Appendix E

Groundwater Flow Model

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E1. INTRODUCTION

This appendix to the Reche Spreading Grounds Recharge Feasibility Study Report (Feasibility Study report) and Groundwater Management Plan for the Ames Groundwater Basin, Pipes and Reche subbasins (GMMP) documents the construction and results of a water balance and numerical groundwater flow model used to assist in estimation of basin sustainable yield, characterization of groundwater flow conditions, and evaluation of recharge basin feasibility.

E1.1 Model Objectives

The objectives of the groundwater flow model are to 1) aid in characterization and evaluation of groundwater flow conditions (sources, sinks, flow rates and directions) in the Pipes and Reche groundwater subbasins and adjacent areas where BDVWA and others operate groundwater supply wells, 2) evaluate hydraulic impacts (water table mounding, groundwater flow paths) associated with future operation of the proposed Reche groundwater recharge spreading basin, and 3) evaluate sustainable yield of the Reche subbasin in support of the focused groundwater management plan and Amendment to the Water Agreement between BDVWA, Hi-Desert Water District (HDWD), and Mojave Water Agency (MWA).

E1.2 Model Approach and Scope

The numerical model simulates steady-state and transient groundwater flow in the Pipes and Reche subbasins. Groundwater recharge rates via subsurface inflow from Antelope Creek/Pipes Wash, Whalen's Wash, Ruby Mountain Wash, and distributed mountain-front recharge were estimated, along with rates of return flow from septic systems. Groundwater outflow via wells was defined based on metered pumping rates, and subsurface outflow from the Reche subbasin to the Giant Rock subbasin was simulated. After calibration, the model was used to predict water table mounding beneath the recharge basin, drawdown around nearby water supply wells, and flowpaths through the subbasins, across major geologic faults, from the recharge basin, and to the production wells.

The model was constructed using the United States Geologic Survey (USGS) numerical finite-difference codes MODFLOW and MODPATH. MODFLOW was selected for its usability, accuracy, efficiency and transportability. In particular the transportability of the public domain MODFLOW program and site model input files are advantageous for future site modeling. MODFLOW files have been provided to BDVWA and can be run without any proprietary software. Model construction and calibration was performed using the Groundwater Modeling System (GMS) v7.1 which pre-processes and post-processes MODFLOW and MODPATH files. Most of the input data were constructed and stored in GMS "GIS", "Map", "Scatter Point" and "2D Grid" modules. GMS software is not required to run the MODFLOW model. The MODFLOW

2000 files created by GMS also can be imported to other commercial MODFLOW software such as Visual MODFLOW or Groundwater Vistas with minor modification, or run using only executable MODFLOW and MODPATH codes.

Critical input parameters and controls on flow include water-budget components (inflows, outflows, and changes in storage), along with aquifer hydraulic properties (aquifer geometry and hydraulic properties of the alluvium and faults). The model inflows and outflows are based on an updated and refined water balance for the Pipes and Reche subbasins. The water balance was developed using available data and methodologies including those previously documented in the *Basin Conceptual Model and Assessment of Water Supply and Demand for the Ames Valley, Johnson Valley, and Means Valley Groundwater Basins* (Todd Engineers, 2007). This previous water balance was developed for the combined Ames Valley groundwater basin for the period 1990 through 2000. For this evaluation, the water balance period was extended through water year 2008-2009 (period ending September 30, 2009). Estimates for some key water balance components (including subsurface inflow and septic return flow) were refined. Each element of the water balance was evaluated independently, including inflows (e.g., recharge from rainfall resulting in subsurface inflow from Antelope Creek/Pipes Wash, Whalen's wash, and Ruby Mountain Wash, and septic return flows), and outflows (e.g., subsurface outflow, groundwater pumping). As part of the development of the GWMP, basin perennial yield was calculated and used in support of the Water Agreement Amendment.

The groundwater model domain and boundaries are shown on Figure E1. The active model area includes all of the Pipes and Reche subbasins, and a portion of the Giant Rock subbasin. The model domain also includes the proposed Reche spreading grounds recharge site and nearby BDVWA, HDWD, and San Bernardino County Service Area No. 70 W-1 (CSA No. 70 W-1) water supply wells.

Hydraulic properties including permeability of basin alluvium and geologic faults, aquifer thickness, and storage coefficients were simulated appropriately across the model area. Appropriate boundary conditions were selected based on the water balance and observed groundwater elevations.

The model was calibrated to observed historical water levels between 1994 and 2009. The calibration process includes trial and error adjustment of input parameters and auto-calibration using the Parameter Estimation (PEST) computer code. Once calibrated, flow paths and travel times between the recharge site and downgradient areas, including production wells, were simulated using anticipated recharge and pumping rates and schedules. Forward flowpaths were simulated to evaluate groundwater flow directions and rates from recharge site to the production wells and outflow boundaries, and reverse flowpaths were simulated to identify capture zones of the existing production wells.

E2. MODEL INPUT AND CALIBRATION

This section documents the approach and input parameters used to calibrate the groundwater flow model. Existing data were used to formulate the initial model input parameters. As described below, initial estimates of some input parameter values were modified during model calibration. Some input parameters, including extraction well pumping rates and water use/septic return flows, were defined on the basis of site measurements or estimates and were not varied for most of the model simulations. Other parameters, including aquifer hydraulic conductivity and boundary conditions, were adjusted within defined ranges to achieve model calibration.

E2.1 Calibration Process and Criteria

Model calibration was accomplished by defining and achieving quantitative and semi-quantitative calibration goals or targets. Calibration was assessed through evaluation of residuals, or the difference between observed and simulated groundwater elevations (heads), hydraulic gradient directions, and volumetric flow rates. For the steady-state site models constructed to simulate 1994 (wet) and 2007 (dry) conditions, head residuals were calculated for wells located throughout the Pipes and Reche subbasins. For the transient calibration period of 1994 to 2009, residuals were calculated in both space and time. Error residuals at each point were averaged in a variety of ways and statistical parameters including mean error and root mean squared error were calculated.

Criteria as were defined to evaluate the quality of model calibration. The two criteria used were the ASTM-recommended Root Mean Square (RMS) head residual error of less than ten percent of the model area groundwater elevation range and a mean error residual of less than five percent. The calibration criteria for the Pipes-Reche model are an RMS of 100 feet and Mean Error of 50 feet. This corresponds to a water table elevation range of about 3,600 feet above mean sea level (feet msl) where Pipes Wash enters Pipes Subbasin and 2,600 feet msl in the western portion of Giant Rock Subbasin.

