation requirements of the two contaminants. To overcome these limitations, the manganese greensand filtration process was developed many years ago and has been used extensively in groundwater iron and manganese removal application. Greensand is a natural occurring mineral (Glauconite) that has ion exchange properties and is very effective for removing iron, manganese and sulfide. The oxidative capacity of manganese greensand is limited, however, and the greensand must be regenerated with potassium permanganate. Regeneration can be either intermittent with periodic application of a concentrated solution of permanganate, or it can be continuous with permanganate metered into the influent. The continuous application method is preferable, especially on larger systems, because intermittent regeneration procedures are time consuming and present a disposal problem with the spent regenerate solution.

The continuous regeneration method is virtually identical to the treatment scheme used with conventional coal/sand filters (dual media) in which the required amount of permanganate to oxidize the manganese is applied to the influent. The greensand media has a theoretical advantage in that if the permanganate dosage is inadequate, the media has the capacity to continue to oxidize and remove manganese. Likewise, if permanganate is overfed, the greensand will absorb the excess, unreduced potassium permanganate and prevent a bleed through that would cause “pink water.” Once the manganese is oxidized, the greensand removes the manganese dioxide by filtration. Typically, a greensand filtration system consists of a bed of greensand filter material installed in pressure vessels, topped by coarser less dense anthracite coal to increase the suspended solids removal capacity of the filtration system.

It is also possible to design an effective iron and manganese filtration system using common silica sand and anthracite coal. In this process both chlorine and potassium permanganate must be fed continuously at the precise dosage to satisfy oxidation requirements, because the absence of greensand and its regenerative ion exchange capacity is not provided by common silica sand. However, after an extended period of operation, a coating of manganese dioxide forms on the silica sand media and this coated media begins to perform like the manganese greensand, although not at the same effectiveness level.

There has been recent renewed interest in using chlorine dioxide as an oxidant in iron and manganese removal. Chlorine dioxide oxidation is advantageous to the manganese greensand process because it eliminates use of two different oxidants. Chlorine dioxide will effectively oxidize both iron and manganese without the need to elevate the pH of the water being treated or provide significant detention time prior to filtration. Improved equipment for producing chlorine dioxide has also made this process more attractive for use in water filtration applications. Rather than producing chlorine dioxide by reacting chlorine gas with sodium chlorite, there is a new process available using two more safely stored and handled mixtures in a specialized reactor. This process involves reacting a proprietary mixture of sodium chlorite and hydrogen peroxide (Purate) with sulfuric acid. All safety measures required for the storage and handling of chemicals will be incorporated into the final design of the selected process. The treatment plant will not be a safety issue for the surrounding properties or the rest of the valley.

The technical feasibility of applying the membrane filtration process to the treatment of these groundwater sources for removal of iron, manganese and arsenic was also evaluated. Membrane filtration offers some advantages over the granular media filtration processes described above, especially if the groundwater source wells are contaminated by infiltrating surface water possibly containing such organisms as Giardia lamblia or Cryptosporidium. Membrane filtration is very effective in removing these microbial contaminants and would be favored by the DOHS over the conventional oxidation/filtration process. The membrane filtration process is assigned a higher particle removing rating in the Surface Water Treatment Rule. Because of this higher rating, the downstream detention time required to meet disinfection requirements with chlorine is less where filtration is provided by membranes. However, where iron and manganese are present, oxidative chemicals, such as chlorine, potassium permanganate or chlorine dioxide, must be applied ahead of the membrane filtration process to convert the soluble iron and manganese into an insoluble filterable form. Unlike surface water treatment applications where contaminants are generally in particulate form, and hence can be removed by membrane filtration without coagulation, this condition is not present in groundwater iron and manganese removal applications.

Further compromising the possible benefits of the membrane filtration process is that there are only a few membrane formulations, which can withstand contact with strong oxidants and still retain viable filtration properties. For example, the membranes used by U.S. Filter Memcor, the most experienced membrane manufacturer, are not resistant to chlorine and cannot be used in this application. One manufacturer, Pall Corporation, produces membranes, which are resistant to these oxidants. According to information provided by their sales representatives, Pall has been involved in several pilot scale groundwater treatment applications for iron, manganese and arsenic.

Although the Pall membrane filtration process may be acceptable for treatment of well water containing iron, manganese and arsenic, the relatively high cost of membrane filtration systems may make the financial feasibility for the District project questionable. Comparable capacity membrane filtration equipment is generally twice the cost of conventional filtration equipment. Because a similar chemical oxidation process is required before filtration by either a conventional or a membrane filtration process, there is no cost savings advantage for membranes.

Based upon our knowledge of applicability and costs of the membrane filtration process, it does not appear to be an economically feasible process for an iron and manganese removal system in Squaw Valley.

Other Treatment Concerns

Another potential treatment problem has developed recently. Traces of MTBE have been detected in Well No. 1. Although this discovery may not represent an ongoing presence of these materials, treatment facilities may have to be designed to remove these contaminants should they be present in any of the proposed well water sources.

Removal of diesel fuel and MTBE requires use of activated carbon to adsorb these contaminants. To supplement the basic iron, manganese and arsenic removal filtration systems, additional pre or post treatment processes using activated carbon would be required. Either powdered activated carbon (PAC) or granular activated carbon (GAC) will effectively remove these materials. However, removal of MTBE by activated carbon is not very efficient. Massive doses are needed adding to expensive treatment costs. Possible preoxidation by chlorine dioxide may provide better removal at lower carbon use rates. Pilot testing would confirm these preliminary assessments.

The use of powdered activated carbon would require a two-stage filtration process with PAC being applied in the first stage to adsorb diesel fuel and MTBE. Iron, manganese and arsenic removal would be accomplished in the second stage filtration process. This process could be contained in open tanks requiring several stages of pumping to deliver treated water into the distribution system.

Alternatively, a slightly different two-stage treatment process could be used. This process would involve first-stage oxidation and filtration to remove iron, manganese and arsenic followed by second stage GAC contactors to adsorb diesel fuel and MTBE. This process would be totally contained in pressure vessels and could eliminate repumping of treated water.

Based upon this preliminary assessment of the best process to contend with removal of diesel fuel and MTBE, a slight variation of the gravity two-stage filtration process is recommended. This variation in process would use GAC in place of anthracite coal in the second-stage filtration process. The GAC in the filters would provide a continuously available barrier to remove diesel fuel or MTBE should it be present in the water. With this process, PAC application capability would only be needed if more than trace amounts of either of these contaminants are found in the well water, and their presence persists over an extended period of time.

TREATMENT PROCESS EVALUATION

The favored treatment processes for removing iron, manganese and perhaps arsenic from the groundwater have been reviewed. A preliminary screening of potential processes was completed prior to identifying the favored processes. The capability of meeting treated water quality objectives applicable to water quality conditions in the valley were the major factor leading to the selection of viable treatment processes. The three process alternatives considered were:

1. Pressure greensand filtration
2. Ozone oxidation/gravity filtration
3. Membrane filtration

Pressure Greensand Filtration

The conventional method for removal of iron and manganese from groundwater involves oxidation, generally with chlorine, chlorine dioxide, potassium permanganate or perhaps ozone, followed by filtration. Pressure filters are generally used in iron and manganese filtration applications. Often where both iron and manganese are present at concentrations above the MCL, the manganese greensand filtration process has certain advantages that make it an attractive process. Consequently, in the ensuing cost analyses, the conceptual design was based upon the use of the manganese greensand filtration process. Costs received from a vendor of iron and manganese filtration treatment systems were used for the greensand pressure filtration alternative.

The greensand filtration process, although generally used where iron and manganese is the principal concern, also has the ability to perform effectively as a filtration media for arsenic removal. The pressurized greensand filtration process could be used for arsenic removal wherein oxidation of the reduced forms of arsenic generally found in groundwater would be accomplished with chlorine and potassium permanganate or perhaps ozone, and then removed through filtration. To improve arsenic removal, a small amount of a primary coagulant such as aluminum sulphate (alum) or ferric chloride could be added to remove the arsenate precipitate.

The inline filtration process (pressure greensand filters) has some limitations when used for removal of microbial contaminants of surface water origin. For example, the DOHS discourages the use of pressure filters using the inline filtration process for surface water sources because of concern for turbidity breakthrough caused by the generally higher pressure used with pressure filters. Where the inline process is used, however, DOHS restricts the pressure filter rates to no more than three gpm per square foot of filter area. Recognizing that there is a possibility that new wells may possibly fall under the influence of surface water contamination, a filtration rate of three gpm per square foot was selected to size the pressure filters for this principally iron and manganese removal application. If a surface water source becomes available in the future, a filtration rate of three gpm per square foot would also comply with the current design standard for the use of pressure filters for surface water treatment. It is likely, however, that for a direct surface water treatment application, or in a situation where groundwater wells could become contaminated with surface water inflow, an additional treatment barrier would be needed to comply with DOHS standards.

A process such as ultraviolet (UV) sterilization could be applied to the filtered water to meet possible cryptosporidia removal standards requiring a higher level of disinfection than could be provided solely by chlorination. Ultraviolet light sterilization has been found to be very effective for inactivation of cryptosporidia oocysts. UV treatment would probably also be the most effective and least expensive addition to the pressure filtration process to meet drinking water standards. Consequently, space should be provided in the treatment facilities for the addition of an ultraviolet disinfection process should it become necessary in the future.

Ozone Oxidation/Gravity Filtration

A treatment process alternative using ozone and gravity filtration was considered because this process would have the capability of oxidizing and removing iron and manganese, and inactivating and removing any microbial contaminant of surface water origin. Further, this complete treatment process, supplemented with ozone, would also be able to effectively treat any quality surface water supply while meeting all current and anticipated future drinking water standards. The ozone/gravity filtration process is significantly more complex than the pressure filtration alternative, but would have substantially greater treatment capability. The process could very adequately treat all groundwater sources in the basin, and effectively remove iron, manganese, and arsenic. Consequently, a process alternative based upon the ozone/gravity filtration alternative furnished in a factory-built package plant by U.S. Filter (Trizone process) was considered in the evaluation.

Membrane Filtration

A preliminary assessment of the feasibility of using membrane filtration for this application was also completed. Discussions with membrane suppliers indicated that iron and manganese would have to be first oxidized with a combination of chlorine and potassium permanganate prior to membrane filtration. The membrane process, thus offers no advantages with respect to possible elimination of chemical treatment requirements.

Some membrane manufacturers provide membrane modules that are resistant to damage by such oxidants as chlorine and potassium permanganate and would be applicable for this application. Other manufacturers use a membrane filter that must not be exposed to any oxidants such as chlorine and permanganate. Selection of a proper membrane would be a consideration if membrane filtration were selected.

This application was discussed with Pall, manufacturers of microfiltration equipment, who provided a budgetary cost estimate for the equipment of approximately $0.50 per gallon per day (gpd) of capacity. This unit cost of membrane filtration equipment compares to a cost of around $0.20 gpd of capacity for pressure filters, and around $0.36 gpd of capacity for the ozone/gravity filtration package plant equipment. In addition to a capital cost disadvantage, there would be no appreciable operating cost savings with the membrane filtration process. As mentioned previously, chemical oxidants must be applied to convert dissolved iron and manganese into filterable floc, negating any possible operation and maintenance cost advantages of membrane filtration. The uncertainty associated with the useful life of the membranes and the consequent high replacement costs also contributed to the reasons why this process was rejected for the District's application.

Following this preliminary screening, the first two process alternatives were retained, and the membrane process was eliminated from further analysis. Conceptual designs were then developed for the pressure greensand and the ozone oxidation/gravity filtration process. The principal components of each system alternative were identified and preliminary design criteria developed for the processes. These criteria and the conceptual design information were then used to prepare projected construction costs for a treatment facility designed around one of these two treatment processes. Cost developed in these analyses is presented in the following sections.

WATER TREATMENT PARAMETERS

Water quality analyses performed on samples taken from various well sites in the valley indicate that there is a high probability that iron and manganese concentrations in the groundwater from any newly developed well will exceed the current drinking water standards. There is also the possibility that background arsenic levels in some of the wells may exceed what presently is believed to be the probable final arsenic standard of 20 µg/l. The present MCL for arsenic is 50 µg/l. Treatment to remove iron and manganese and perhaps arsenic, if the arsenic MCL is exceeded, must be provided.

As many as six new wells are anticipated to be constructed in the western half of the basin. It is anticipated that the pumping capacity of these individual wells will be between 100 and 400 gpm from each well head. The resulting total production from the new well sites that require treatment will be approximately 1,200 gpm.

The future water supply demand studies indicate that the District's projected maximum day demand from ultimate development of the service area is about 2,525 gpm (3.6 mgd). As described in Section 7 – Alternative Water Supplies, the needed treatment capacity is estimated to be about 1,200 gpm. The nominal treatment capacity should be about 10 percent larger than this to account for plant production downtime for backwashing of the filters. The required capacity of the proposed new treatment facilities has, therefore, been selected to be 1,400 gpm. A flow of 1,400 gpm (2 mgd) has been used in sizing the treatment process alternatives that are described below.

Previously, the District performed a preliminary assessment of possible water treatment plant sites in Squaw Valley to meet system expansion requirements. Based upon such factors as cost, access, availability, site development difficulty, timing, flexibility of development, construction cost, easement accessibility, potential environmental conflicts and public acceptance, six potential sites were evaluated. The outcome of this evaluation indicated that the preferred site for the new treatment facilities was adjacent to the present District office/fire station.

The ensuing analysis of process alternatives incorporated site-specific conditions into the conceptual designs developed for the two process alternatives. Such factors as the size and configuration of the building and arrangement and orientation of particular components within the building for accessibility for both construction and maintenance were the major considerations. Incorporating treatment facilities into the site without requiring any of the existing facilities including the District administrative center and the fire station to have to curtail any activities or relocate services was also a factor considered in the analysis. Consequently, the new treatment facilities were placed on open and undeveloped areas on the site. In establishing the location of treatment facilities, future expansions to handle the ultimate water treatment needs of the District were also considered.

PRESSURE GREENSAND FILTRATION ALTERNATIVE

A process schematic and a conceptual plant layout were created for a nominal 2-mgd capacity treatment facility designed around the use of pressure filters. As shown in the process flow schematic in Figure 8-1, groundwater from the new wells would be delivered to the treatment plant in a dedicated raw water line and processed through the pressure filters for removal of iron and manganese. Chlorine and perhaps potassium permanganate would be applied to the raw water to oxidize iron and manganese. Alternatively, chlorine dioxide could be used which could oxidize both iron and manganese without requiring the use of the greensand media. More studies will be needed to evaluate the feasibility of chlorine dioxide. A polymer filtration aid could be applied as needed to assist in the filtration process.

After chemical application and mixing, the water containing oxidized iron and manganese would be introduced into the pressure filters. Multi-cell pressure filters are proposed to permit internal generation of backwash water during operation. With this design the output of three cells remaining in service can be used to backwash the fourth cell reducing the need to use appreciable treated water from distribution system storage reservoirs. Optionally, backwash water obtained from the distribution system can be used to backwash these filters.

The cost analysis presumed that the pressure filters would be furnished with manganese greensand filtration media. The process would be totally pressurized from the well head to the distribution system reservoirs. This alternative would probably require construction of a dedicated finished water supply line to the existing 1.1 million gallon reservoir which would be used to meet disinfection detention time (CT) requirements should CT compliance become necessary with this system.

Costs are based upon the use of two multi-cell pressure filters, each rated at a capacity of 700 gpm at a filtration rate of three gpm per square foot. The three gpm per square foot filtration rate was selected so that future treatment of either a direct surface water source or groundwater sources judged to be under the influence of surface water could be processed without incurring reduced plant capacity caused by DOHS filtration rate limitations for pressure filters. If the treatment plant were to treat for the removal of iron and manganese only, the filtration rate could be higher than 3 gpm per square foot.