E2.1.1. Historical Calibration Periods

Both transient and steady-state models were constructed for calibration. The transient model included 180 monthly stress periods between October 1994 and September 2009. Steady-state models were also constructed and calibrated to reflect “average” groundwater flow conditions. To simulate the variability in hydrologic conditions at the site, steady-state models were constructed to simulate two different historical periods, Water Year (WY) 1994 and WY 2007. WY 1994 represents peak “wet” or “high groundwater” conditions, based on several years of above-average rainfall and associated recharge rates preceding this period, high pumping rates, and peak historical groundwater elevation conditions, based on water level hydrographs. WY 2007 represents “dry” or “low groundwater” conditions after several below-average rainfall years between late 2005 and 2007. Models were calibrated to both 1994 and 2007 conditions by adjusting hydraulic conductivities and boundary conditions. The transient calibration simulated changes in groundwater elevations over time, and presentation of the calibration results in this Appendix focuses on this transient simulation.

E2.1.2. Water Level Calibration Data

Available water level data were reviewed to define a set of wells used for model calibration. Figure E2 shows the location of wells used to assess model calibration. The observed water level calibration data set includes most of the existing active production wells and a few dedicated monitoring wells. As shown in the figure, the spatial distribution calibration wells is favorable considering calibration wells are located both in the upgradient and downgradient portions of the Pipes and Reche subbasins.

E2.1.3. Calibration Approach

The model was constructed and calibrated using both trial-and-error and PEST auto-calibration methods. Initial model construction and calibration runs were based on estimated input parameter values. Boundary conditions were developed based on observed groundwater elevations and estimated fluxes through Pipes, Whalen’s, and Ruby Mountain washes. Minor modifications to the flux boundary locations and rates were made during calibration. Estimates of groundwater recharge from septic return flow were developed from water use data over time and a consumption factor for each land parcel. Estimated recharge from septic return was not adjusted during calibration. Initial estimates of hydraulic conductivity and aquifer storage coefficients were developed based initially on aquifer pumping test results and subsequently modified based on initial calibration and PEST results. Production well pumping rates were not adjusted during calibration runs.

Several parameter estimation simulations were performed using PEST simulating steady-state 1994 and 2009 conditions and a transient period of 1994 through 2009. The parameters selected for inversion were hydraulic conductivity and specific storage. PEST simulations inverted all polygons for hydraulic conductivity and specific storage simultaneously, with the exception of a polygon between the recharge site and Production well HDWD No. 24, which was assigned a fixed hydraulic conductivity value based on the constant-rate pumping test performed in October 2010. Minor hand adjustments of PEST-calculated conductivities were made for the final calibration. Results of the PEST simulations and final calibration are discussed in Section E3.0.

E2.2 Model Domain and Discretization

The MODFLOW model simulates groundwater flow in a defined area and solves the governing equations controlling groundwater flow using the finite-difference method. For this numerical method, a rectangular grid of model cells is constructed, and hydraulic head is calculated at each grid cell.

E2.2.1. Model Area and Grid

The Pipes/Reche subbasin active model domain includes the area bounded by:

- The valley floor at the base of the mountain front to the west;
- An east-west trending arc to the north coinciding with a broad bedrock high and thin saturated aquifer thickness in the northern portion of the Pipes and Reche subbasins;
- An arc east of the Homestead Valley Fault within the Giant Rock subbasin to the east
- A southwest-northeast trending arc beneath the Mesa area (southeast of Pipes Wash) where the alluvial aquifer becomes unsaturated.

A uniform row and column grid spacing of 100 feet was used. The model comprises 430 rows by 387 columns. A single MODFLOW layer represents the alluvial aquifer.

E2.2.2. Model Depth

The model grid was constructed using the MODFLOW Layer Property Flow (LPF) Package and a “true layer” approach, with defined aquifer bottom elevations. Figure E3 illustrates the geometry of the base of the alluvial aquifer. The bedrock contact surface dips to the east from elevations of around 3,400 feet msl at the edge of the valley at the base of the mountains to elevations of around 2,600 feet in the thickest portions of the alluvial basin. A shallow bedrock ridge occurs beneath the Mesa area with alluvium-bedrock contact elevations of around 3,500 feet msl in the southwestern portion of the model to around 3,000 feet msl in the southeastern portion. A broad shallow bedrock ridge also occurs along the northern model boundary in the northern portion of the Pipes and Reche subbasins. The modeled bedrock elevation in the western portion of the Giant Rock Subbasin is around 2,500 feet msl. A bedrock surface discontinuity of 200 feet was simulated across the Homestead Valley Fault separating the Reche and Giant Rock subbasins.

E2.3 Boundary Conditions

Figure E4 shows the model boundary condition locations and types. The Pipes/Reche groundwater model includes the following boundary conditions:

- Lateral time-varying specified fluxes via arcs across Pipes Wash, Whalen's Wash, Ruby Mountain Wash, and along the valley-mountain front boundary to the west;
- Lateral specified flux (no-flow) boundaries representing 1) the broad bedrock ridge and thin saturated aquifer thickness to the north and 2) shallow bedrock ridge beneath the Mesa area (southeast of Pipes Wash) where the alluvial aquifer becomes unsaturated to the south;
- A lateral general head boundary east of the Homestead Valley Fault within the Giant Rock groundwater basin to the east;
- Time varying specified flux boundaries via the top of the model representing aerial recharge from septic return flow;
- A specified flux (no flow) boundary at the base of the model.

For the specified flux boundaries (subsurface inflow and return flow), monthly rates were estimated and used in the transient flow model. The following sections describe quantification of the boundary flux rates and heads used in the mathematical model.

E2.3.1. Western Specified Flux Boundaries

The principal source of natural groundwater recharge to the Pipes and Reche subbasins is the subsurface inflow of groundwater through the alluvium within Pipes Wash, Whalen's Wash, and Ruby Mountain Wash. This groundwater inflow originates from runoff of rainfall in the San Bernardino Mountains and recharge to the alluvium in the wash channel valleys east of the Pipes Subbasin. Runoff from rainfall infiltrates through the vadose zone to the water table prior to entering Pipes Subbasin as subsurface inflow mainly through the three major drainages entering the valley. Subsurface inflow rates from bedrock along the rest of the mountain-front are unknown, but the amount is assumed to represent a small portion of subsurface inflow, as discussed below.

Direct recharge from rainfall on the basin floor is assumed to be negligible given the small amounts of rainfall on the valley floor, deep water table, and high evapotranspiration rates. Intermittent flash flooding through Pipes Wash, Whalen's Wash, Ruby Mountain Wash and other drainage pathways occasionally brings water into and through the valley floor, but for the purposes of this analysis, the net amount of stormwater recharging groundwater is assumed to be negligible.

Figure 3 in the Feasibility Study report shows the contributing watershed area and annual rainfall isohyets for the model flux boundaries. The contributing watershed area is divided into three major drainages. Antelope Creek (tributary to Pipes Wash) has the largest contributing catchment area to the basin, representing over 60 percent of the overall contributing watershed area. Whalen's Wash and Ruby Mountain Wash to the north have smaller catchment sizes and lower average annual rainfall rates.

Based on a focused study of the watershed area and groundwater flow rates through Whalen's Wash and Antelope Creek/Pipes Wash, average natural subsurface inflow to the Pipes Subbasin is estimated at 2 percent of rainfall in the contributing watershed area. This average rainfall-recharge ratio is the basis for the boundary condition flux rates developed for the model.

Based on a 20-year study period from water year (WY) 1989-1990 to WY 2008-2009, the average annual recharge from rainfall for the Pipes Subbasin is 668 acre-feet per year (AFY). The Antelope Creek Catchment is the largest contributor of recharge (472 AFY), followed by Whalen's Wash (127 AFY), and Ruby Mountain Wash (69 AFY).

In order to vary the amount of natural subsurface inflow to the model boundary over time, precipitation over time across the contributing watersheds was calculated based on data from the rainfall gage at Big Bear and the average annual precipitation isohyetal map (Figure 3 in the Feasibility Study report). The Big Bear rainfall gage has been active since July 1960. Average annual precipitation for Water Year (WY) 1960-61 through WY 2008-2009 for the Big Bear gage is 21.60 inches. To estimate monthly rainfall in which precipitation at the Big Bear gage was not reported, the average relative monthly precipitation between the Big Bear gage and Lake Arrowhead gage was applied to Lake Arrowhead gage data for that month. Note that average annual rainfall in the contributing watershed areas of the three major drainages to the Pipes Subbasin is much lower than rainfall reported at the Big Bear gage, ranging from 8.54 inches for Antelope Valley (Pipes Wash), 6.35 inches for Whalen's Wash, and 5.39 inches for Ruby Mountain Wash.

To estimate annual recharge from rainfall over varying climatic conditions, the ratio of annual rainfall at the Big Bear gage to the long-term average annual rainfall at the Big Bear gage was applied to the average annual rainfall for the contributing watershed (based on spatial analysis of the isohyetal map) multiplied by 2 percent.

Additionally, for any given period, the percentage of rainfall that represents runoff is expected to be positively related to the rainfall amount (i.e. less than 2 percent runoff is expected when rainfall is below normal, while greater than 2 percent runoff is expected when rainfall is above average). To account for this variability, a variable runoff factor ranging from 0.5 percent (applied to years when annual rainfall at the Big Bear gage is less than 10 inches) up to 3.0 percent (for years when annual rainfall is 30 inches or greater) was applied to rainfall in the contributing catchment areas. The weighted-average runoff factor of 2 percent was maintained over study period.

To account for the vadose and saturated zone travel time and time lag for recharge entering the Pipes Subbasin as subsurface inflow, monthly rainfall reported at the Big Bear rainfall gage was compared with groundwater elevations in Well 1N/5E-2N1, located along Pipes Wash near the intersection of Pipes Wash and Highway 247 (Figure E2). Figure E5 shows that groundwater levels in Well 1N/5E-2N1 respond gradually to significant rainfall events in the San Bernardino Mountains and continue to do so for up to 2 years before receding. This process reflects the capacity of the alluvial materials to detain runoff generated in the contributing watersheds of the major drainages upgradient of the modeled area. For the model, a retention time was developed to “lag” and re-distribute the subsurface inflow over time. To simulate this process in the MODFLOW model, the effective monthly subsurface inflow rate was calculated by lagging rainfall amounts by one year and applying a detention coefficient of 0.90. A lag of one year combined with a detention coefficient of 0.90 was found to best simulate the effective subsurface inflow rate over the model period. Figure E5 shows the effective subsurface inflow rates for Antelope Creek/Pipes Wash using the method described above compared to groundwater levels in Well 1N/5E-2N1. Table E1 shows the effective annual subsurface inflow rates for all three of the major drainages in the model (Flux Arcs 2, 5, and 9).

A small portion of the total estimated subsurface inflow for each period was redistributed along the mountain-front arc segments between the three washes (see Figure E6 for final specified flux arc boundary locations). Again, the overall total subsurface inflow flow was maintained at 2 percent of rainfall. During calibration, the amount re-allocated to mountain-front recharge was varied, and ultimately 10 percent was used in the final calibrated model.

The annual flux rates used for each specific flux boundary arc are tabulated in Table E1. The average total model influx through Pipes Wash, Whalen’s Wash, Ruby Mountain Wash, and mountain front arcs for the simulated period from WY 1994-95 to WY 2004-05 was 796 AFY, of which 703 AFY represents the influx through the main washes, 61 AFY represents the influx through mountain flux arcs, and 31 AFY represents return flows from parcels west of the flux arc boundaries (see Section E2.3.4. for additional discussion on return flows). It is noted that the estimated natural inflow (764 AFY) for the transient model period (WY1994-95 to WY 2004-05) is slightly higher than the average annual recharge estimated for the 20-year study period (WY 1989-1990 to WY 2008-2009) in the basin conceptual model report (Todd Engineers, 2007). This is due primarily to the modeled detention/lag of rainfall runoff generated during the winter storms of 1992/1993.

E2.3.2. Northern and Southern No-Flow Boundaries

Portions of the alluvial aquifer beneath the Mesa separating the Pipes/Reche subbasins from the Copper Mountain Subbasin to the south and in the northern portion of the Pipes/Reche subbasins are thinly saturated to unsaturated (the water table occurs below the alluvium/bedrock contact). The location of these unsaturated areas were determined based on comparisons of the water table and bedrock elevation surfaces and defined in the MODFLOW model as no-flow boundaries (Figure E4).

E2.3.3. Eastern General Head Boundary

Figure E4 shows the location of a constant-head boundary arc used along the eastern model boundary in Giant Rock Subbasin. A constant head of 2,600 feet above mean sea level (feet msl) was defined along the arc based on groundwater elevations measured in the subbasin. The location of the boundary head arc and elevation value was based on a regional groundwater elevation map (Figure 4 in the Feasibility Study report).

E2.3.4. Return Flow Recharge Boundary

In addition to natural runoff from rainfall, inflow to the groundwater basin occurs via return flow from septic tanks. Return flows in the Pipes and Reche subbasins were simulated as a time-varying recharge boundary at the top of the model using the MODFLOW recharge package. Water use over time for each BDVWA water customer and estimated net septic return flow rates were analyzed to accurately simulate the rate and distribution of aerial recharge.

Monthly water use rates for each assessor parcel number for the period 1995 – 2009 was obtained from BDVWA. Figure E7 shows the locations of the BDVWA water customer parcels and recharge areas. Monthly water use rates were converted to recharge rates using a consumptive use factor of 20 percent, or a return flow rate of 80 percent of water use. The relatively high consumptive use factor was selected, since water use in the area is predominantly indoor, and because water use as metered at each customer site is considered under-reported by 10 to 20 percent by BDVWA. Historic water use of HDWD customers in the Mesa area was not available for this study but is relatively small compared to natural recharge estimates and water use of BDVWA customers in the study area.