The conceptualized system would be designed to operate under automatic control using a PLC based control system. It was assumed that the pressure filters would operate at variable flow depending upon the number of wells being used to meet water demands within the system at any one particular time. Chemical feed systems would be paced to accommodate this mode of operation.

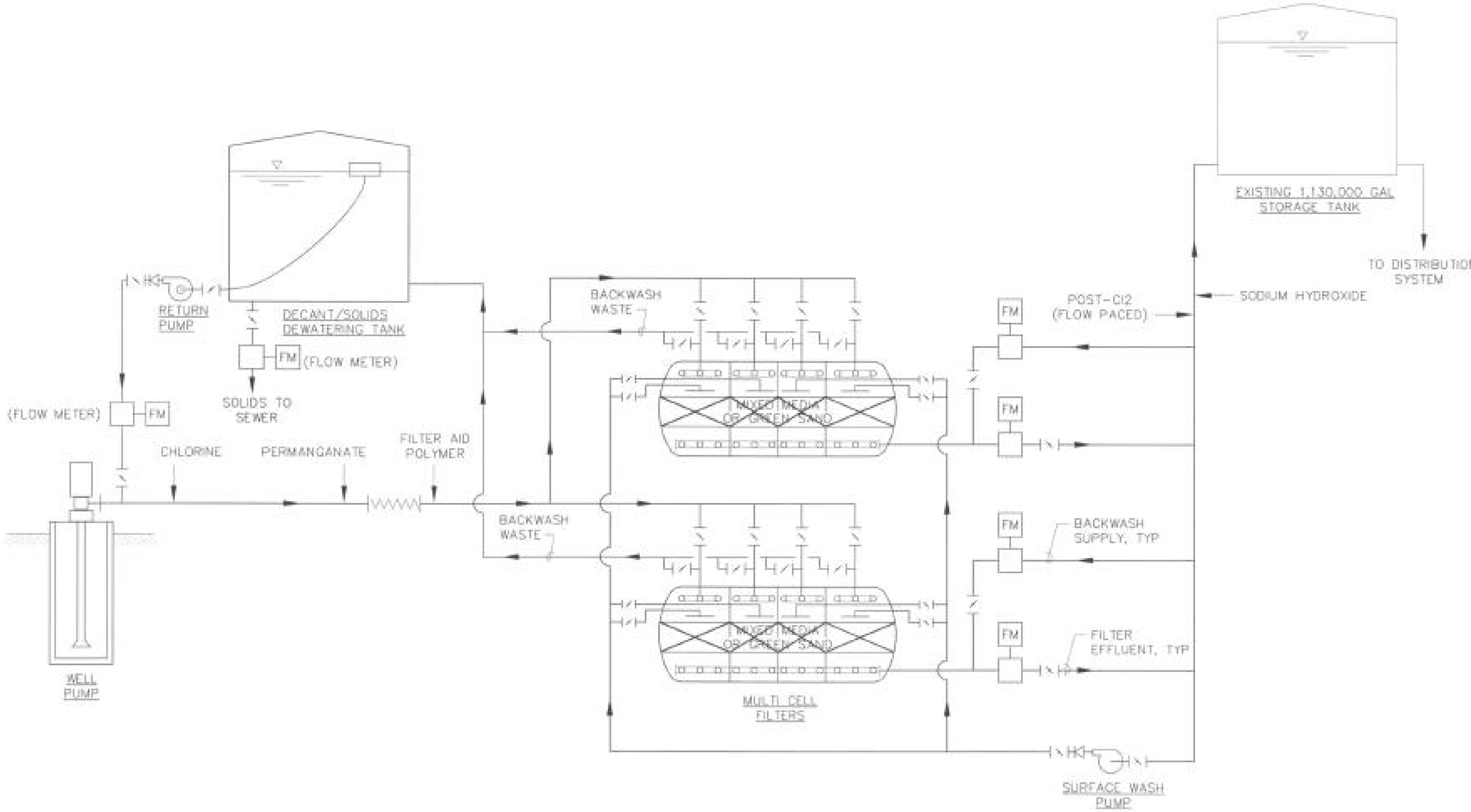

FIGURE 8-1
FLOW SCHEMATIC
PROPOSED PRESSURE FILTER LAYOUT ALTERNATIVE
SQUAW VALLEY PSD

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Figure 8-2 is a floor plan layout of the conceptual pressure greensand filtration alternative. The entire treatment facility would be housed within a single story 60-foot wide by 87-foot long building. In addition to the initial pressure filters, the facility would have space for two future units to provide an ultimate plant capacity of four mgd, if treatment of all supplies was needed. Within the building, dedicated space is provided for a lab/control room, operator's office, treatment chemical storage and feed area, process pumps, an electrical room and a standby emergency power generator capable of running the complete facility.

Additional and supplemental treatment may be needed to enable the pressure filtration alternative to be used to treat a surface water source or a groundwater proven to be under the influence of surface water. Ultraviolet (UV) light treatment to provide an additional disinfection barrier would be the favored process. An in-line type, pressurized unit could be installed on the filter effluent piping prior to discharge into the distribution system.

Suitable UV equipment is available from several manufacturers. A unit that has a capacity of 1,400 gpm would be suitable for this application. The unit would consume about 10 kilowatts per hour (kwh) of operation. The additional electrical equipment could be accommodated in the plant electrical room.

The estimated cost to install two units (one for service with one for standby) would be about $350,000. The installation could be completed as a retrofit at the time the additional disinfection facilities are required.

Construction costs developed for this alternative presume that the building would be constructed of concrete masonry block with a steel roof system. The roof system would consist of steel columns, beams, open web joists, steel decking, insulation and finished standing ridge seam steel roofing. Architectural treatment of the building would be selected to harmonize with the two existing structures on the site.

Figure 8-3 provides a west elevation view of the proposed building where architectural features incorporated in the conceptual design would reflect and complement the design used for the fire station. Only the west elevation of this facility would be visible from community streets. It is likely that architectural refinements to the building design would be needed in the final design, which could influence the costs. The costs presented in this study are based upon a facility comparable to the one illustrated by the floor plan and elevation views in Figure 8-3.

Following discussions with District personnel, a site north of the existing administration building, which is presently vacant of any structures, was selected as the most favorable location for the facilities. The selected location of the proposed pressure filtration facilities on the existing site is illustrated in Figure 8-4. At this location, the building was configured so that access for construction and maintenance purposes is from the front and the rear. The building would be placed on the north property easement line.

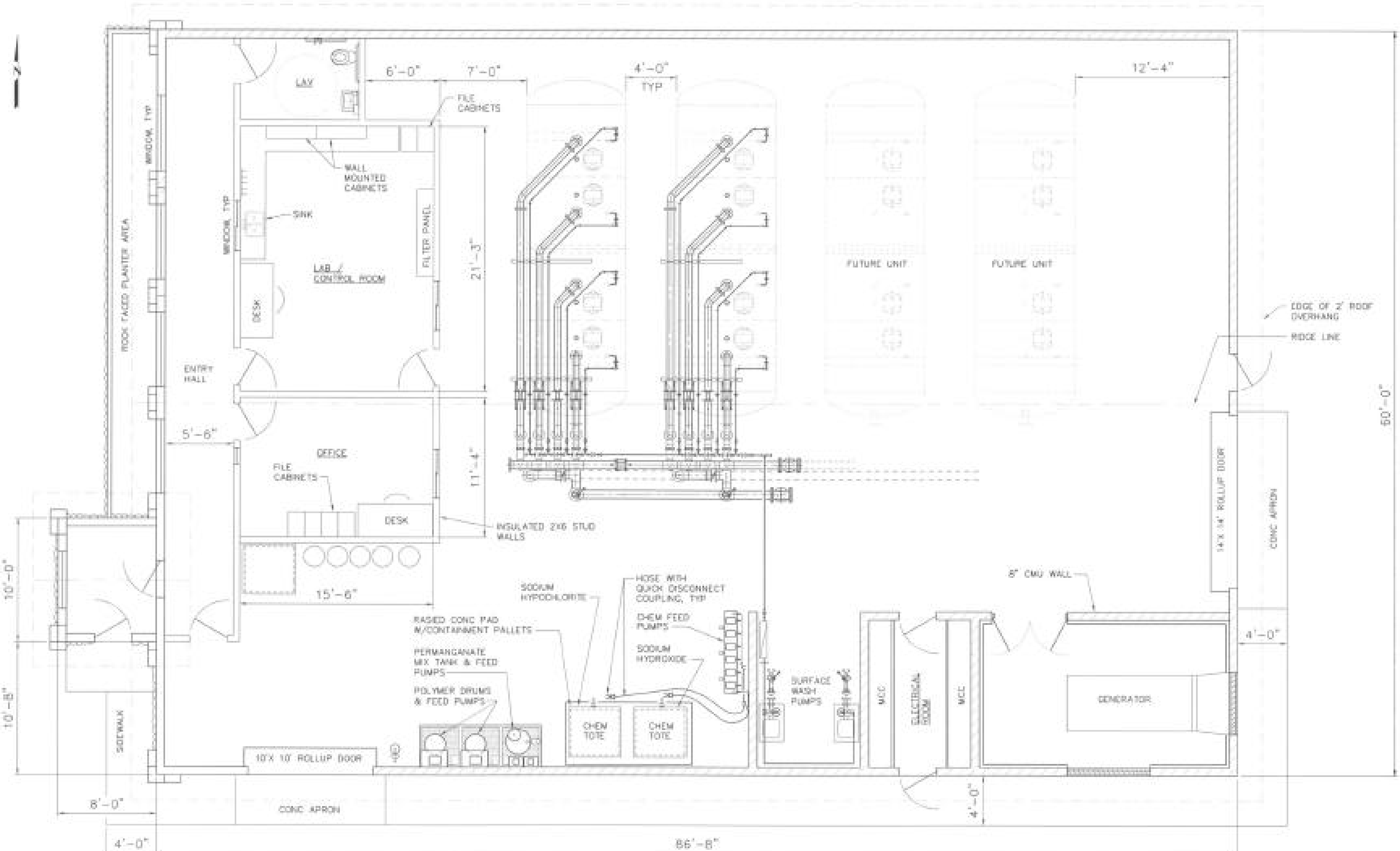

FIGURE 8-2
FLOOR PLAN
PROPOSED PRESSURE FILTER LAYOUT ALTERNATIVE
SQUAW VALLEY PSD

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FIGURE 8–3
WEST ELEVATION
PROPOSED PRESSURE FILTER LAYOUT ALTERNATIVE
SQUAW VALLEY PSD

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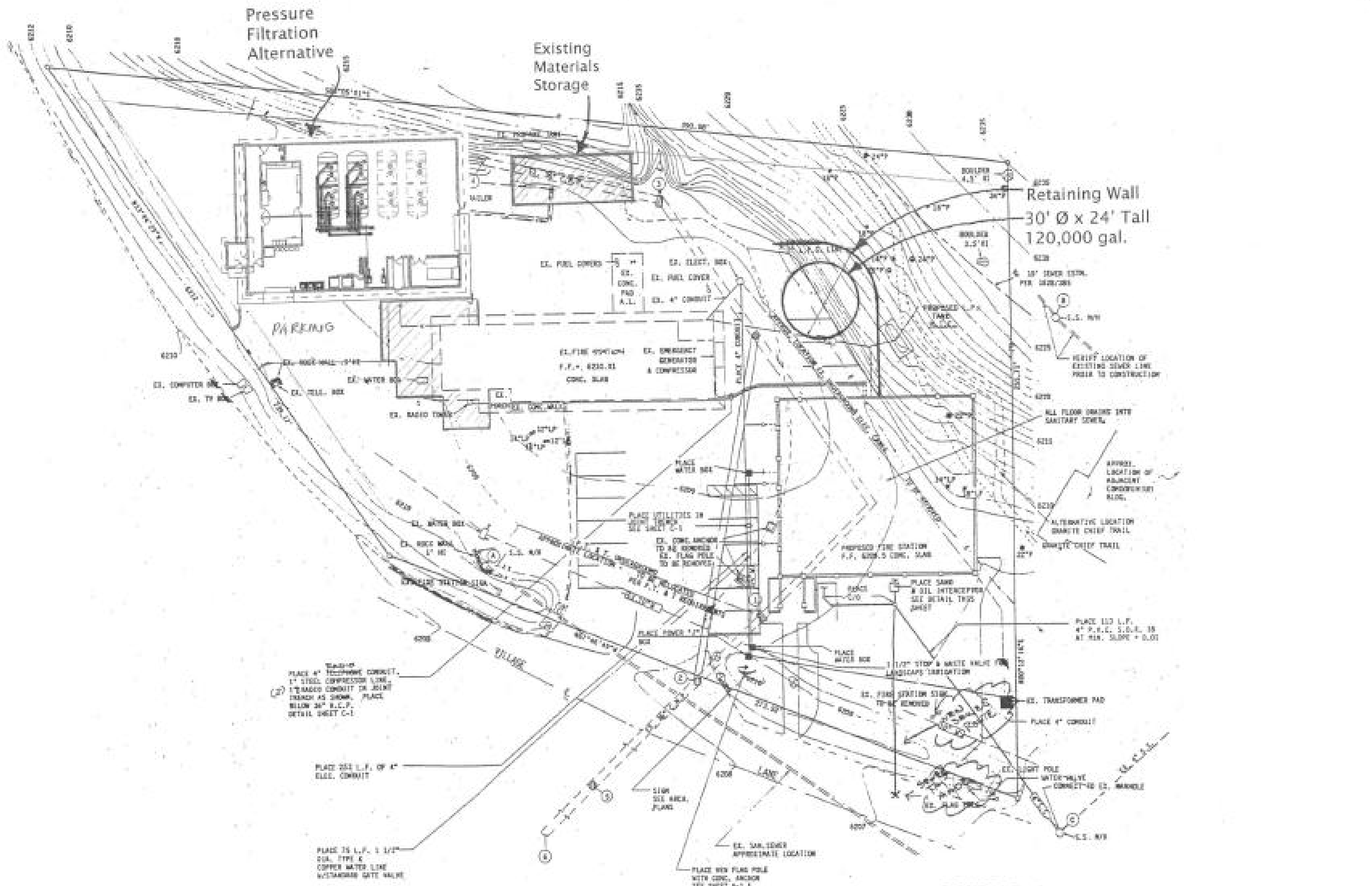

FIGURE 8-4
SITE LAYOUT
PROPOSED PRESSURE FILTER LAYOUT ALTERNATIVE
SQUAW VALLEY PSD

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An additional facility required with the pressure filtration alternative is a backwash wastewater decant/solids separation tank. The most favorable location for this 120,000-gallon capacity tank, which is 30 feet in diameter and 24 feet tall, would be behind the existing fire station. As illustrated on Figure 8-4, considerable excavation would be required to develop a suitable site for this tank and a fairly significant retaining wall would also be needed. This tank would be used to separate solids removed by the filters from the waste backwash water. Clarified water would be collected by a floating decant mechanism and recycled through the treatment facilities. The waste solids would be periodically discharged to the sanitary sewer. T-TSA has been contacted about discharging the treatment plant wastes, but has not responded. T-TSA permits this discharge if the wastewater does not have a detrimental impact on the sewer treatment plant. Metering would be provided on both flows leaving this tank for monitoring and for billing purposes. Waste solids discharged to the sewer would be subject to connection and disposal fees from T-TSA based both upon flow and solids content. These costs will need to be added to the estimated construction and operation costs.

Preliminary estimated construction costs were developed for the pressure filtration alternative using the information developed in preparing the conceptual design, supplemented by proposals from equipment suppliers. Historical cost data maintained in SPH Associates files and costs from "Means Construction Costs" were also used to develop preliminary estimated construction costs for the pressure filtration alternative. This cost estimate is presented in Table 8-2. The expected construction cost of the facility is based upon a summation of the component costs of the major features of the pressure filtration treatment plant alternative. The cost was developed principally for comparison to the other alternative evaluated in this study. Site specific costs common to both alternatives such as a sewer connection fee, any specialized site landscaping, raw water or finished water transmission piping not located on the plant site, unique building architectural features or refinements, etc. would have to be added to costs shown in Table 8-2.

The total estimated capital cost of the iron and manganese treatment system alternative is $3,450,000. This cost includes 25 percent for contingency and 20 percent for engineering, legal and administrative costs. Referring to the component costs in Table 8-2, site work and yard piping was estimated at approximately $260,000. The cost of the filtration equipment is projected at $320,000 supported by a proposal received from LoPrest Filter Company. A cost of $80,000 to install the filtration equipment was estimated based upon experience from previous similar projects. Filter face piping and valves were estimated to cost about $95,000. The cost to expand the plant to four mgd by adding two pressure vessels, piping, and chemical feed systems would be about $600,000 to $700,000.

Flow meters and other process flow meters would be required. These costs in Table 8-2 are based upon the use of magnetic flow meters. The cost table also includes about $65,000 for pumping equipment. The pressure filtration design would require surface wash pumps. A pump would be required to pump supernatant from the backwash decant tank back into the treatment process. A sludge pump would be required to deliver waste solids from the backwash decant tank into the sewer system.

An allowance of $200,000 has been included for a dedicated finished water transmission line from the treatment plant site to the 1.13 million gallon (MG) storage tank. Pumping the treated water to the tank will also provide removal of Radon that may be present in the groundwater through off-gassing while stored in the tank.

A major component of the estimated cost is the building estimated to be around $700,000. This cost equates to a unit building construction cost of approximately $135 per square foot. At this cost, the building is regarded to be only a functional facility without any frills.

Table 8-2. Estimated Capital Cost
2 mgd Pressure Greensand Filtration Alternative [(a)]

Component	Estimated Construction Cost
Site Clearing/Preparation	35,000
Relocate Site Drainage	25,000
Excavation and Backfill	30,000
Paving and Surfacing	45,000
Retaining Wall	15,000
Concrete Building/Tank Foundations	155,000
Filtration Equipment	320,000
Install Filtration Equipment	80,000
Filter Facing Piping and Valves	95,000
Flow Meters	20,000
Yard Piping	110,000
Offsite Piping (line to 1.13 MG tank)	200,000
Chemical Feed Equipment	25,000
Surface Wash Pumps	15,000
Reclaimed Water Return Pumps	40,000
Backwash Decant Tank	120,000
Sludge Discharge Pumps	10,000
Building Superstructure	210,000
Building Interior Walls	105,000
Building Roof	85,000
Windows	8,000
Roll-up Doors	15,000
HVAC	70,000
Furnishings/Cabinets	30,000
Painting	55,000
Interior Doors	12,000
Instrumentation	90,000
Electrical	200,000
Emergency Generator	80,000
Contingency @ 25%	575,000
Total Estimated Construction Cost	**2,875,000**
Engineering, Legal, Administrative @ 20%	575,000
Total Capital Cost	**$3,450,000**

(a) Costs are in 2001 dollars

Electrical and instrumentation was estimated to be about $290,000. An emergency generator capable of operating at least one of the filter units during a power outage is estimated to cost $80,000.

Because of the very preliminary nature of this cost estimate, a 25 percent cost estimating contingency was added to the projected construction cost. The costs also include engineering, administrative or legal services that for this project would probably amount to about 20 percent of the construction cost. No costs have been included for property or easement acquisition for offsite pipelines. The treatment plant instrumentation and control system is based upon a relatively rudimentary PLC based system designed for plant control only. If more extensive system-wide SCADA facilities were desired for monitoring and control of the various raw water pumps and storage reservoirs in this system, additional costs would be incurred. Costs of an expanded SCADA system could be included in a subsequent more detailed predesign effort after the particular requirements are more firmly known.

OZONE OXIDATION/GRAVITY FILTRATION ALTERNATIVE

The second process alternative considered for the iron and manganese treatment facility would involve ozone oxidation/filtration. A conceptual layout drawing of a 2-mgd capacity water treatment facility based upon the use of factory-built modular treatment units using a process of ozone oxidation followed by two-stage clarification/filtration was prepared and is illustrated in Figure 8-5. Figure 8-6 is a process flow schematic for this alternative. As noted in the flow schematic, there is substantial difference between this process and the pressure filtration process. With the ozone/gravity filtration process, raw water would be delivered from the well sites to the treatment plant where the water would flow by gravity through the treatment plant and then into a below-grade filtered water storage basin or clearwell. From the clearwell, designed for storage of backwash water and to meet CT requirements, finished water would be pumped into the distribution system. The finished water or high service pumps would be sized to meet delivery and pressure requirements of the distribution system. The “breaking of head” required to incorporate gravity treatment units into the process requires an additional pumping step and would contribute to potentially higher system operating costs. Waste backwash water and flushing water from the clarifier would be discharged by gravity into a below-grade decanting basin similar in design but smaller than the clearwell. This basin would be designed to separate solids by settling and then decanting the clarified supernatant back to the treatment process. Waste solids would be pumped to the sanitary sewer.

As illustrated in Figure 8-6 two modular treatment units would be provided. The treatment units include a first stage ozone oxidation process followed by upflow contact clarification and down flow mixed media gravity filters for polishing. Iron and manganese would be oxidized by ozone. Coagulants of alum and polymer could be added to improve clarification and filtration to provide effective removal of surface water contaminants. Ozone oxidation would also provide a higher level of removal of any microbial contaminants, which could be present in a surface source or groundwater under the influence of surface water. Unlike the pressure filtration system, a dedicated source of backwash water is needed to clean the gravity filters. This process is obviously more complex than simple pressure filtration, but has the capability of providing a higher level of treatment, which could be beneficial if surface water is later used as a supply source.

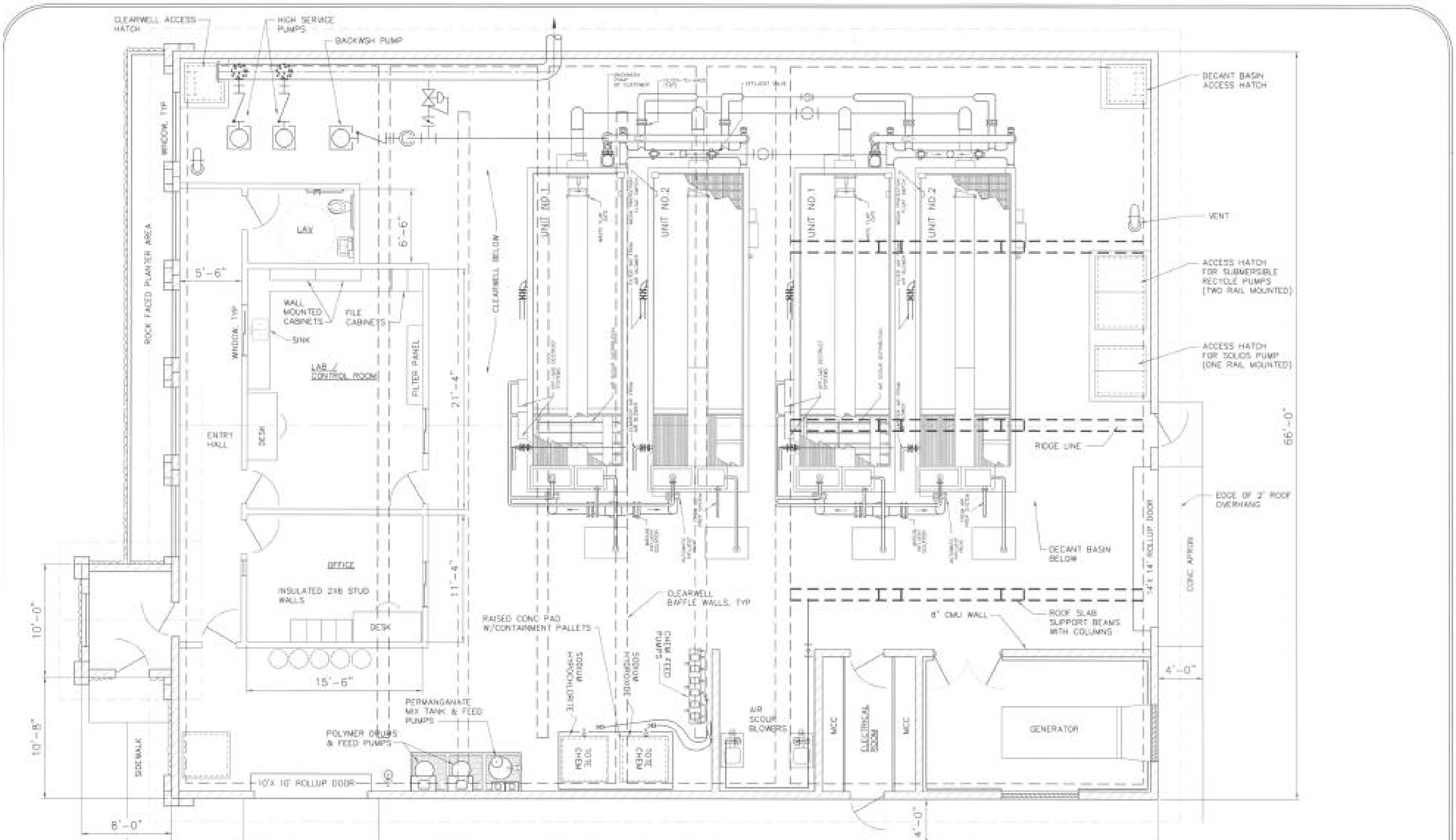

FIGURE 8-5
FLOOR PLAN
PROPOSED OZONE/GRAVITY FILTER ALTERNATIVE
SQUAW VALLEY PSD

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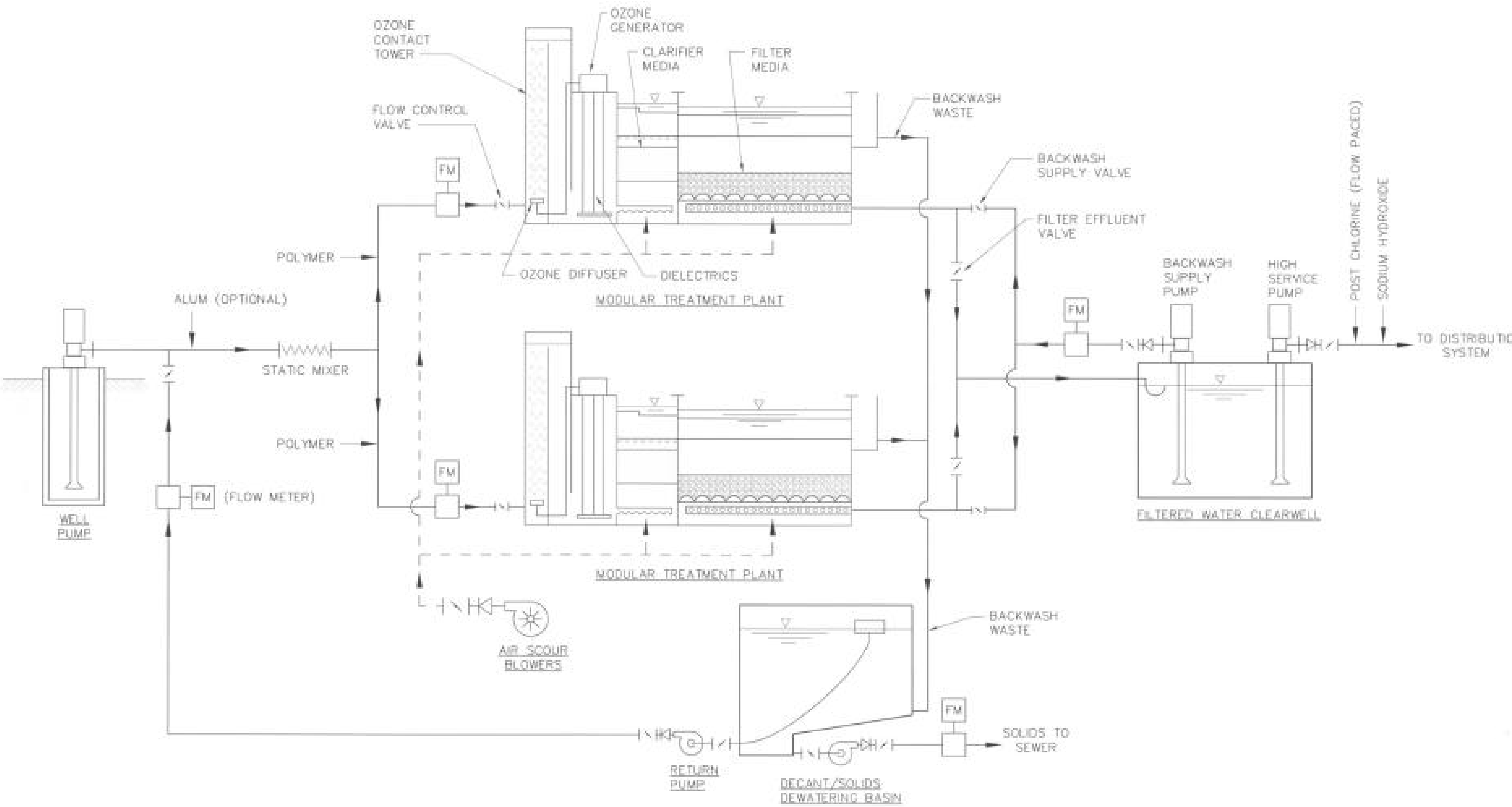

FIGURE 8-6
FLOW SCHEMATIC
PROPOSED OZONE/GRAVITY FILTER LAYOUT ALTERNATIVE
SQUAW VALLEY PSD

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Consulting Engineers

In addition to converting soluble iron and manganese into filterable floc, the ozone will also oxidize arsenic to arsenate and, coupled with alum coagulation, provide complete removal of this contaminant. Ozone it is an extremely powerful disinfectant and would also effectively inactivate cryptosporidia, giardia or any other pathogenic organism that may be present in the water. No additional treatment beyond the ozone oxidation followed by gravity filtration would be needed with this alternative to treat either a surface water source or groundwater under the influence of surface water.

Figure 8-5 is a detailed floor plan for the conceptual design based upon the U.S. Filter Trizone modular treatment units. The overall layout of the building is similar to that of the pressure filtration alternative; however, to accommodate the larger equipment, the building is 66 feet in width, 6 feet wider than the pressure filtration alternative. The overall length of the building, however, is the same.

There are major differences in the building between the two alternatives. The building for this alternative would require below-grade basins for both finished water and for backwash wastewater. High service pumps to deliver water to the distribution system and a dedicated backwash pump would also be necessary. The space requirements for the lab/control room, the operator's office, an area to store and apply chemicals and the room for an emergency generator, are similar to the other alternative.

Figure 8-7 is a layout of the facilities designed around the filtration alternative. The treatment plant would be located immediately north of the existing administration building, between the north property line and the administration building. Note that a portion of the administration building would have to be demolished to accommodate the 66-foot wide building at this location. Access for installation of future equipment would be from the east end of the building. Chemical deliveries would be made at the west end of the building. Because below-grade storage basins for filtered water and waste backwash water would be located beneath the building, no other structures would be constructed on the site to support this alternative.

The estimated costs for this alternative are presented in Table 8-3. These costs are based upon the conceptual design developed for the alternative. The considerably greater building costs are due to extensive excavation and the concrete involved in the construction of the below-grade clearwell and backwash decant basin.

A proposal for a Trizone modular treatment plant consisting of two 700-gpm (1 mgd) treatment units was obtained from U.S. Filter. The estimated cost of the filtration equipment was $605,000. An additional $120,000 was estimated for equipment installation.

The estimated cost for flow meters of $20,000 matches the cost estimate for the pressure filtration alternative. Chemical feed equipment would be somewhat less extensive as reflected by the estimated cost of $25,000 for this equipment. At an estimated cost of $123,000, the cost of the pumping facilities for this alternative exceeds that of the pressure filtration alternative. The building is of similar design, but is slightly larger which is reflected in the greater cost.