To account for travel time from the near-surface septic systems to groundwater, the vadose zone flow model CHEMFLO™-2000 (USEPA, 2003) was used. Input parameters for the vadose zone model include soil hydraulic properties, initial soil water conditions, and assignment of appropriate boundary conditions at the top and bottom of the soil profile. A vertical hydraulic conductivity of 3 centimeters per hour (cm/hr) (or about 2.4 feet per day) was selected for use in the model. This was initially based on an average horizontal hydraulic conductivity of about 30 cm/hr (or about 24 feet per day) for existing wells in the Pipes and Reche subbasins and an assumed 10-to-1 ratio for horizontal-to-vertical hydraulic conductivity. The estimate is on the lower end of the range of vertical hydraulic conductivities from soil cores collected from the recently installed monitoring well (MW1) in Pipes Wash. Other required soil hydraulic properties for the model (vanGenuchten coefficients) are provided in CHEMFLO for various soils. These hydraulic properties were estimated based on interpolation between sandy loam and loam soils with vertical hydraulic conductivity of 4.0 and 1.0 cm/hr, respectively. The hydraulic boundary condition at the point of applied water was simulated by applying a soil matric potential of zero at the top of the soil profile (i.e., saturated conditions). This approach assumes that the amount of indoor water use by parcel is positively correlated with the number of septic tanks required to treat the water (i.e., as such, vadose zone travel times are considered similar for smaller and larger water use parcels). A uniform volumetric water content of 11 percent (matric potential of -300 mm) was assigned to the soil profile to simulate initial conditions, and a free drainage boundary condition was applied to the bottom of the soil profile.

Results of the vadose zone model were applied to the average depth to water beneath all return flow parcels in the model area (233 feet below ground surface) to estimate the average travel time of septic return flows through the vadose zone. Results of the model suggest that return flows require an average of about one year to travel through the vadose zone. Accordingly, return flow rates for each parcel were lagged by one year prior to introducing recharge to the MODFLOW model. Field and laboratory confirmation of vadose zone hydraulic properties are needed to further refine estimated vadose zone travel times. However, for the purposes of the groundwater model, the one-year travel time is considered reasonable.

Time-varying recharge rates were used during the transient model simulations. For the steady-state simulations representing 1994 and 2007 conditions, representative return flow rates corresponding to the average rate over the three-year period prior to and during the calibration period were used. Figure E8 shows the average return flow recharge rates by parcel over time.

E2.4 Groundwater Production

Groundwater pumping from all existing BDVWA, HDWD, and CSA No. 70 W-1 production wells were simulated using the MODFLOW Well Package. Production well locations are shown on Figure E7. Time-varying pumping rates were used during the transient model simulations. For the steady-state simulations representing 1994 and 2007 conditions, representative flow rates corresponding to the average rate over the three years period prior to and during the calibration period were used. Pumping rates are tabulated and plotted in Table E2 and Figure E9.

E2.5 Aquifer Hydraulic Properties

The model grid and aquifer hydraulic properties were simulated using the LPF Package. Heterogeneous hydraulic conductivities were assigned to polygons representing wash and non-wash areas and fault zone hydraulic barriers. Based on evaluation of the aquifer pumping test results and geologic mapping of alluvium, the aquifer permeability distribution appears to be controlled by the extent of relatively high permeability alluvium in the wash areas, and by the faults crossing the study area, which represent partial barriers to groundwater flow. Therefore, hydraulic conductivity polygons were constructed to represent the more permeable areas along Pipes Wash, Whalen's Wash, and Ruby Mountain Wash, the areas between the washes, and the fault zones. Figure E10 shows the polygon distribution.

Initial values of hydraulic conductivity were developed based on the mapped distribution of geologic materials and aquifer pumping testing data and were adjusted during model calibration. Analysis of existing pumping test data was performed in the 2007 study of the Ames Basin. In addition, a constant-rate pumping test of Well HDWD 24 was performed in October 2010, and the results of this test were applied in the vicinity of Well HDWD 24.

During model calibration, trial-and–error and PEST simulations were performed and permeabilities for the alluvium and faults were adjusted relative to the initial estimated values. The following Sections discuss the initial and final simulated properties of the alluvium and fault barriers.

E2.5.1. Alluvium Hydraulic Properties

Forty-four permeability polygons were ultimately used to simulate the alluvium and faults. The polygons were constructed on the basis of the mapped distribution of the wash and non-wash areas, with the wash areas assumed to have the highest permeabilities. During calibration, additional polygons were constructed to provide detail and flexibility to increase calibration quality. For the initial model setup and runs, relatively higher permeabilities of 20 to 100 feet per day were assigned to wash areas and lower permeabilities of 10 feet per day were assigned to areas between the washes.

Based on the results of the pumping test performed on HDWD 24, the hydraulic conductivity polygon representing the eastern portion of Pipes Wash between the proposed recharge site and HDWD 24 was assigned a fixed hydraulic conductivity of 150 feet per day (ft/day). Hydraulic conductivities for all other polygons were optimized using PEST. Figure E10 shows the final hydraulic conductivities used in the calibrated model. In general, the final hydraulic conductivity values used in the model are consistent with the site conceptual model with higher permeability in the washes and lower permeability in non-wash (more clay-rich) areas. The PEST results are also consistent with the range of hydraulic conductivities estimated from reported production well specific capacities. The highest permeabilities were simulated in the wash channels. Lower hydraulic conductivities were calculated for non-wash areas. The simulated hydraulic conductivity values are consistent with the site conceptual model and available aquifer property data.

E2.5.2. Fault Barrier Hydraulic Properties

Narrow hydraulic conductivity polygons were constructed to simulate the fault barriers including the Johnson Valley Fault and Pipes Barrier, separating the Pipes and Reche subbasins, and the Homestead Valley Fault, separating the Reche and Giant Rock subbasins (Figure E10). Hydraulic conductivity zones were used to represent the fault barriers (rather than the MODFLOW Horizontal Flow Barrier Package), because the polygons better represented the multiple en-echelon fault splays associated with each fault zone rather than a single fault alignment. Horizontal hydraulic conductivities for the fault polygons calculated by PEST ranged from 0.0012 to 100 feet/day. Higher permeabilities were estimated for the Johnson Valley Fault segment crossing Pipes Wash than for the other fault segments. These results are consistent with the site conceptual model, which indicates significant groundwater flow occurs through the Pipes Wash area, while more resistance to flow is created by the Pipes Barrier, just west of the proposed recharge site.