Table 8-3. Estimated Construction Cost
2 mgd Ozone/Gravity Filtration Alternative [(a)]

Component	Estimated Construction Cost
Site Clearing/Preparation	35,000
Relocate Site Drainage	30,000
Excavation and Backfill	85,000
Concrete Below-Grade Clearwell	465,000
Concrete Below-Grade Backwash Decant Basin	215,000
Paving and Surfacing	45,000
Concrete Floor Slab	90,000
Filtration Equipment	605,000
Install Filtration Equipment	120,000
Filter Facing Piping and Valves	85,000
Flow Meters	20,000
Yard Piping	40,000
Chemical Feed Equipment	25,000
High Service Pumps	48,000
Backwash Pump	35,000
Reclaimed Water Return Pumps	25,000
Sludge Discharge Pumps	15,000
Building Superstructure	230,000
Building Interior Walls	105,000
Building Roof	100,000
Windows	8,000
Roll-up Doors	15,000
HVAC	85,000
Furnishings/Cabinets	30,00
Painting	75,000
Interior Doors	12,000
Instrumentation	110,000
Electrical	260,000
Emergency Generator	120,000
Contingency @ 25%	783,000
TOTAL ESTIMATED CONSTRUCTION COST	**3,916,000**
Engineering, Legal, Administrative @ 20%	784,000
Total Capital Cost	**$4,700,000**

(a) Costs are in 2001 dollars

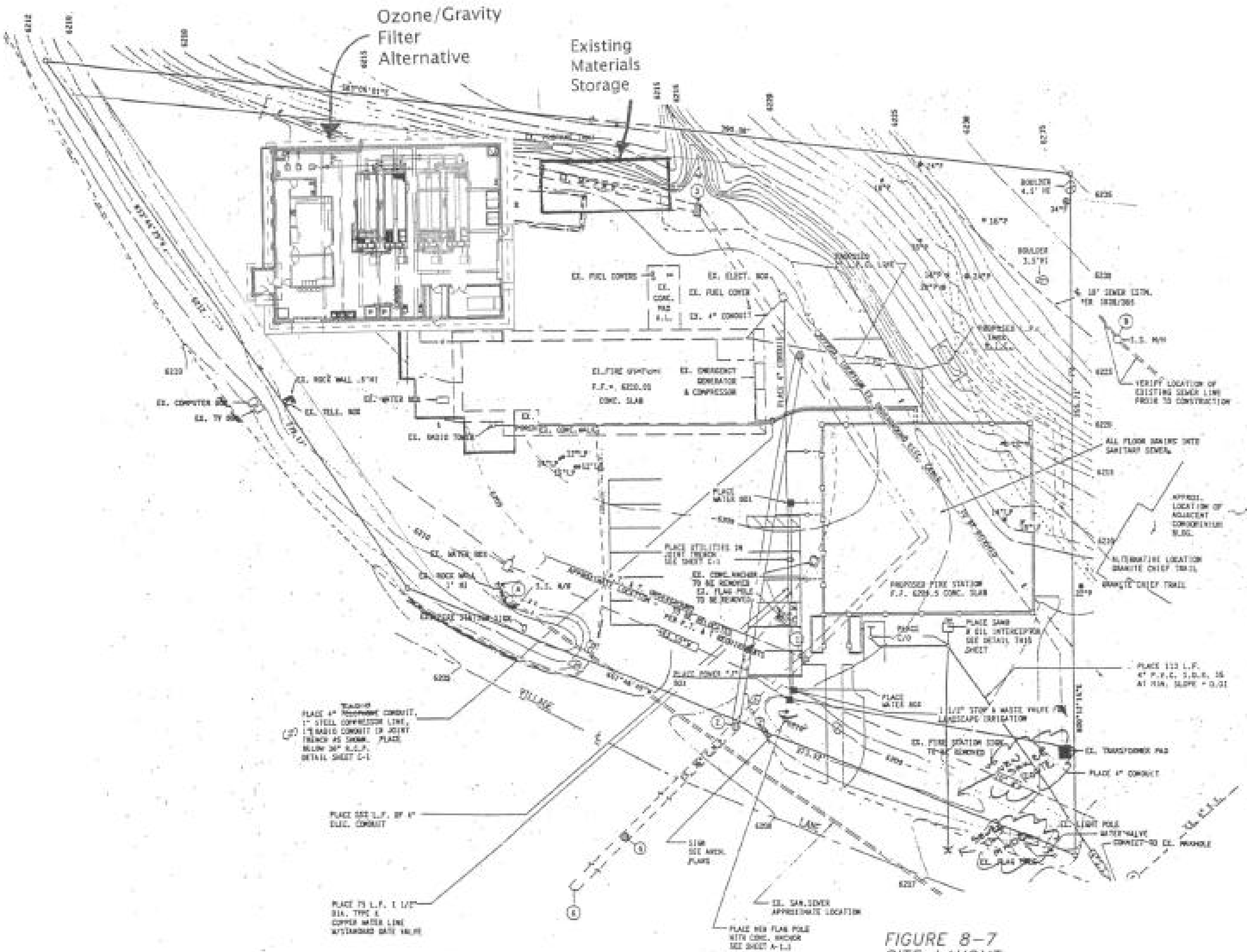

FIGURE 8–7
SITE LAYOUT
PROPOSED OZONE/GRAVITY FILTER LAYOUT ALTERNATIVE
SQUAW VALLEY PSD

SPH associates
Consulting Engineers

No dedicated filtered water transmission line to the major system storage reservoir would be required with this alternative. The high service pumps would discharge directly into the distribution system through a connection at the plant.

Overall instrumentation and electrical costs will be greater with this alternative principally because of the increased number of pumps. Electrical and instrumentation costs do not include any SCADA facilities.

The total estimated cost of the ozone/gravity filtration alternative of $4,700,000 is approximately $1,250,000 higher in cost than the pressure filtration alternative. As is apparent from the conceptual layout, the flow schematic and the description of the process, the ozone/gravity filtration alternative has substantially greater treatment capability, but is equipment intensive and operationally more complex. Because the alternative is substantially more expensive, the additional cost cannot be justified or warranted unless it is anticipated that a surface water source could be developed, or the groundwater will be of substantially poorer quality than that anticipated.

WATER PRODUCTION COST PROJECTION

A very preliminary projection of expected water production costs was prepared for the two alternative processes. Water production costs include operational and maintenance labor, power for the process and pumping water. Power is also included for heating and lighting the building. Chemicals for removing iron and manganese and maintenance materials, which include disposables, small replacement parts, lubricants, etc., are two other operation and maintenance cost categories. A sinking fund for equipment replacement is included under the maintenance materials category.

Table 8-4 provides a summary compilation and comparison of the water production cost categories associated with both alternatives. The total cost per million gallons of water treated and pumped into the system ranges from $367 to $729 per million gallons for the pressure filtration and the ozone/gravity filtration alternatives, respectively.

Operation and maintenance labor costs are greater for the ozone/gravity filtration alternative because this alternative involves more complex treatment equipment than the pressure filtration alternative.

The power cost category is made up of process costs, which include raw water and finished water pumping, surface wash and backwash pumping, production of ozone and operation of other process related equipment. Building heating, lighting and ventilating is a significant source of power consumption. This parameter is related to building square footage. For this analysis a value of 50-kilowatt hours (kwh) per square foot per year obtained from energy projection tables was used. Power costs were determined using a projected rate of $0.15 per kwh.

The estimated costs of chemicals are similar for both alternatives. Chemicals involved include chlorine, potassium permanganate, filter aid and sodium hydroxide for corrosion control. Ozone, although a treatment chemical, is included in the process energy cost category.

Table 8-4. Water Production Cost Summary Comparison Pressure Greensand Filtration and Ozone Gravity Filtration Alternatives

Cost Category	Cost Per Million Gallons of Production	
	Pressure Filtration Alternative, dollars	Ozone/Gravity Filtration Alternative, dollars
Operation and Maintenance Labor	57	73
Energy-Process	107	429
Energy-Building HVAC	43	47
Treatment Chemicals	70	65
Maintenance Materials	80	107
Solids Disposal Costs [(a)]	10	8
Total Cost Per Million Gallons Processed	367	729

(a) Cost based upon current sewer discharge rate of $4.00 per 1,000 gallons

The most feasible method of disposal of residual solids would be to discharge them to the sanitary sewer. T-TSA permits this practice if the wastewater containing the solids removed in the treatment process do not have a detrimental impact on the sewer treatment plant. Presently, the rate is $4.00 per 1,000 gallons of wastewater discharged into the system.

During normal operation, and at the plant design capacity of 2 mgd, it is anticipated that about 2,300 to 4,500 gallons per day of wastewater will be discharged to the sewer. This stream (after being concentrated in the decant/solids dewatering tank) will be a very dilute suspension of iron and manganese floc with a suspended solids concentration of about 1,000 to 2,000 mg/L. This suspension will have the characteristics of wastewater-treatment process activated-sludge mixed liquor and will not clog sewers.

The disposal costs using the present rate of $4.00 per 1,000 gallons are not significant, about $10 per million gallons of water processed. These costs could become even less significant if a more favorable disposal rate is offered to the District by T-TSA.

Also included in Table 8-4 are projected production costs for disposal of solids produced by the ozone/gravity filtration process alternative. Using ozone rather than chlorine and potassium permanganate, it is expected that slightly less solids will be produced by this process compared to the pressure filtration alternative. This reduction in solids production is reflected in the lower disposal cost per million gallons of production noted in Table 8-4. Projecting about 2,000 gallons per day for the ozone/gravity filtration process alternative yields a disposal cost of $8 per million gallons of disposal water.

Maintenance materials are similar for both alternatives and include a capital facilities replacement fund increment. This replacement fund is based upon 2 percent of the capital facilities costs proportioned to plant production.

TREATMENT PROCESS RECOMMENDATION

Evaluation of the two most appropriate treatment process alternatives indicates that a treatment facility designed around the pressure greensand filtration process for iron and manganese removal would be the preferred alternative. It appears that the treatment requirement is primarily for iron and manganese removal and pressure filtration is substantially less costly than the other alternatives. This treatment process can also remove arsenic should levels in the groundwater rise above the MCL. Only if a surface source becomes available would the ozone/gravity filtration modular treatment alternative process be more suitable than the recommended process. However, the pressure greensand filtration system can be upgraded with UV treatment of the filtered water permitting DOHS to approve the use of the process for treating a surface source or a groundwater under the influence of a surface source. The estimated cost to add UV sterilization to the pressure greensand filtration treatment system would probably be about $350,000. Space has been provided in the conceptual facility layout to accommodate future UV disinfection equipment. Operation and maintenance costs favor this alternative over the ozone/gravity filtration alternative by a wide margin.

After the District selects the preferred treatment facilities around the pressure greensand filtration iron removal treatment alternative, the next step would be to refine the conceptual design to be certain that it incorporates all features needed by the District to meet specific phasing requirements. All safety issues associated with chemical storage and handling would be addressed at this time. The refined design would also be developed in greater detail and a refined cost estimate prepared that would be representative of the expected total project cost for this facility. The total project cost would take in account any specialized or specific architectural features that would have to be incorporated into the project to meet local requirements and the discharge connection fee to T-TSA. Also, although it is not anticipated that the treatment water will have any difference in taste or odor compared to current supplies, the water quality from the new wells will be evaluated to ensure acceptance of the treated product by the District’s customers. The selection of new well locations will be predicated on providing water that, while high in iron and manganese, is still of good quality for domestic use.

SECTION 9. RECOMMENDED WATER SUPPLY AND TREATMENT FACILITIES AND MANAGEMENT PLAN

Section 7 – Alternative Water Supplies identified the required supplemental water supply in terms of the maximum day production needed in gallons per minute and the annual water supply required in acre-feet on a sustained basis during a critically dry year. The three water supply alternatives were evaluated and compared. The recommended water supply and treatment facilities include additional wells located in Squaw Valley and an iron and manganese treatment plant located adjacent to the existing District office building. This section provides additional discussion on the recommended alternative and a phasing plan for implementing the facilities. In addition, a recommended groundwater management plan is presented.

RECOMMENDED WATER SUPPLY AND TREATMENT FACILITIES

As stated previously, the District is expected to need an additional 1,600 gpm in water production capacity to meet buildout water demands. The annual groundwater supply requirements at buildout is estimated to be 1,605 af. The Squaw Valley Groundwater Supply Alternative provides the needed production capacity to supplement the existing wells to result in sufficient supply capacity to meet the maximum day demand at buildout with the largest producer out of service. The groundwater basin has been shown to have a sustainable yield of about 1,524 af with the District's shared amount of it being 1,100 af. The groundwater supply alternative develops the full sustainable yield of the basin. To meet build out demands, additional supply from outside the valley would be needed. The Squaw Valley groundwater supply will be adequate if the future development is limited or the District's conservation ordinance is enforced in critically dry years to reduce demands by 31 percent. It is anticipated that five new wells will be constructed to provide the needed production capacity. In addition it is assumed that the idle capacity of the Resort Wells 18-2 and 18-3 can be made available to the District. This recommended plan is to be used as a guide and can only be implemented if well sites can be acquired and the expected production is developed.

A supply connection with Alpine Springs County Water District (ASCWD) is recommended at the time the Homesites at Squaw Creek #2 subdivision is constructed. The intertie with ASCWD could be used under emergency conditions or provide water to the District on a regular basis if idle supply capacity is available and an agreement can be negotiated between the districts. The sizing of the intertie pipeline would be determined at the time the subdivision project moves ahead and the amount of water available from ASCWD is known.

It is also recommended that a water rights application be filed immediately for a surface water diversion from the Truckee River. This would establish a placeholder for this supply so the District will have some flexibility in the future should conditions change. Additional investigations should be performed to determine how the reliability of the supply could be enhanced to provide benefit to Squaw Valley during drought years. The treatment plant can easily be retrofitted with UV disinfection equipment to be able to treat this supply.

The recommended treatment plant is a pressure greensand filtration system located in a new building in back of the existing district office complex. The plant is envisioned to have a buildout capacity of 2 mgd, with the building sized to accommodate treatment facilities capable of treating up to 4 mgd. This will provide assurances of being able to build a 4-mgd treatment plant if the need arises to treat more of the groundwater supply or a surface water source. The 2-mgd treatment plant is comprised of two pressure filters with a treatment capacity of 1 mgd each. This configuration lends itself to phasing the facility by installing one filter first and waiting until demands increase before installing the second filter and associated equipment.

Recommended Improvements

The recommended facilities to be constructed to meet buildout demands were identified in Section 7. The facilities and estimated capital cost are shown on Table 9-1. These costs do not include the cost of the land for the wells or the cost for pipeline easements not in public rights-of-way. These costs, when they are identified, will need to be added to the costs shown in Table 9-1.

Table 9-1. Recommended Water Supply Facilities and Estimated Capital Costs [a]

Item	Unit Cost, Dollars	Estimated Cost, Dollars
2 mgd Water Treatment Plant for Iron and Manganese Removal	L.S.	2,875,000
5 New Wells	425,000	2,125,000
3,300 feet of 4-inch pipe	40	132,000
3,500 feet of 6-inch pipe	60	210,000
2,100 feet of 8-inch pipe	80	168,000
1,700 feet of 10-inch pipe	100	170,000
Total Construction Cost		5,680,000
Engineering, Legal, & Admin Costs @ 20%		1,136,000
Total Project Cost		6,816,000

(a) Costs are in 2001 dollars

Phasing of Improvements

The facilities included in the recommended supply plan can be phased as demands increase. The initial facility that should be constructed is the drilling of the replacement well for Well 4R. Wells at this location have proven to be good producers with good water quality that does not need treatment. It is expected that this well will be similar to Well 5R in terms of production and will provide the District with additional pumping capacity and reliability in meeting peak demand periods with the largest producer out of service. Should it be found that Well 4RII produces groundwater under the influence of surface water, it can be connected to the treatment plant for treatment and disinfection. UV disinfection equipment would need to be added at the treatment facility as discussed previously in Section 8.

The next activities to be undertaken include further exploration and testing of potential well sites to identify the next set of wells to be added to the system. Test Wells 4 and 5 should be test pumped to identify the source of poor water quality. Water bearing strata should be isolated and water quality samples obtained. The results of the pump testing and water quality testing should identify the feasibility of developing production wells at these locations of suitable quality for domestic purposes.

The potential for the use of the Resort at Squaw Creek Wells 18-2 and 18-3 should be investigated. The wells should be retrofitted with level monitoring equipment and the pumping amounts and drawdown should be monitored to ascertain the production capabilities and potential for use as a year-round supply for the District.

A study should be performed to identify the improvements that would be necessary to include the production from the horizontal wells near the 4th Fairway Well into the supply system. The horizontal wells' production could be pumped into the existing distribution system pipeline that serves the Resort or added to the production from the 4th Fairway Well delivered to the water treatment plant.

The treatment plant can be phased to provide just 1 mgd (700 gpm) of capacity with room in the building to provide another pressure filter later. The cost savings for phasing the treatment would only be about $500,000, which is the cost of a pressure filter; associated piping and chemical feed system. At least two additional wells will need to be added to provide the 700 gpm of production capacity, although, the treatment plant could begin operation with only one well. The most likely wells to pursue initially are the Condo Well and the 4th Fairway Well. The Condo Well was drilled in 1992 and will require some cleanup and development work before a pump and motor can be installed. The 4th Fairway Well will need to be redrilled using a larger casing.

In addition to these wells, the connections to the Resort Wells 18-2 and 18-3 should be made. The District will need to develop an agreement with the Resort at Squaw Creek for the use of the idle well production capacity and an easement for the pipelines to deliver the water to the eastern edge of the parking lot near Well 5R. The rest of the pipeline easements will also need to be acquired to connect the pipeline to the public rights-of-way along Squaw Valley Road. The combination of the two new wells and the idle capacity of the Resort wells will provide at least 700 gpm of pumping capacity to the first phase of the treatment plant. A total of 8,500 feet of the 4-inch to 10-inch pipelines shown on Figure 7-2 will also need to be constructed in the initial phase.

The treatment plant would be expanded when demands increased and the two new wells in the western parking lot are needed. The piping and much of the ancillary support structure for the new pressure filter will have been constructed as part of the initial phase of the treatment plant project. The wells will need to be sited, test drilled and the property acquired prior to the construction of the wells. The 1,300 feet of 6-inch and 8-inch pipelines connecting the wells to the treatment plant will also need to be constructed.

The recommended plan provides the District with flexibility for developing the needed supplemental water supply in terms of the number and location of wells required to meet future demands. The identified locations are based on previously drilled test holes. If other sites are found to be better well locations and can be developed to produce 300 to 400 gpm while maintaining or increasing the basin's sustainable yield, then they can substitute for the any of the wells. Having a pipeline through the golf course provides opportunities to develop wells along that route if reasonable water quality and production can be developed. Supply from additional horizontal wells along the valley's south side or drilled wells in the east end of the valley could be delivered to the treatment plant via the golf course pipeline. The area between the Resort at Squaw Creek and the existing well field area has been identified as the most promising target area for future successful production wells.

GROUNDWATER MANAGEMENT PLAN

Groundwater is a precious resource in Squaw Valley that is and will continue to be relied upon as the water supply for the valley. The objective in developing a groundwater management plan is to provide a long-term strategy for sustainable groundwater basin use for all entities relying on this water supply. Development and implementation of the recommended groundwater management plan will allow the entities pumping from the basin to effectively manage the basin with respect to the quantity and the quality of the water pumped. The goals of the plan are to keep the pumped amounts within the sustainable yield of the basin and protect the wells from potential contamination. The entities should work closely to manage this resource.

The groundwater model developed for this project provides the District with a tool for making management decisions on the use of the groundwater resource in Squaw Valley. It has been shown to reasonably simulate the geohydrology of the basin. It is the best tool available for estimating the effects of various pumping and recharge scenarios, and should be used for planning future groundwater basin management. Pumping rates from existing wells, placement of new wells, and effects of pumping on streamflow can all be studied with the existing model The use of the model will improve any future planning studies.

As with all groundwater models, additional data will help validate the results and direct modifications to uncertain model parameters. Data that will be particularly helpful include measured streamflow entering and leaving the basin and additional water level data from or near the pumping wells. The collection of this data will allow more accurate calibration of the basin model, particularly in terms of the annual water balance.

As described in Section 6 – Groundwater Model Simulations to Estimate Sustainable Yield the definition of sustainable yield is the maximum amount of pumping that can be pumped from the groundwater basin during a critically dry year without significantly impacting water levels in existing wells. The definition of significant impact to existing wells is not lowering the pumping levels to below the top of the perforations. A pumping scenario has been identified that maximizes monthly pumping of existing wells without lowering the pumping levels below the top of the perforations. This monthly pumping pattern should be used by the District as a guide for managing the pumping from the groundwater basin.

The water levels in each well should be monitored and pumping adjusted so levels remain above the top of the perforations. During dry years, the need for increased water conservation measures should be coordinated with ongoing review of precipitation, stream flow and pumping levels and amounts. The District has recently enacted a water conservation ordinance that calls for restrictions on the use of water in the event of any threatened or existing water shortage. The ordinance also provides for the implementation of a mandatory reduction in demands by 20 percent or more during a critical water supply shortage. In any year that experiences precipitation that is below normal amounts, the District should review the condition of the groundwater basin and the need for increased water conservation. The results of this study have shown that the water supply in Squaw Valley is a limited resource. The initiation of water use restrictions should be considered in the fall of each year that the groundwater levels are at or below the top of the perforations in Well No. 2. The use of groundwater for snowmaking should also be critically reviewed if the groundwater levels have not substantially recovered prior to the onset of the ski season. The model is currently being updated with the current year's hydrology and expected pumping amounts to assist in managing the basin this summer. This year has had the least amount of precipitation ever recorded at the Squaw Valley Fire Station in its 37 years of record. Decisions will be made on managing the groundwater resource after the model results are reviewed.

A program to obtain additional information for the model and to manage the basin has been identified. The program includes monitoring water levels of all pumping wells, establishing stream gages on Squaw Creek and providing groundwater protection by abandoning unneeded existing wells and establishing a monitoring network in the western basin. The benefits to the District and Mutual Water Company from a more complete monitoring program include the establishment of an early warning system in case of groundwater contamination from spills in the production well capture zone as described in Section 3, and to obtain more information for the refinement of the groundwater model as described in Section 4. The monitoring network in the western basin would include real-time water level monitoring in all production wells and a ring of monitoring wells upstream of the production wells where water quality samples could be collected and tested quarterly to identify the presence/absence of containments that may have entered the groundwater. The early warning monitoring and testing will give the District and Mutual time to react to the contamination and maintain adequate water supplies with wholesome quality for their customers. The information obtained from the monitoring program will also be useful in updating and improving the groundwater model with more complete information on groundwater levels in response to pumping and stream flows entering and leaving the valley. An updated model would provide additional confidence in using the model for making decisions on management of the basin and to verify and refine the estimate of sustainable yield of the resource. The recommended activities to be undertaken as part of the groundwater management plan are listed below.

1. Identify, locate and map test wells and monitoring wells in the western end of valley.
2. Determine which wells may be used for monitoring and which need to be abandoned.
3. Complete the well SCADA system to monitor pumping and level at all wells. Expand system to other pumping wells in valley, if possible.
4. Properly abandon all unnecessary wells and equip others for monitoring levels or periodic sampling to identify possible contamination plume movement.
5. Identify other locations for additional monitoring wells, construct wells and install monitoring equipment.
6. Install three stream gages within Squaw Creek; one on each major branch entering the west end of the valley and one at the upstream side of the Squaw Valley Road bridge.
7. Establish an ongoing monitoring program for the collection of surface water and groundwater data and to monitor quality of water in the production well's capture zone. Update the groundwater model when sufficient data has been collected.

8. Prepare a groundwater management report consistent with the requirements of AB 3030 and submit it to the State Department of Water Resources. Apply for grant funds to support ongoing groundwater management program activities.
9. Develop public outreach and education program as described in the Groundwater Protection Plan in Section 3.

The cost of the above-described activities is estimated to be about $250,000. The implementation of the groundwater management plan will directly benefit all users of the Squaw Valley groundwater basin. While the District and Mutual are not required to develop a groundwater management plan as defined by AB 3030, the State's groundwater management planning legislation, the implementation of a plan provides a definitive program for the collection and use of monitoring data to help ensure the maintenance of the quality and quantity of the local groundwater resource. With this program, informed decisions in managing the available groundwater can be made to assure an available supply in the future.

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APPENDIX

Groundwater Model 2003 Update

MEMORANDUM

To: Mr. Rick Lierman/SVPSD

From: Derrik Williams

Project: Squaw Valley Groundwater Model 2003 Update

Date: June 24, 2003

Subject: Groundwater Model Update and Sustainable Yield Analysis

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Section 1
Introduction and Approach

The Squaw Valley Groundwater Model has been modified and updated based on data not available during the initial development and calibration. These data include numerical data such as construction dewatering drawdown, and supplementary non-numerical data such as an assumed depth of the Plumpjack well. This update was not intended as a complete re-evaluation of the groundwater model, rather it was an attempt to simply incorporate new data.

This memorandum details the data incorporated into the model, the model calibration and verification, presents a new evaluation of the sustainable yield, and outlines further steps that should be undertaken to improve the groundwater model.

Section 2
Data Incorporated into the Updated Model

Both numerical data and observational data were incorporated into the updated groundwater model. The new data included the following:

A map of the historical location of Squaw Creek. Mr. Carl Gustafson of Squaw Valley, California provided this hand-drawn map. The map shows the location of Squaw Creek in the western portion of the basin before the parking lot was built. Data from this map was used to delineate a high conductivity zone corresponding to the streambed deposits associated with the historical creek location

Oblique photograph of Squaw Valley. Mr. Paul Arthur of Squaw Valley, California Valley provided this photograph. The photograph shows the location of Squaw Creek prior to the development of Squaw Valley. This photograph was used to confirm the creek location information included on the map discussed above.

Detailed Mapping of Squaw Valley East of the Parking Lot. West Yost & Associates conducted a detailed mapping of ground surface elevations in the meadow and golf course area of Squaw Valley. Included in the mapping were a number of surveyed streambed elevations. These data were used to refine both the ground surface elevations and the creek bottom elevations in the model.

Groundwater levels observed during construction dewatering. Dewatering was necessary during construction of Intrawest's underground parking structures in 2002. The parking structure was included as part of the Phase II construction activities of Intrawest's new Squaw Valley Village. During dewatering, recorded drawdowns in observation wells were less than predicted by the previous model. The dewatering data, along with a preliminary model analysis of the data, are included in the *Intrawest Dewatering Impact Study Memorandum* (Williams, 2002).

Plumpjack well information. The Plumpjack well is located in the parking lot west of the Plumpjack development. The construction details of the Plumpjack well are unknown, however incidental information suggests that the basin sediments may be up to 130 feet thick at the Plumpjack well location.

Observations of the small reservoir in Olympic Lady Canyon. A small reservoir located in the Olympic Lady Canyon retains water throughout much of the summer. No data is available on potential leakage from this reservoir. The bottom of the reservoir appears to be unlined. It is reasonable to assume that some amount of water leaks from the reservoir throughout the year, providing a small source of water for the Squaw Valley Basin.

Groundwater level data from the Olympic House Loading Dock Underground Storage Tank (UST) site. Water level data from the Olympic House Loading Dock site appear to be referenced to Mean Sea Level (MSL). This allows a direct comparison of this water level data with groundwater model results. Groundwater level data for this study were derived from the *Groundwater Monitoring Report – 2nd Quarter 2002, Olympic House Loading Dock* (McGinley & Associates, 2002). Groundwater level data from four monitoring wells are plotted on Figure 1. This figure shows that most of the groundwater level data lie between 6190 and 6196 feet msl.

Section 3
Groundwater Model Modifications

Based on the interpretations and analyses of the data discussed above, a number of modifications were incorporated into the existing groundwater model. The model modifications include the following.

Honored the historical location of Squaw Creek. A high hydraulic conductivity band was added to the model, representing the gravelly streambed deposits along the historical location of Squaw Creek. The high conductivity band was added to both of the top two model layers. The location of this band is shown on Figure 2. Wells that lie in these stream deposits are generally in closer hydraulic contact with the existing Squaw Creek than other wells.

Deepened the groundwater model around Plumpjack well. Based on unverified information, the groundwater model was deepened to 130 feet around the Plumpjack well. The model was previously approximately 60 feet deep around the Plumpjack well, based on the depth of existing water supply wells and a reasonable interpretation of simple basin geometry.

Refined the ground surface and creek bottom elevations. The simulated ground elevation was modified based on the mapping provided by West Yost & Associates. Additionally, the thalweg of Squaw Creek in the meadow was modified based on the survey data. Generally, the creek at the western end of the meadow was lowered to honor the survey data.

Added a water source at the foot of the Olympic Lady Canyon. This water source represents leakage from the small reservoir in Olympic Lady Canyon. The water source was added as a general head boundary in the top layer of the groundwater model. Because there are no leakage measurements or estimates from the reservoir, we attempted to estimate the amount of leakage during calibration. As discussed in the calibration section below, the amount of leakage from the Olympic Lady Reservoir is highly uncertain.

Section 4
Groundwater Model Calibration

The groundwater model was calibrated to groundwater elevations measured between May 1992 and April 1999. This is the same period used for the original groundwater model calibration. The same period was used so that the current calibration statistics could be compared to the previous calibration statistics.

Calibration generally consisted of adjusting model parameters to fit the observed data. Very little effort was made to refine the distribution of model parameters. The obvious exception is the addition of the high hydraulic conductivity band that represents the historical stream deposits.

As part of the calibration, the leakance from Squaw Creek was redefined. The previous model divided Squaw Creek into three sections: a north fork, a south fork, and the entire reach below the confluence of the two forks. Squaw Creek, however, flows more rapidly west of the meadow than within the meadow. Streambed deposits are therefore likely coarser west of the meadow than within the meadow. This difference in streambed deposits likely affects the leakance from Squaw Creek. The reach of Squaw Creek below the confluence of the two forks was therefore divided into two sections: one section between the confluence and the meadow, and one section within the meadow. A separate stream leakance was assigned to each section.

2.1 Calibrated Parameters

The model parameters that were adjusted during calibration included the following:

- Horizontal and vertical hydraulic conductivities of the western end of Squaw Valley (Figure 3). This parameter has an important impact on the simulated water levels in Squaw Valley Public Service District Well#2. The horizontal hydraulic conductivity was set to 60 feet per day in the original model, and was increased to 120 feet per day in the current model. The vertical hydraulic conductivity was set to 0.6 feet per day in the original model, and was increased to 3.6 feet per day in the current model.

- Horizontal and vertical hydraulic conductivities of the low conductivity band in the center of Squaw Valley (Figure 4). This low conductivity zone plays an important role in the water elevation drop between Squaw Valley Public Service District Well#2 and Squaw Valley Mutual Water Company Well#1. The horizontal hydraulic conductivity was set to 2 feet per day in the original model, and decreased to 0.15 feet per day in the current model. The vertical hydraulic conductivity was set to 0.007 feet per day in the original model, and was increased to 0.01 feet per day in the current model.

- Stream leakance. The stream leakance is a measure of the ability with which groundwater can flow into and out of Squaw Creek. The two stream leakances below the confluence of the two forks of Squaw Creek were modified during the calibration. As mentioned above, the model now assigns two separate leakances to the this stretch of Squaw Creek, one leakance for the stream west of the meadow and one leakance for the stream within the meadow. The hydraulic conductivity of the streambed west of the meadow is set at 11.7 feet per day in the current model. The hydraulic conductivity of the streambed within the meadow is set at 0.24 feet per day in the current model.