E2.5.3. Aquifer Storage Properties

For the transient flow simulations, specific storage was defined to account for release of water from aquifer storage. Specific storage is equivalent to the aquifer storage coefficient divided by the aquifer thickness. For the preliminary simulations, a uniform specific storage of 0.001 ft^{-1} was used. During the transient PEST simulation, an optimal specific storage of 0.0021 $foot^{-1}$ was estimated. Although the aquifer saturated thickness varies, on average it is around 150 feet, which yields a storage coefficient of approximately 30 percent.

E3. MODEL RESULTS

This section presents the model results, including calibration quality, simulated groundwater elevations, volumetric mass budgets for the model inflow and outflow components, and flowpath results. The results presented in this Section focuses on the 1994 through 2009 transient calibration.

The final model was developed after calibration runs based on the initial results and modified based on observed model response to input parameter changes. After construction and specification of model depth, boundaries, pumping well flow rates, and septic return flows, the PEST program was used to adjust net hydraulic conductivities and specific storage. For the 1994-2009 auto-calibration run, hydraulic conductivities and specific storage values were optimized with good results. Final manual adjustments were then made to some of the parameter values, including certain hydraulic conductivity zones.

Over the course of model development, numerous modifications of the values and distribution of input parameters were made in attempts to improve model calibration. Due to uncertainties in the actual distribution of hydraulic conductivity, and the inherent limitations of groundwater model approximations, perfect calibration in space and time is difficult or impossible to achieve. However, the Pipes/Reche MODFLOW model was reasonably well calibrated with respect to observed and simulated groundwater elevations in both space and time.

E3.1 Calibration Results

To assess model accuracy, simulated heads were compared with observed heads. Model calibration also focused on simulating flow through the groundwater subbasins in accordance with the basin conceptual model. The final calibrated models simulate flow conditions which are consistent with the basin conceptual model.

Charts E1 through E15 present observed versus simulated groundwater elevations between 1994 and 2009. As illustrated on the charts, the simulated and actual groundwater elevations and fluctuations over time are well-correlated. In particular, the overall water-level declines observed in many of the wells between 1994 and 2009 accurately simulated.

Observed and simulated heads at each calibration point were compared and calibration was assessed quantitatively through head residuals. Overall calibration of the model meets the calibration criteria defined in Section 2.0. As shown on Table E3, correlation between observed and simulated heads is good. The mean head residual and RMS error are significantly less than the ASTM guideline of five and ten percent of the model area groundwater elevation range.

Because the simulated groundwater elevations across the study area are well calibrated with observed elevations in both space and time, the model calibration is judged to be acceptable.

Accordingly, the model can be applied confidently to assess groundwater flow paths and flow rates and used to predict effects of recharge at the proposed spreading grounds.

E3.2 Simulated Heads

Model-simulated groundwater elevation contour maps and charts of observed and simulated elevations over time were constructed (Figures E11 and E12). For the 1994 through 2009 transient calibration, simulated groundwater elevations within the entire model domain range from around 3,600 feet above mean sea level (feet msl) at the eastern flux boundary in Pipes Wash to 2,600 feet msl in Giant Rock Subbasin.

The final calibrated model simulates flow conditions that are consistent with the basin conceptual model. Groundwater inflow occurs via the western boundary conditions along the mountain front. Within the model area, the groundwater elevation contour patterns reflect the boundary conditions, recharge sources and pumping sinks, and permeability zones, which cause changes in gradient magnitudes and directions. The low-permeability zones associated with the fault barriers result in groundwater elevation drops across the faults, particularly across the Pipes Barrier, where the water table difference across the fault is about 100 feet.

Groundwater elevation contour patterns for 1994 (Figure E11) are generally similar to patterns for 2009 (Figure E12), but 2009 groundwater elevations are lower reflecting the observed declines in basin wells. Figure E13 shows the simulated differences in groundwater elevations between 1994 and 2009. In the area of the proposed Reche spreading grounds, water levels declined between 20 and 30 feet from 1994 to 2009.

E3.3 Flowpath Results

Using the calibrated model, forward and reverse flowpaths were simulated using the USGS particle track code MODPATH. MODPATH uses flow budget files generated by MODFLOW and calculates groundwater flow paths and travel times for particles in the groundwater flow system. MODPATH was used to determine ultimate discharge points for particles entering the groundwater system as recharge as well as the capture zones of production wells. Forward flowpaths were simulated by generating single particles in selected individual model cells along the western model boundaries, which move advectively through the flow field. Reverse flowpaths were simulated by generating a series of particles in an arc around each pumping well which move advectively backward through the flow field to the sources of inflow contributing to the extraction point.

Figure E14 shows the forward flowpaths for particles generated along the western model boundaries. Forward particles track through the flow field and ultimately discharge to the production wells or into the Giant Rock Subbasin. Most of the flowpaths originating along the mountain front between Pipes and Ruby Mountain washes are captured by BDVWA production wells 2, 3, 4, and 8. The sources of water pumped from BDVWA wells 6, 7, and 9 include both inflow from Ruby Mountain Wash and adjacent mountain-front areas and septic return flows. The sources of water to production wells HDWD 24 and CSA No. 70 W-1 1, 2, and 3 are inflow via Pipes Wash and septic return flows. Figure E15 shows reverse track flowpaths or “capture zones” of the production wells.

E3.4 Water Balance and Volumetric Fluxes

Volumetric inflow and pumping data used as model input and subsurface outflow and change in storage rates generated by MODFLOW were plotted and evaluated to determine the magnitudes of water balance components within the model domain. Tables E4 and E5 summarize the annual and cumulative water balance results for the 1994-2009 transient simulation; water balance components over time are charted on Figure E16. The overall water balances for the model simulation had very low net error, and the magnitudes of inflows (through recharge and boundary conditions) and outflows (through boundaries and wells) are consistent and in accordance with the rates assigned in the basin conceptual model.

E3.5 Predicted Mounding and Flowpaths from Reche Spreading Grounds

To determine the fate of water recharged via the proposed spreading grounds, additional MODFLOW and forward MODPATH simulations were made using a future recharge scenario of three recharge events of 1,500 AF recharged over 6 months in alternating years. A six-acre recharge area was simulated in Pipes Wash, and transient flow was simulated in response to the multiple recharge events. Groundwater elevations and flowpaths were simulated over time and used to assess performance of the recharge facility and groundwater basin response.

For a surface recharge project, water levels rise beneath the recharge area creating a groundwater mound. The height and lateral extent of the mound varies over time as a function of aquifer hydraulic properties, recharge rate, and recharge area. The development of a groundwater mound beneath the spreading grounds was evaluated using the MODFLOW model. The model estimates the groundwater elevations and corresponding height of the groundwater recharge mound as a function of time and distance from the recharge area.

The calculated shape of the mound at the end of the first six-month recharge period is illustrated on Figure E17. The mound height directly beneath the spreading grounds over time is illustrated on Figure E18. As shown on the figures, the maximum mound height beneath the spreading grounds is approximately 19 feet after the first six-month recharge period, 20 feet after the second six-month recharge period, and 22 feet after the third six-month recharge period. Groundwater levels are expected to increase 1 foot or more up to 8,000 feet to the northwest of the spreading grounds. As shown on Figure 15, water levels contours stack up against Pipes

Barrier due to the low permeability of the fault zone. The predicted maximum groundwater level rise is approximately 5 feet at HDWD 24 (4,300 feet from the center of the spreading grounds).

To assess the fate of recharged water, MODPATH particles were started at the water table beneath the spreading grounds and forward-tracked to their downgradient discharge locations. Figure E19 shows the simulated groundwater flowpaths from the Reche Spreading Grounds after three 6-month recharge events. As shown on the figure, recharge water diverges radially away from the recharge area before trending northeast in the general direction of HDWD 24. The travel time between the recharge site and HDWD 24 is approximately 2 to 3 years.

E4. REFERENCES

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Lewis, R.E. (1972) Ground-water Resources of the Yucca Valley-Joshua Tree Area, San Bernardino County, California. USGS Open File Report.

United States Environmental Protection Agency (USEPA) (2003) CHEMFLO™-2000: Interactive Software for Simulating Water and Chemical Movement in Unsaturated Soils. National Risk Management Research Laboratory (by D.L. Nofziger and Jinqaun Wu, Department of Plant and Soil Sciences, Oklahoma State University).

Tables

Table E1
Boundary Condition Specified Flux Rates

Flux Arc	1	2	3	4	5	6	7	8	9	Total Influx Western Model Boundary
	North of Ruby Mountain Wash (+ return flow)	Ruby Mountain Wash	South of Ruby Wash (+ return flow)	North of Whalen's Wash (no return flow)	Whalen's Wash	South of Whalen's Wash (no return flow)	South of Whalen's Wash (+ return flow)	North of Pipes Wash (no return flow)	Pipes Wash	
Water Year										
1994-95	17	106	15	12	194	8	21	11	690	1,073
1995-96	17	136	15	12	252	8	21	11	893	1,366
1996-97	19	84	16	12	159	8	21	11	559	890
1997-98	21	45	17	12	84	8	21	11	296	515
1998-99	20	115	14	12	212	8	20	11	756	1,168
1999-00	25	69	16	12	125	8	21	11	450	736
2000-01	27	42	19	12	77	8	22	11	275	493
2001-02	29	34	20	12	63	8	22	11	224	424
2002-03	25	15	20	12	29	8	23	11	101	244
2003-04	23	62	20	12	115	8	22	11	406	680
2004-05	23	54	18	12	100	8	23	11	355	604
2005-06	22	157	16	12	288	8	22	11	1,028	1,564
2006-07	24	105	17	12	191	8	23	11	676	1,066
2007-08	24	41	18	12	74	8	24	11	266	477
2008-09	21	59	18	12	108	8	24	11	379	640
Average	23	75	17	12	138	8	22	11	490	796

Values in acre-feet

Table E2
Well Production

	BDVWA							HDWD	CSA 70 W-1			BDVWA	HDWD	CSA 70 W-1	Total Well
	Well 2	Well 3	Well 4	Well 6	Well 7	Well 8	Well 9	Well 24	Well 1	Well 2	Well 3	Total	Total	Total	Production
Water Year															
1994-95	88	112	124	109	79	404	20	495	67	71	0	935	495	138	1,568
1995-96	88	231	219	99	80	305	89	815	107	98	166	1,112	815	370	2,297
1996-97	79	77	80	156	190	77	78	511	149	99	40	737	511	288	1,537
1997-98	87	90	82	156	156	110	135	851	94	86	55	815	851	235	1,901
1998-99	37	38	39	51	-	57	168	773	77	67	117	391	773	261	1,424
1999-00	27	0	109	41	22	72	135	532	45	38	116	406	532	198	1,135
2000-01	45	0	50	33	28	66	175	706	60	40	91	398	706	191	1,296
2001-02	60	39	79	51	40	42	202	755	35	30	56	515	755	120	1,390
2002-03	34	37	20	47	35	110	184	549	28	24	79	468	549	131	1,148
2003-04	41	30	81	39	52	49	171	723	30	29	77	464	723	136	1,322
2004-05	10	17	58	28	34	116	180	473	43	42	63	442	473	149	1,064
2005-06	35	35	48	12	73	113	175	255	48	47	61	490	255	155	899
2006-07	65	49	42	33	91	73	145	514	48	48	48	499	514	144	1,156
2007-08	54	39	27	145	98	100	13	599	48	150	48	476	599	246	1,321
2008-09	50	64	-	118	73	96	62	640	51	63	69	462	640	183	1,285
Average	53	57	71	74	70	119	129	613	62	62	72	574	613	196	1,383

Values in acre-feet
BDVWA = Bighorn-Desert View Water Agency
HDWD = Hi-Desert Water District
CSA 70 W-1 = San Bernardino County Service Area 70 W-1

Table E3
Model Calibration Summary

Well	Measured Nov-1994 Groundwater Elevation (feet msl)	Measured Sep/Oct-2009 Groundwater Elevation (feet msl)	Mean Error Measured minus Simulated	Root Mean Error Measured minus Simulated
BDVWA 1	3247.50	Dry	-0.60	2.35
BDVWA 2	3245.48	3225.01	-2.10	3.55
BDVWA 3	3245.34	3224.84	-2.55	3.77
BDVWA 4	3245.17	3230.27	-2.59	3.32
BDVWA 6	2912.85	2895.05	2.20	4.40
BDVWA 7	2913.88	2895.71	2.43	4.45
BDVWA 8	3242.88	3222.28	-2.26	4.22
BDVWA 9	2923.47	2909.00	0.68	3.02
HDWD 24	3009.00	2985.73	-7.41	8.61
CSA 70 W-1 1	2867.00	2834.00	-9.37	10.80
CSA 70 W-1 2	2867.50	2849.50	-7.86	9.38
1N/5E-2N1	3462.73[a]	3465.52	7.83	16.79
USGS Monitoring	3246.80	3228.10	-1.49	3.16
Gubler Farm 1G1	2897.60	2906.10	-0.23	1.87
Gubler Farm 1K1	2897.60	2903.92	-5.54	5.93
Average			-1.92	5.71

[a]May-1994 measurement

Table E4
Annual Water Budget

	Subsurface Inflow	Return Flow	Pumping	Subsurface Outflow[1]	Annual Storage Change
Water Year					
1994-95	1,051	204	-1,568	-579	-893
1995-96	1,344	204	-2,297	-579	-1,329
1996-97	864	238	-1,537	-579	-1,014
1997-98	486	240	-1,901	-579	-1,754
1998-99	1,144	243	-1,424	-579	-617
1999-00	705	268	-1,135	-579	-742
2000-01	456	297	-1,296	-579	-1,122
2001-02	382	293	-1,390	-579	-1,294
2002-03	207	304	-1,148	-579	-1,216
2003-04	645	270	-1,322	-579	-986
2004-05	570	265	-1,064	-579	-808
2005-06	1,534	252	-899	-579	308
2006-07	1,033	273	-1,156	-579	-429
2007-08	442	295	-1,321	-579	-1,163
2008-09	608	273	-1,285	-579	-984
Average	765	261	-1,383	-579	-936