- Specific yield of the western end of Squaw Valley (Figure 3). This parameter, in conjunction with the horizontal and vertical conductivities of this zone, play an important role in the simulated water levels in Squaw Valley Public Service District Well#2. The specific yield was set to 0.1 in the original model, and was increased to 0.2 in the current model.

2.2 Calibration Results

Table 1 compares the calibration results from the initial model with the calibration results from the current model. The calibration statistics are slightly better in the initial model, although the model is still considered well calibrated.

Table 1 Calibration Results		
Parameter	Initial Model	Current Model
Mean Error	-0.49	-0.40
Mean Absolute Error	1.96	2.13
Standard Deviation	2.52	2.75
RMSE	2.57	2.78

Figures 5, 6, and 7 show the measured and simulated water levels for District Well#2, Mutual Well#1, and Mutual Well#2, respectively. These figures show good correlation between the measured and simulated water levels. The greatest error appears to be associated with District Well#2 in the first two years of simulation. This may be associated with inappropriate initial conditions.

The groundwater elevations from the Olympic House Loading Dock site were not simulated directly, as the data are from the years 2000 to 2002 and the groundwater model was run only through 1999. However we assumed that the water levels at the Olympic House Loading Dock site are relatively constant over time, and the groundwater elevations observed since 2000 should be similar to those that existed in the mid 1990s. Figure 8 shows the simulated water levels around the Olympic House Loading Dock site. The graph shows that water levels vary between 6192 feet msl and 6200 feet msl, with an average water level of around 6197 feet msl. This is somewhat higher than the observed groundwater levels shown on Figure 1. Attempts to lower the simulated water levels were relatively unsuccessful. One reason the water levels are so high is the addition of water from the Olympic Lady reservoir. This water flows directly towards the Olympic House Loading Dock site, raising the water levels there. Further analysis is needed to simulate the flow regime in the Olympic House Loading Dock site area.

Section 5
Dewatering Simulation

Further model verification was undertaken using data from the 2002 Intrawest construction dewatering project. The simulation outlined in the *Intrawest Dewatering Impact Study Memorandum* (Williams, 2002) was repeated with the new model.

Significant drawdown was observed in two monitoring wells during the construction dewatering. Dewatering Well#9 showed between nine and ten feet of drawdown during construction dewatering. Dewatering Well#11 showed between 12 and 16 feet of drawdown during construction dewatering. The simulated drawdown for both wells was between nine and ten feet. This is a reasonably accurate simulation of the drawdown given the regional nature of the model. Additional accuracy might be obtained with additional calibration. Of particular note, both wells 9 and 11 currently lie within the high conductivity stream deposits displayed on Figure 2. Moving Dewatering Well#11 outside of the stream deposit may increase the drawdown, more accurately simulating the observed drawdown.

Section 6
Sustainable Yield Analysis

The updated groundwater model was used to simulate the Squaw Valley Groundwater Basin sustainable yield. Three sets of sustainable yield analyses were conducted. The first analysis estimated the sustainable yield available from the existing production wells. The second and third analyses estimated the sustainable yield from both existing and new wells. The difference between the second and third analyses are the assumptions embedded in the analyses. The second analysis incorporates two important assumptions used in previous analyses:

1. The minimum safe simulated water level in well District Well#2 is 6177 feet msl.
2. The relative pumping distribution between production wells is constant throughout the year. In other words, if a certain well provides 21% of the production in January, it also provides 21% of the production in May.

The third analysis differs from the second analysis by relaxing these two assumptions.

The first two analyses build on the sustainable yield analysis detailed in the Draft Squaw Valley Groundwater Development & Utilization Feasibility Study Report (West Yost & Associates, 2001). Much of the following discussion is based on the discussion in that document.

Sustainable yield is defined as the maximum amount of water that can be pumped from the groundwater basin during a critically dry year without significantly impacting the pumping water levels of existing wells. After reviewing precipitation records from the Squaw Valley Fire Department, we determined that the 1993-1994 water year could be considered a critically dry year. Only water year 1976-1977 was drier than 1993-1994. Based on this analysis, 1994 was chosen as the dry year to simulate for the sustainable yield analysis.

The rational for substantiating the water levels which significantly impact existing wells is detailed in the Draft Squaw Valley Groundwater Development & Utilization Feasibility Study Report (West Yost & Associates, 2001). In essence, minimum acceptable water levels are those that remain above both the pump intake and the top of the perforations in each well. Table 2 lists the minimum acceptable water levels for each of the existing production wells. Minimum acceptable water levels for two assumed well efficiencies are included on Table 2. Because the groundwater model does not account for well efficiencies it was necessary to assume well efficiencies to estimate true pumping levels in production wells.

Table 2 **Minimum Acceptable Well-Bore Water Levels**				
Well	Ground Elevation, ft	Elevation of Top of Perforations, ft	Minimum Acceptable Simulation Water Level Elevation, ft	
			70% Efficiency	50% Efficiency
District#1	6202.6	6125.6	6143.6	6155.6
District#2	6202.2	6167.0	6173.0	6177.0
District#3	6201.9	6125.0	6139.7	6149.5
District#5	6199.3	6133.3	6147.4	6157.0
Mutual#1	6190.5	6130.5	6149.3	6158.8
Mutual#2	6195.0	6160.0	6163.7	6169.5

Pumping scenarios for the first two sustainable yield analysis were based on the approach outlined in the Draft Squaw Valley Groundwater Development & Utilization Feasibility Study Report (West Yost & Associates, 2001).

6.1 MAXIMUM PUMPING OF EXISTING WELLS

A series of groundwater model simulations were conducted to estimate the basin's sustainable yield using the existing production wells. The simulations used the hydrology of 1994 to simulate critically dry conditions. Two consecutive critically dry years were simulated in each run.

The initial estimates of monthly pumping rates using only existing wells reflected the maximum monthly yield of each existing production well. These rates are shown on Table 3.

Table 3
Maximum Monthly Pumping Rate for Initial Simulation (Acre-feet)

Well	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
District#1	19.66	17.47	18.41	16.22	16.85	24.65	33.07	33.70	25.58	17.16	11.86	17.16	251.78
District#2	17.14	15.23	16.05	14.14	14.69	21.49	28.83	29.38	22.30	14.96	10.34	14.96	219.50
District#3	6.05	5.38	5.66	4.99	5.18	7.58	10.18	10.37	7.87	5.28	3.65	5.28	77.47
District#5	20.16	17.92	18.88	16.64	17.28	25.28	33.92	34.56	26.24	17.60	12.16	17.60	258.24
Mutual#1	6.00	5.00	5.50	5.00	7.50	11.00	15.50	15.00	12.50	8.00	4.50	5.50	101.00
Mutual#2	6.00	5.00	5.50	5.00	7.50	11.00	15.50	15.00	12.50	8.00	4.50	5.50	101.00

As with the original groundwater model, the maximum pumping rates shown on Table 3 could not be sustained during two consecutive critically dry year. A series of simulations were then performed to determine the maximum acceptable pumping rates. Each simulation maintained the relative pumping distribution used in the base simulation, but the total amount of water pumped was reduced.

The simulation that maintained water levels near the minimum acceptable levels established on Table 2 for two consecutive critically dry years simulated pumping at 70% of the base case simulation. This corresponds to an annual yield of approximately 706 acre-feet. Although this number is lower than previous estimates of sustainable yield, it is supported by recent conversations with districtstaff. During the late summer of 2002, the District lowered the pumping rate on District Well#2 to prevent the water level from falling below the minimum acceptable level. This happened during a relatively dry year, while the District was on course to pump between 600 and 700 acre-feet.

It is worth noting that the minimum acceptable water level of District Well#2 is the controlling factor on the amount of water that can be extracted during a critically dry year. District Well#2 has the highest minimum acceptable water level of all the existing wells, and it is the first well to be impacted by a lowered water table. If the minimum acceptable water level of District Well#2 could be lowered, more water may become available.

6.2 Maximum Pumping of Both Existing and New Wells under Previously Used Assumptions

A series of groundwater model simulations were conducted to estimate the basin's sustainable yield using both existing and new wells. As with the previous simulations, two consecutive critically dry years were simulated in each run. The 1994 hydrology was used to simulate the hydrology of a critically dry year.

The initial estimate of additional wells that could be added to the existing wells was derived from the Draft Squaw Valley Groundwater Development & Utilization Feasibility Study Report (West Yost & Associates, 2001). Based on work conducted for that report, the following modifications were initially made to the basin pumping:

- Pumping from the Resort at Squaw Creek wells 18-2 and 18-3 was increased to take advantage of the unused capacity of these wells.
- Pumping from the Mutual Well#1 was transferred to the Condo Well
- Additional pumping was added at the 4th Fairway well
- Pumping from District Well#3 was transferred to a portion of the unused capacity in Well 18-3
- Two new wells were added at the western end of the Valley

Initial simulations showed that the basin could not sustain all of the pumping. One new well was eliminated from the western end of the basin, and the pumping rate of the other new well was adjusted to minimize impacts to existing wells. The final pumping rates for the sustainable yield using both new and existing wells is shown in Table 4.

Table 4 Maximum Pumping Rates for Simulation with Both Existing and New Wells													
WELL	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Totals
Condo	5.10	4.25	4.68	4.25	6.37	9.35	13.18	12.75	10.63	6.80	3.83	4.68	85.85
Mutual#2	4.20	3.50	3.85	3.50	5.25	7.70	10.85	10.50	8.75	5.60	3.15	3.85	70.70
18-1	1.70	0.00	0.00	0.00	1.70	1.70	1.70	1.70	1.70	1.70	1.70	1.70	15.30
18-2	17.04	4.57	4.81	4.24	14.61	18.35	20.55	20.71	18.59	13.84	15.00	16.39	168.70
18-3	35.25	18.12	19.19	16.52	17.06	37.05	45.58	46.65	37.12	17.59	42.26	47.59	379.97
District#1	13.76	12.23	12.89	11.36	11.79	17.25	23.15	23.59	17.91	12.01	8.30	12.01	176.25
District#2	12.00	10.66	11.23	9.90	10.28	15.04	20.18	20.56	15.61	10.47	7.24	10.47	153.65
District#3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
District#5	14.11	12.54	13.22	11.65	12.10	17.70	23.74	24.19	18.37	12.32	8.51	12.32	180.77
4th Fairway	5.70	3.78	4.36	3.71	4.45	6.55	8.73	8.91	6.72	4.45	4.22	5.61	67.19
District #4R	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
New-1	4.18	3.71	3.91	3.45	3.58	5.24	7.03	7.16	5.44	3.65	2.52	3.65	53.50
Totals	113.04	73.37	78.14	68.58	87.19	135.93	174.69	176.72	140.83	88.43	96.72	118.26	1351.89

As shown on Table 4, the sustainable yield of the basin using both new and existing wells is approximately 1,352 acre-feet annually. Of this, 261 acre-feet are assumed to be used by the Resort at Squaw Creek for golf course irrigation and snow making, leaving 1,091 acre-feet annually available for potable use by the District and the Mutual.

6.3 Maximum Pumping of Both Existing and New Wells Under New Assumptions

As stated at the beginning of this section, we conducted an additional sustainable yield analysis by relaxing two important assumptions contained in the previous analysis. This third analysis again looked at the sustainable yield available from both existing and new wells. The significant assumptions changed during the third analysis included:

1. We assumed that the upper 15 feet of perforations in District Well#2 can be sleeved, effectively lowering the acceptable simulated water level in that well. As shown in Table 2, the previously assumed minimum water level in District Well#2 was 6177 feet msl. The new assumption yields a minimum water level of 6162 feet msl for District Well#2.

2. We assumed that wells could provide different percentages of water throughout the year. In the previous analysis, the pumping rates for all wells were increased and decreased by the same amount each month. A 20% increase in total pumping was attained by all wells increasing production by 20%. In the current analysis, all wells can be increased or decreased independently.

As with the second analysis described above, two consecutive critically dry years were simulated in each run. The 1994 hydrology was used to simulate the hydrology of a critically dry year.

Following a series of iterative simulations, the following changes were made to the pumping distribution:

- Two new wells were installed at the western end of the basin. One well is approximately at the Plumpjack well location. The second well is located between the first well and District Well#2. We assumed these two wells are hydraulically similar to District Well#2. The maximum pumping rate for each of these wells in the buildout simulation is 112 gallons per minute.
- Pumping from the Resort at Squaw Creek wells 18-2 and 18-3 was increased to take advantage of the idle capacity of these wells.
- Pumping from the Mutual Well#1 was transferred to the Condo Well
- Additional pumping was added at the 4th Fairway well
- Pumping from District Well#3 was transferred to a portion of the idle capacity in Well 18-3
- Pumping from District Well#1 was increased relative to other District wells.
- Pumping from District Well#5 was reduced relative to other District wells.
- District Well#3 was been taken out of service.

The final pumping rates for the sustainable yield using both new and existing wells under the relaxed assumptions is shown in Table 5.

Table 5
Maximum Pumping Rates for Simulation with Both Existing and New Wells (Relaxed Assumptions)

Well	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
District#1	16.51	14.68	15.46	13.63	14.15	20.70	27.78	28.30	21.49	14.41	9.96	14.41	211.48
District#2	12.00	10.66	11.24	9.90	10.28	15.04	20.18	20.55	15.62	10.48	7.24	10.48	153.67
District#5	10.58	9.41	9.91	8.74	9.07	13.27	17.81	18.14	13.78	9.24	6.38	9.24	135.57
Mutual#1	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Mutual#2	4.20	3.50	3.85	3.50	5.25	7.70	10.85	10.50	8.75	5.60	3.15	3.85	70.70
18-1	1.70	0.00	0.00	0.00	1.70	1.70	1.70	1.70	1.70	1.70	1.70	1.70	15.30
18-2	17.04	4.57	4.81	4.24	14.61	18.35	20.55	20.71	18.59	13.84	15.00	16.39	168.70
18-3	35.25	18.12	19.19	16.52	17.06	37.05	45.58	46.65	37.12	17.59	42.26	47.59	379.98
Condo	5.10	4.25	4.68	4.25	6.37	9.35	13.18	12.75	10.63	6.80	3.83	4.68	85.87
4th Fairway	5.70	3.78	4.36	3.71	4.45	6.55	8.73	8.91	6.72	4.45	4.22	5.61	67.19
New1	9.19	8.17	8.61	7.58	7.88	11.52	15.46	15.75	11.96	8.02	5.54	8.02	117.70
New2	9.19	8.17	8.61	7.58	7.88	11.52	15.46	15.75	11.96	8.02	5.54	8.02	117.70
Totals	126.46	85.31	90.72	79.65	98.70	152.75	197.28	199.71	158.32	100.15	104.82	129.99	1523.86

As shown on Table 5, the sustainable yield of the basin using both new and existing wells is approximately 1,524 acre-feet annually. Of this, 261 acre-feet are assumed to be used by the Resort at Squaw Creek for golf course irrigation and snow making, leaving 1,263 acre-feet annually available for potable use by the District and the Mutual.

Section 7
CONCLUSIONS AND FUTURE ISSUES

The Squaw Valley Groundwater Model has been modified and updated based on additional data. The calibration statistics show that the model remains capable of accurately predicting groundwater conditions in Squaw Valley. For issues such as the construction dewatering, the current model is a better predictor of groundwater conditions than the previous model was.

As with all models, the calibration is not unique. Many parameter combinations are possible to produce a relatively good calibration. Of particular note, there may be many combinations of hydraulic conductivity of the western basin and the stream leakance that produce reasonable simulations. A better understanding of stream flows, hydraulic conductivity, and stream leakance would enhance the groundwater model.

Other outstanding issues that may improve the calibration include:

- A new set of initial conditions may improve the calibration for the first two years of simulation.
- A new method of simulating Squaw Creek may be necessary. The current model does not allow water to enter the basin from Squaw Creek when water levels fall more than 15 feet below the bottom of the creek bed. An optional approach has been investigated, and should be implemented.
- Additional study on the Olympic House Loading Dock site should be undertaken. The survey data should be investigated to make sure the reported groundwater elevations at the Loading Dock site are compatible with the data from the District wells. The impact from the Olympic Lady Reservoir should also be studied further
- Mr. Ken Loy of West Yost Associates is developing a new geologic interpretation of Squaw Valley. This interpretation may influence how the District and Mutual wells interact. It may be useful to incorporate this new geologic interpretation into the model.

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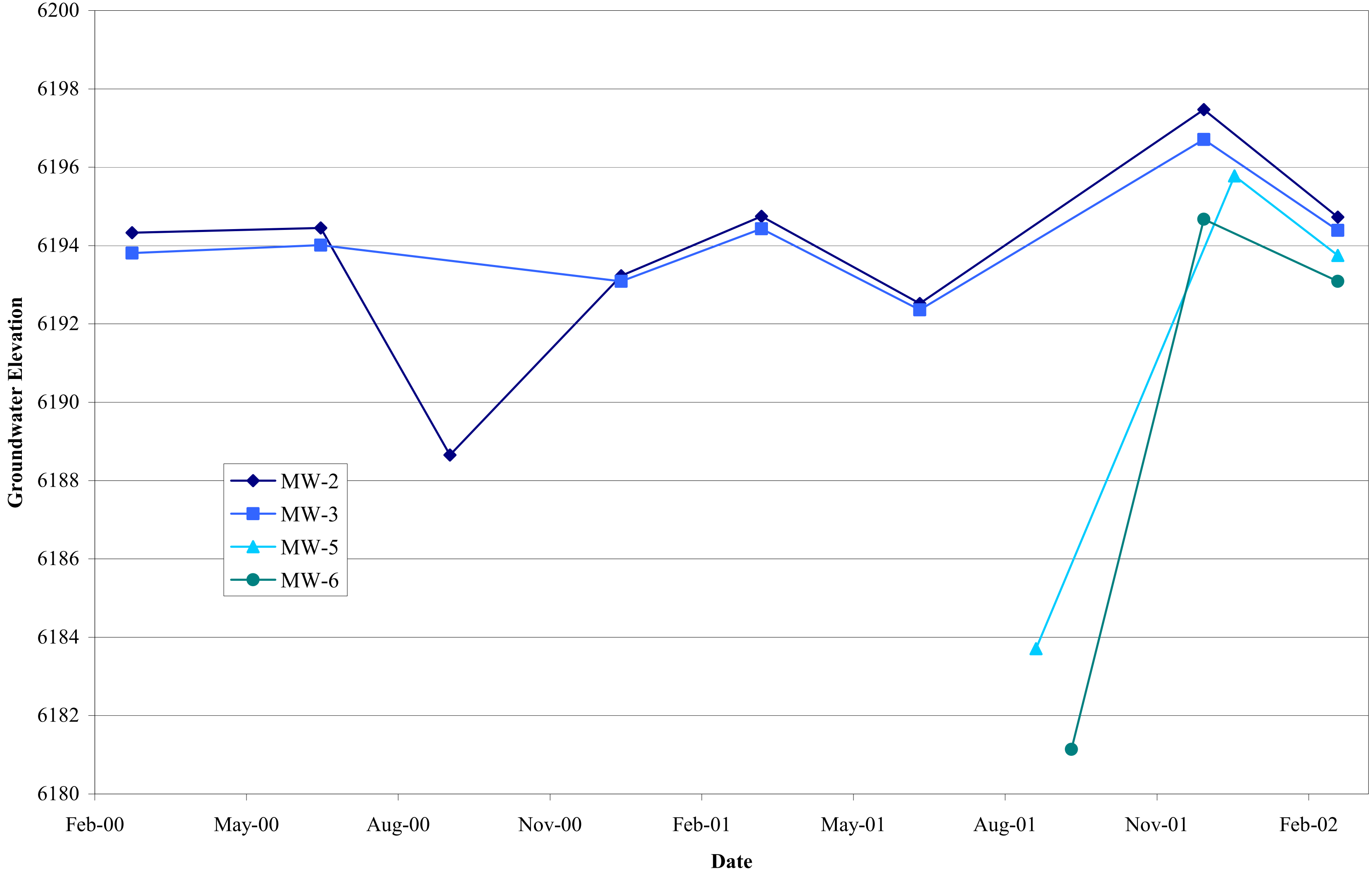
West Yost & Associates, 2001, *Draft Report, Squaw Valley Groundwater Development & Utilization Feasibility Study*

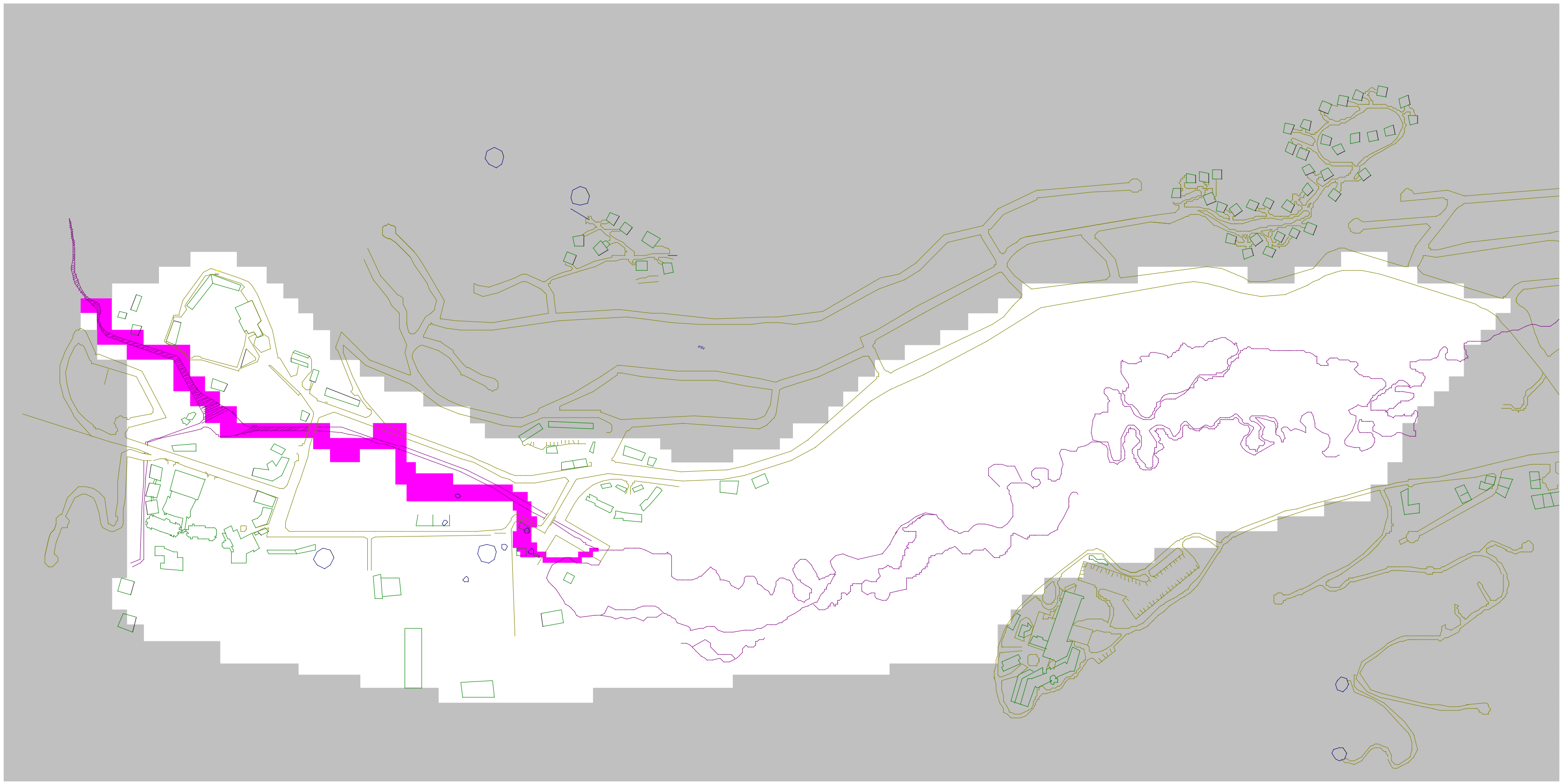
Williams, Derrik, 2002, *Construction Dewatering Study Results.*



derrik williams

Observed Water Levels, Olympic House Loading Dock Site

Figure 1



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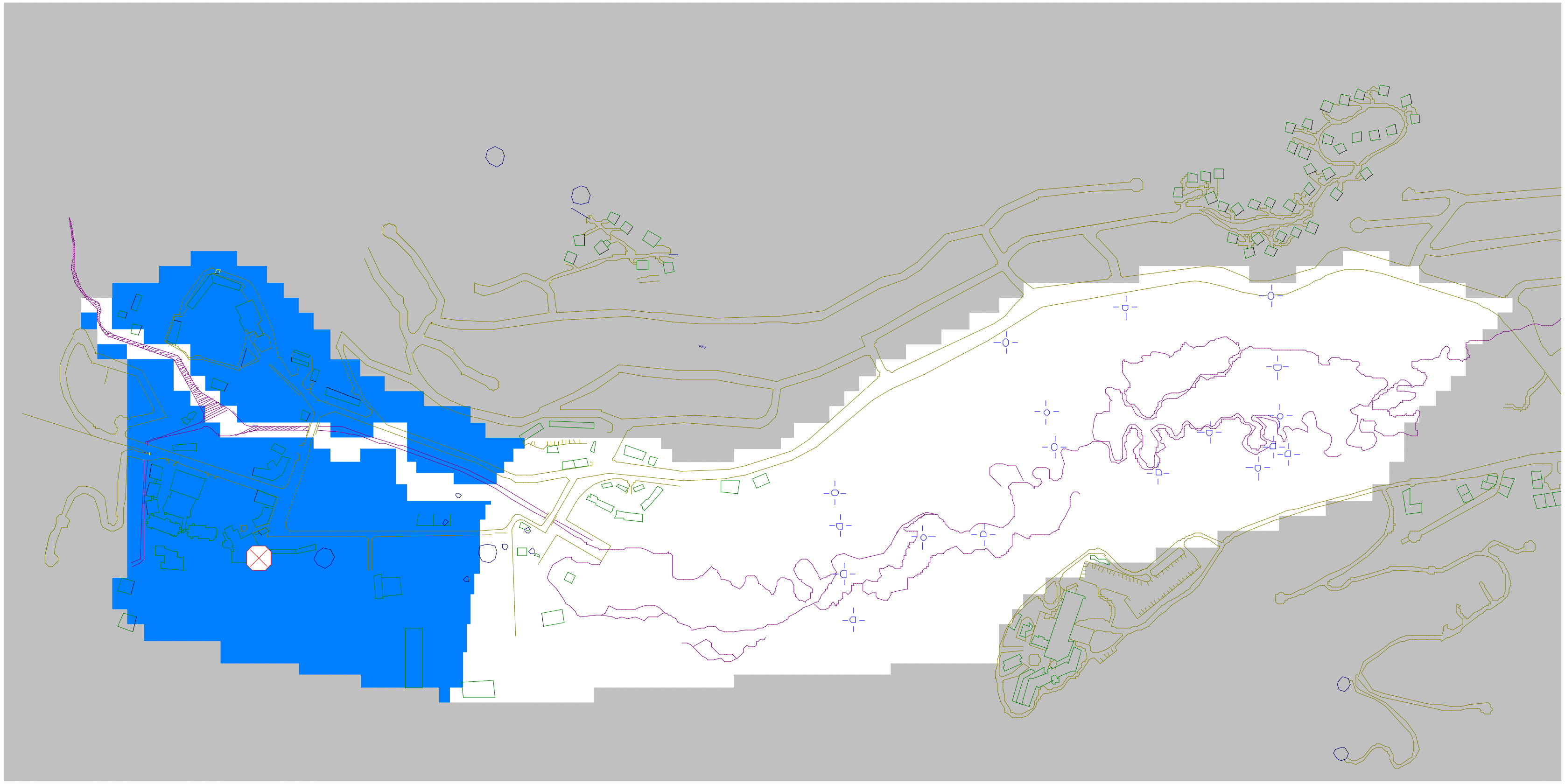
Location of Simulated Streambed Deposits

Figure 2



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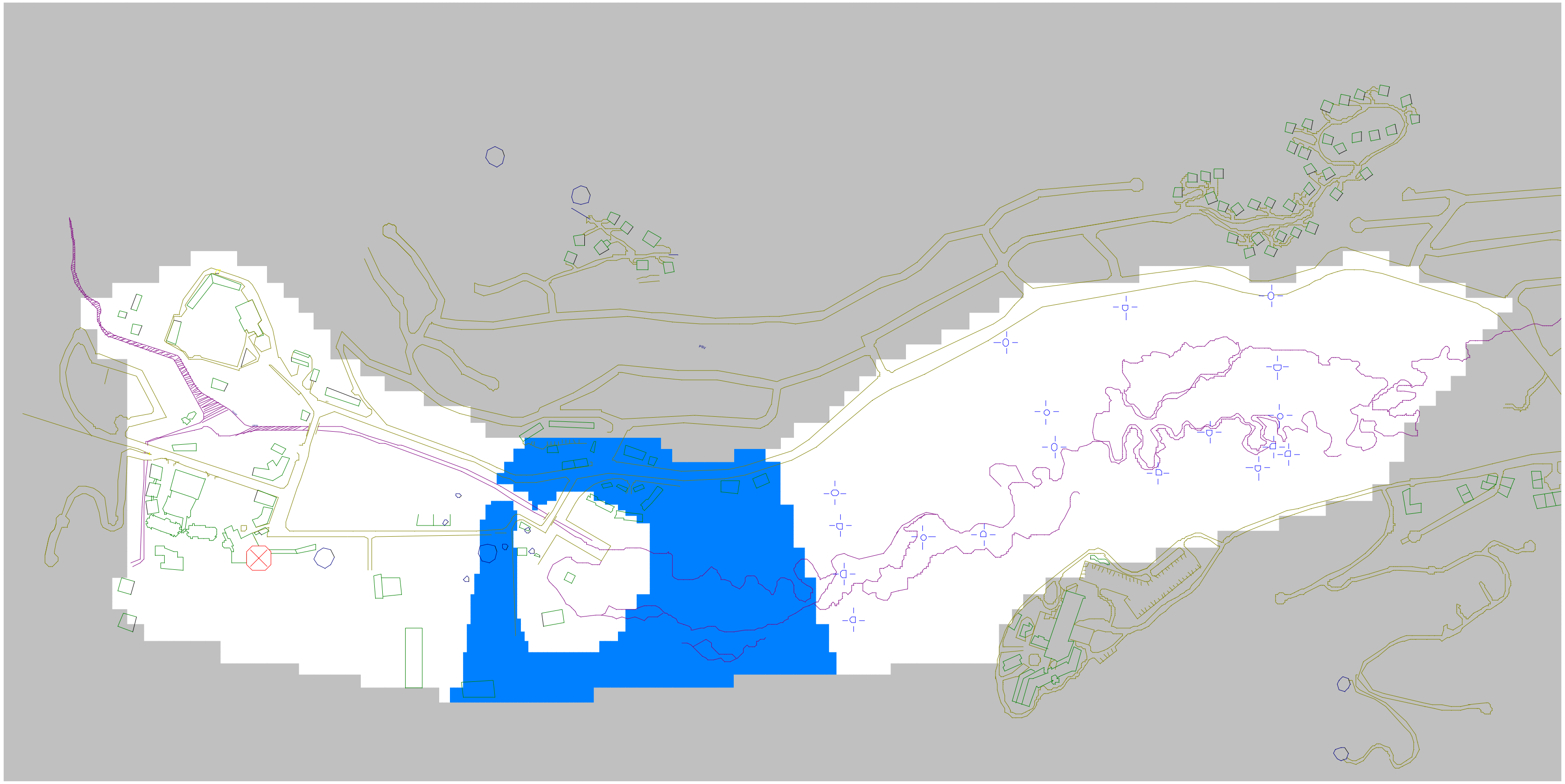
Hydraulic Conductivity Zone, Western End of Squaw Valley