Values in acre-feet

[1]Value represents average based on steady-state simulation

Table E5
Cumulative Water Budget

	Cumulative Subsurface Inflow	Cumulative Return Flow	Cumulative Pumping	Cumulative Subsurface Outflow	Cumulative Annual Storage Change
Water Year					
1994-95	1,051	204	-1,568	-579	-893
1995-96	2,394	407	-3,865	-1,159	-2,222
1996-97	3,258	646	-5,402	-1,738	-3,236
1997-98	3,744	886	-7,303	-2,317	-4,991
1998-99	4,888	1,129	-8,727	-2,896	-5,607
1999-00	5,593	1,397	-9,863	-3,476	-6,349
2000-01	6,049	1,694	-11,159	-4,055	-7,471
2001-02	6,431	1,987	-12,548	-4,634	-8,764
2002-03	6,638	2,291	-13,696	-5,213	-9,980
2003-04	7,282	2,562	-15,018	-5,793	-10,966
2004-05	7,853	2,827	-16,082	-6,372	-11,774
2005-06	9,387	3,079	-16,981	-6,951	-11,466
2006-07	10,419	3,352	-18,137	-7,530	-11,896
2007-08	10,861	3,647	-19,458	-8,110	-13,059
2008-09	11,469	3,920	-20,743	-8,689	-14,043

Values in acre-feet

Figures

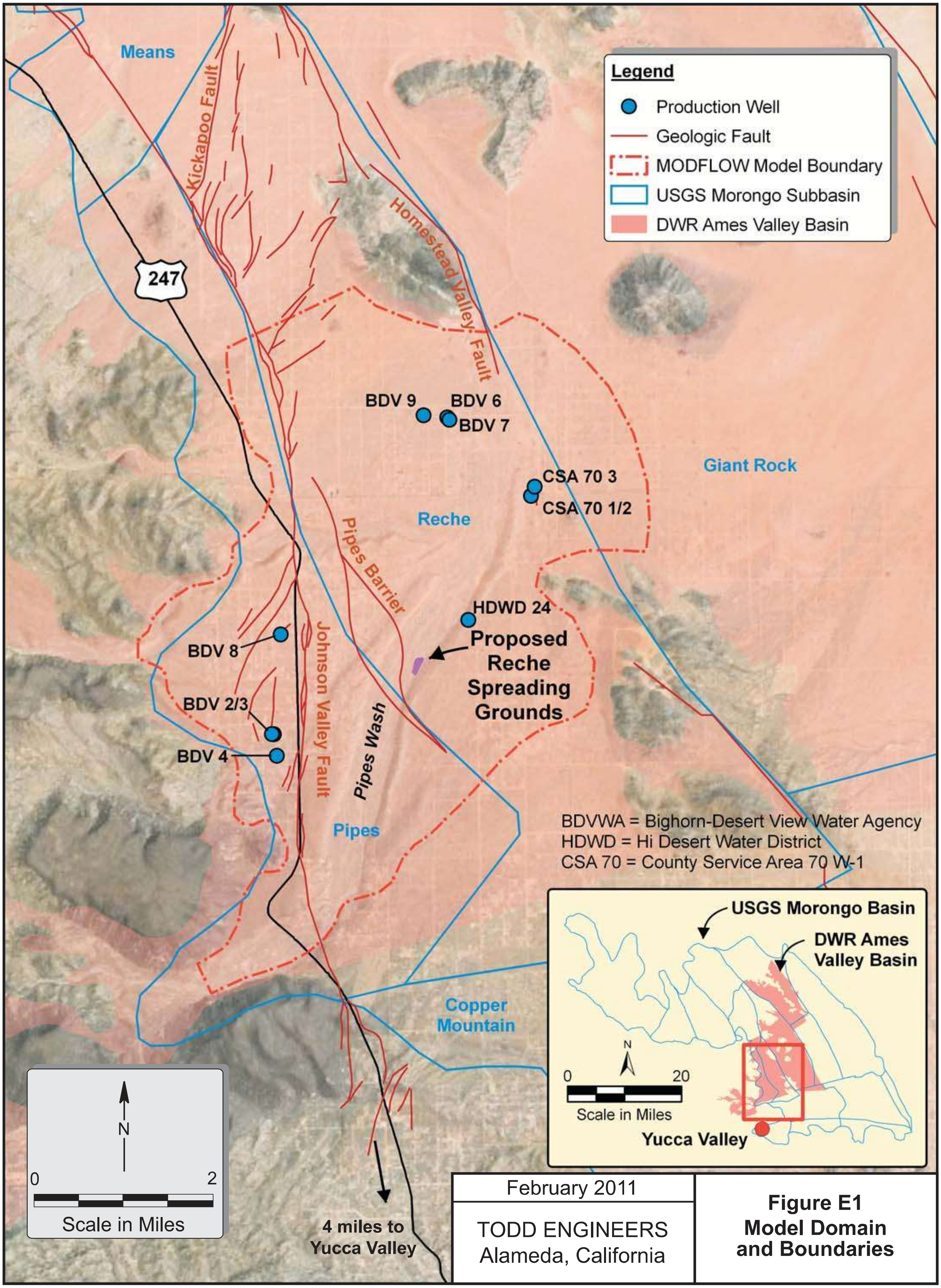

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Figure E1
Model Domain and Boundaries