Figure 3



Low Hydraulic Conductivity Zone, Middle of Squaw Valley

Figure
4

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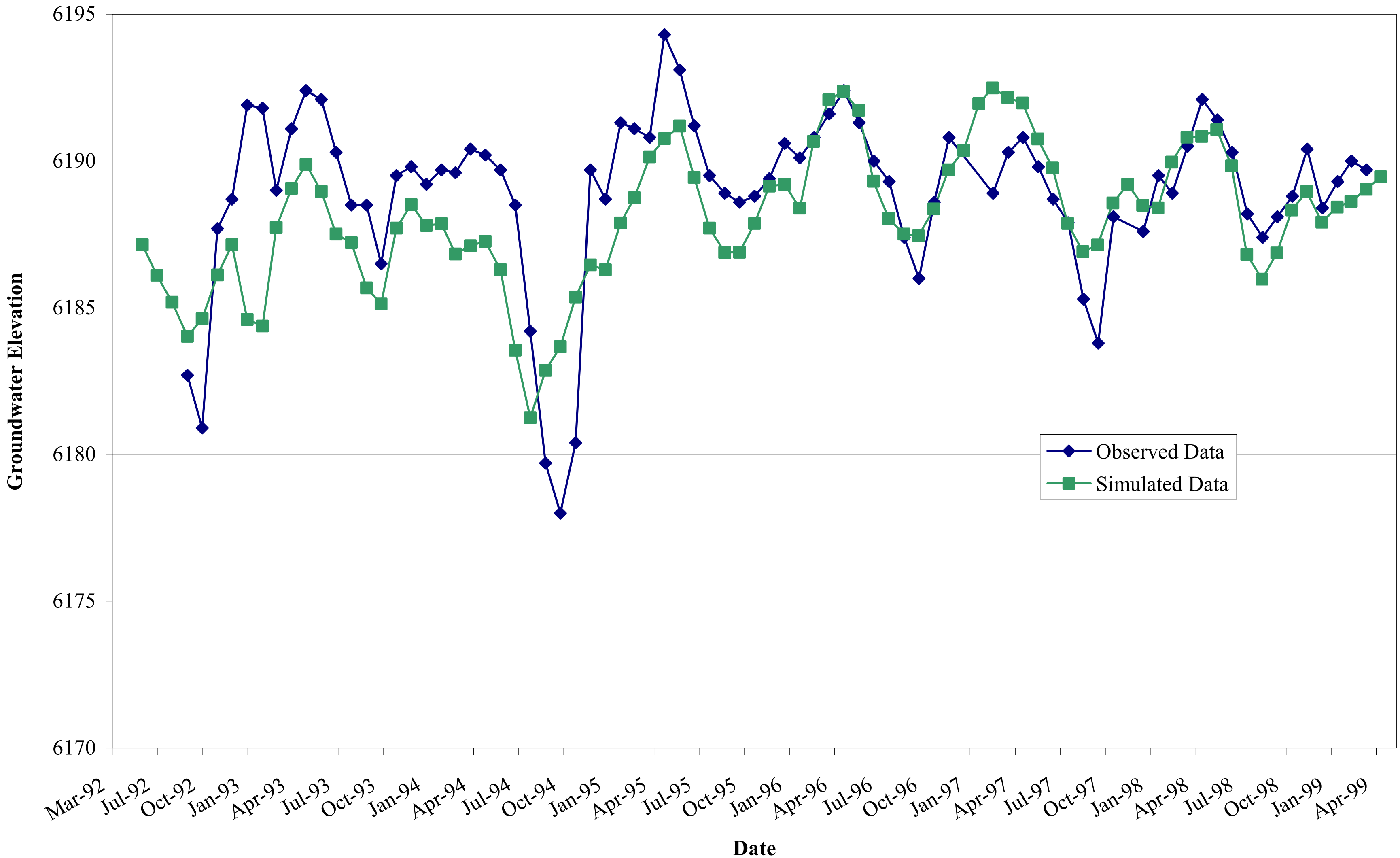

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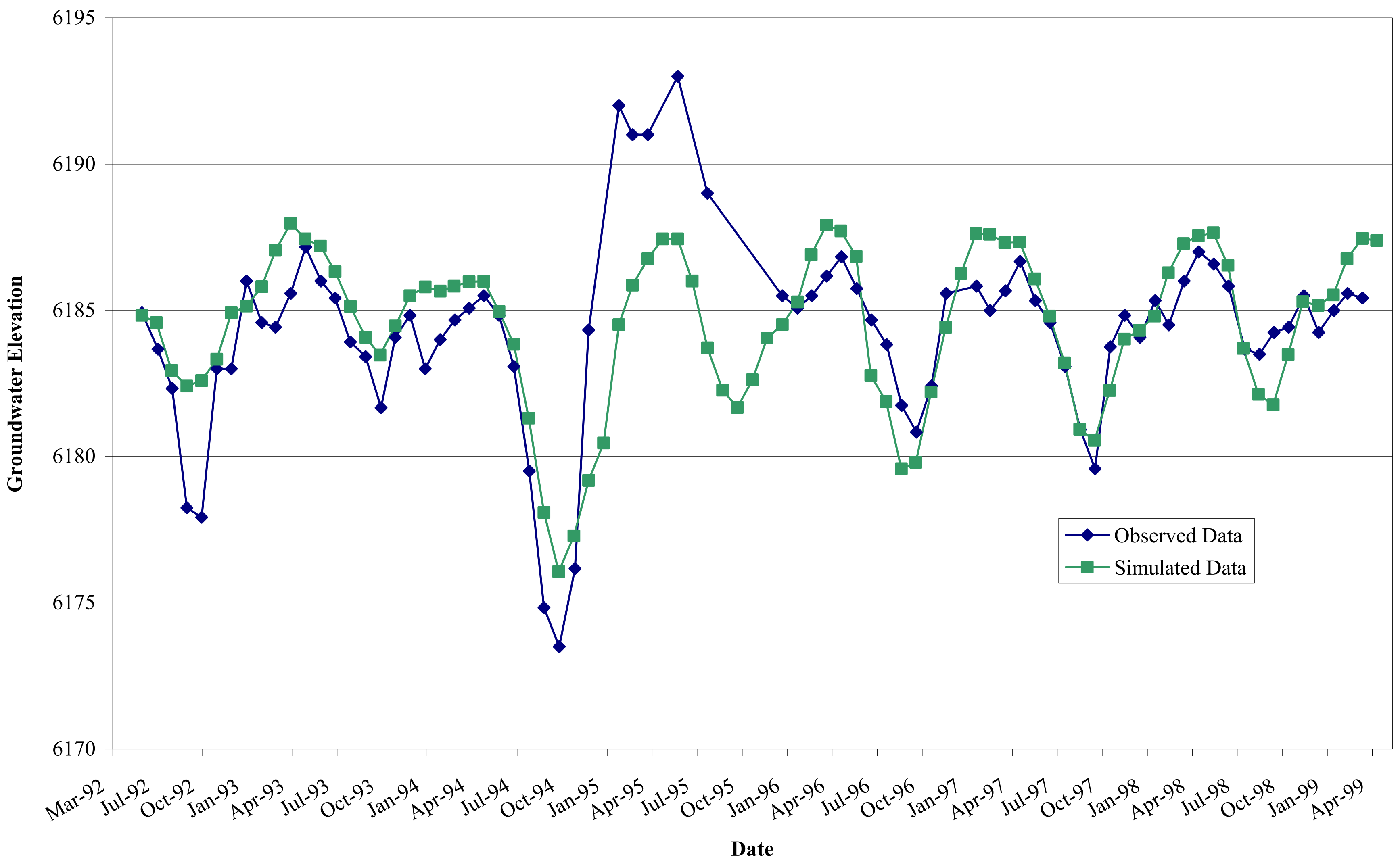
Observed and Simulated Water Levels, SVPSD Well#2

Figure
5



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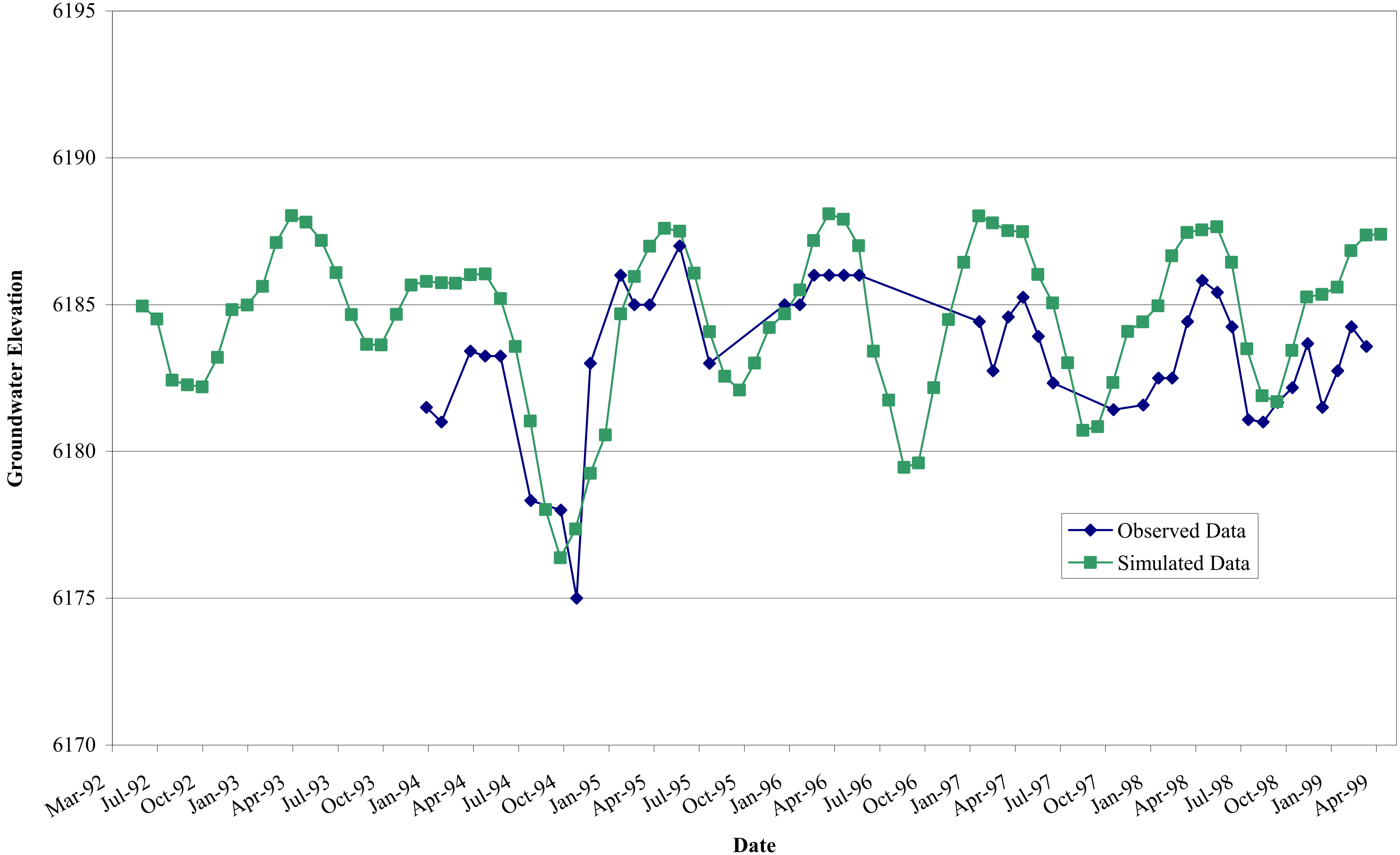
Observed and Simulated Water Levels, SVMWC Well#1

Figure 6



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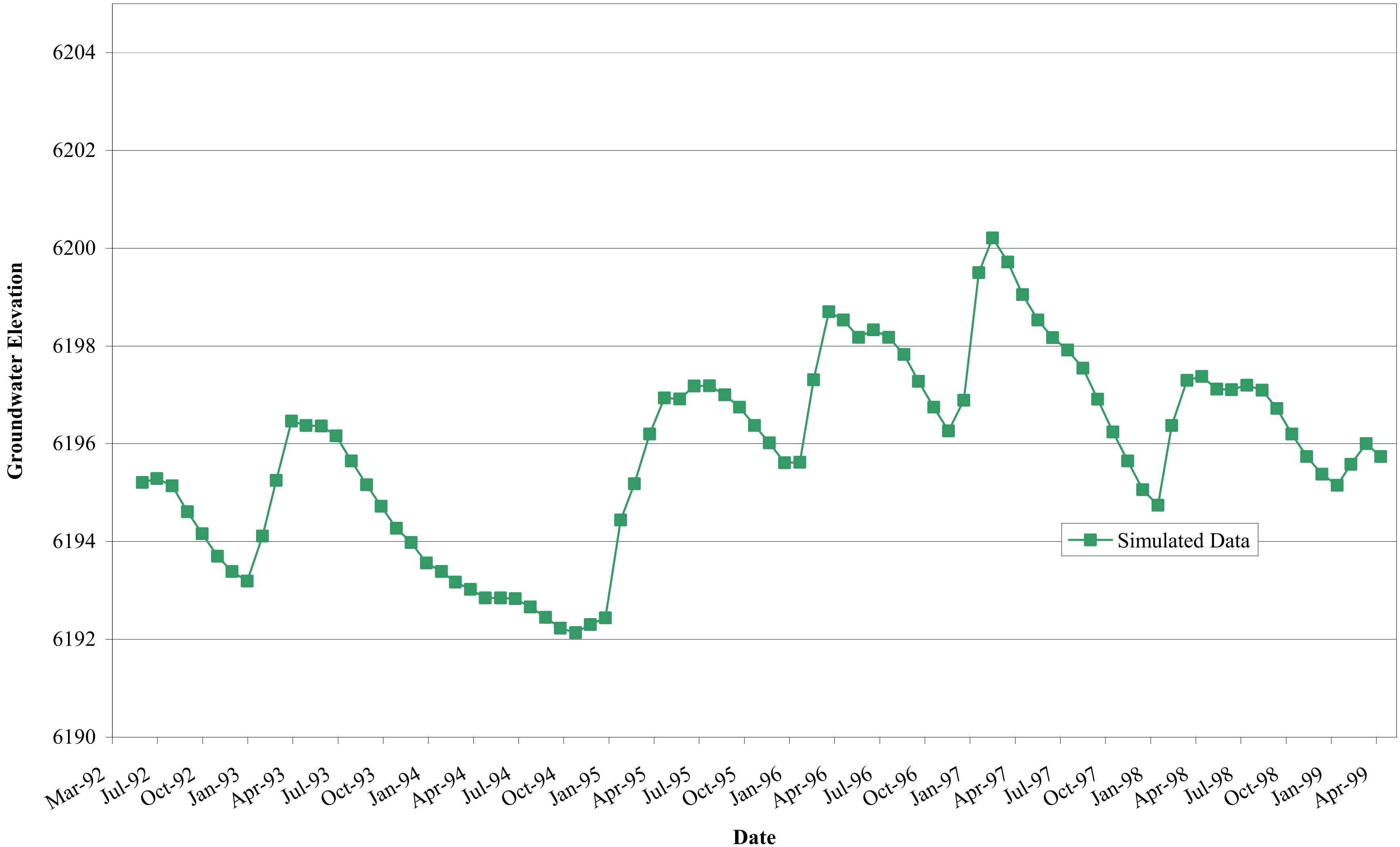
Observed and Simulated Water Levels, SVMWC Well#2

Figure 7



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Simulated Water Levels, Olympic House Loading Dock Site

Figure
8