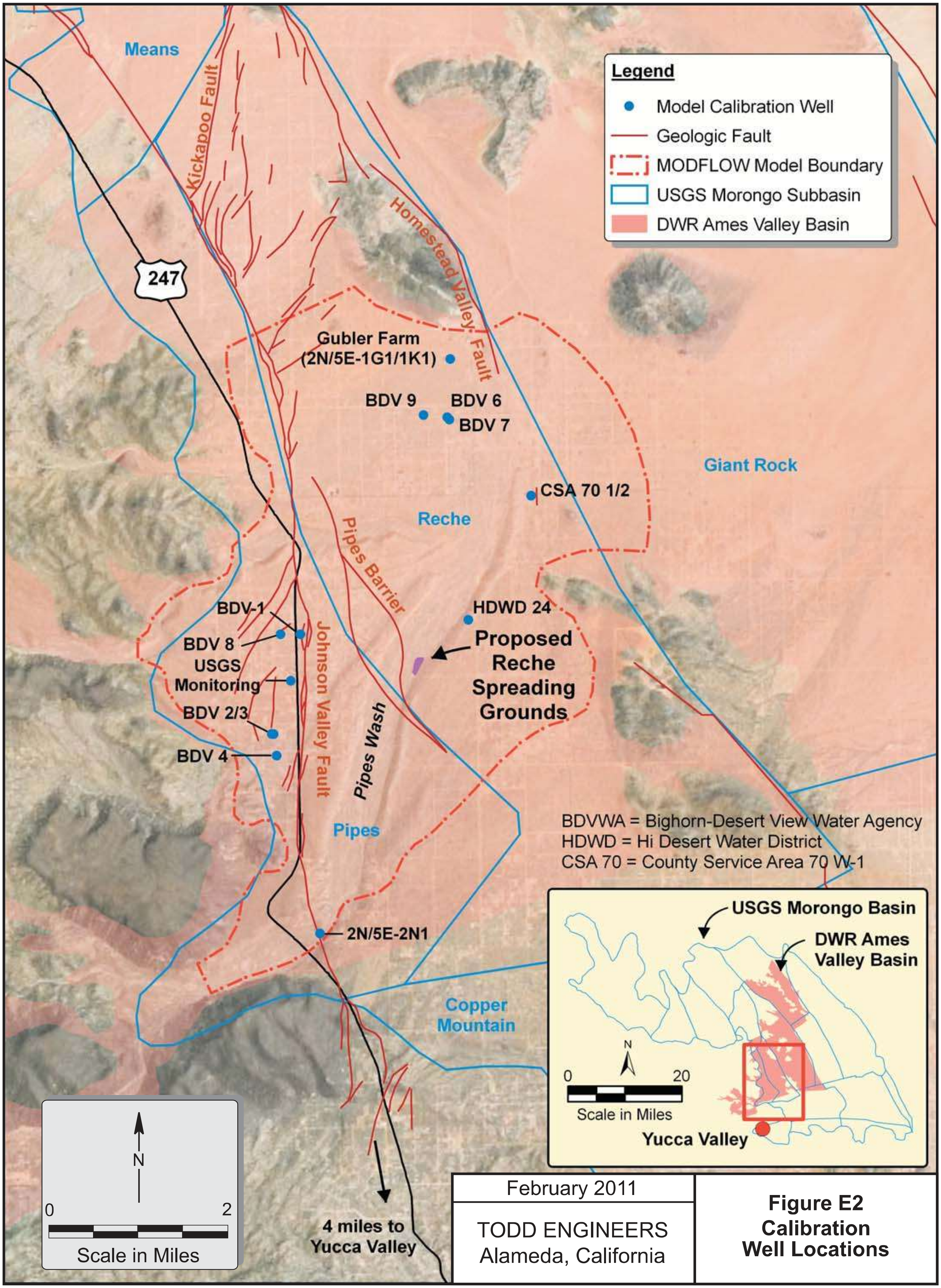

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Figure E2
Calibration
Well Locations

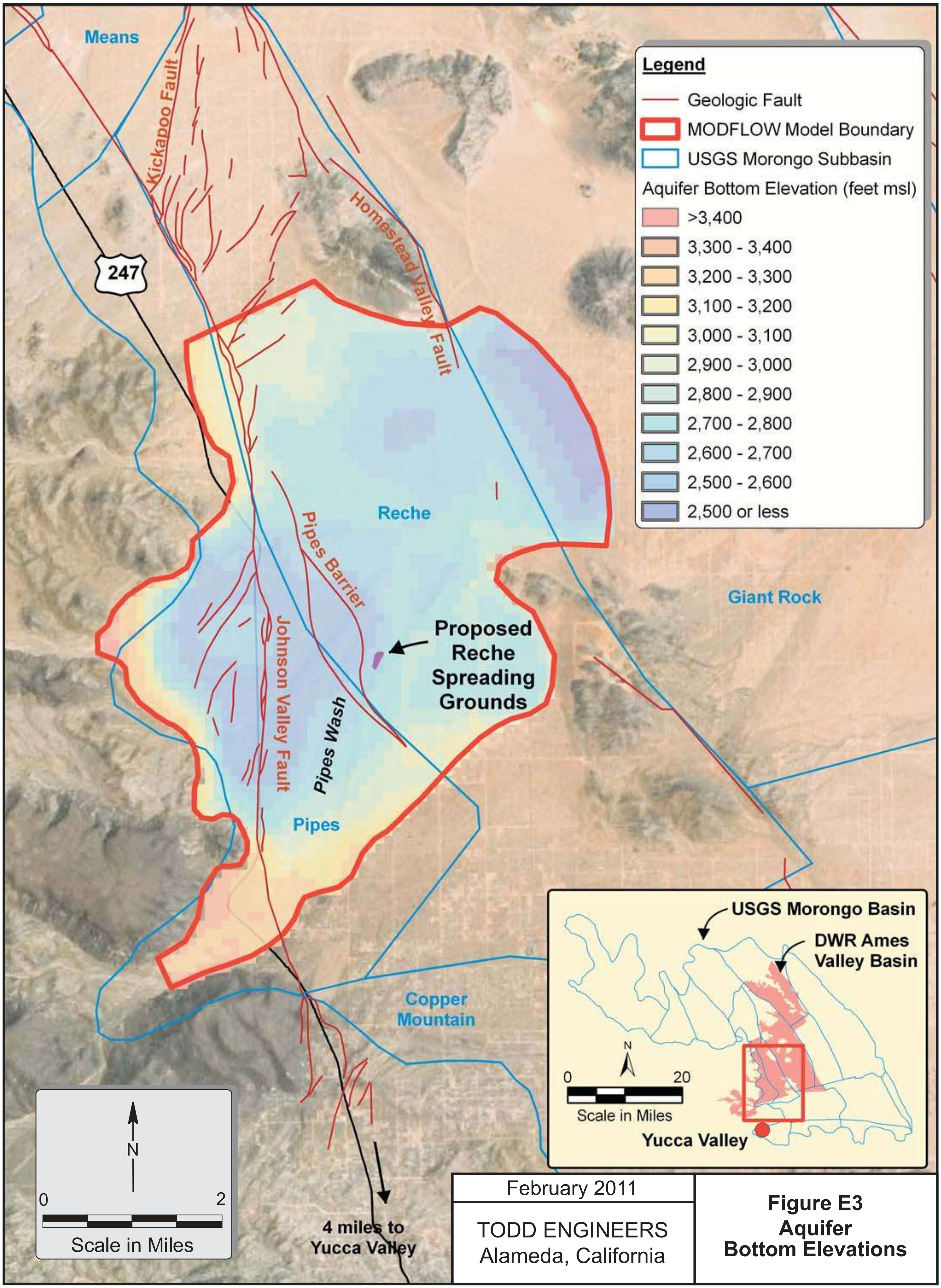

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Figure E3
Aquifer Bottom Elevations

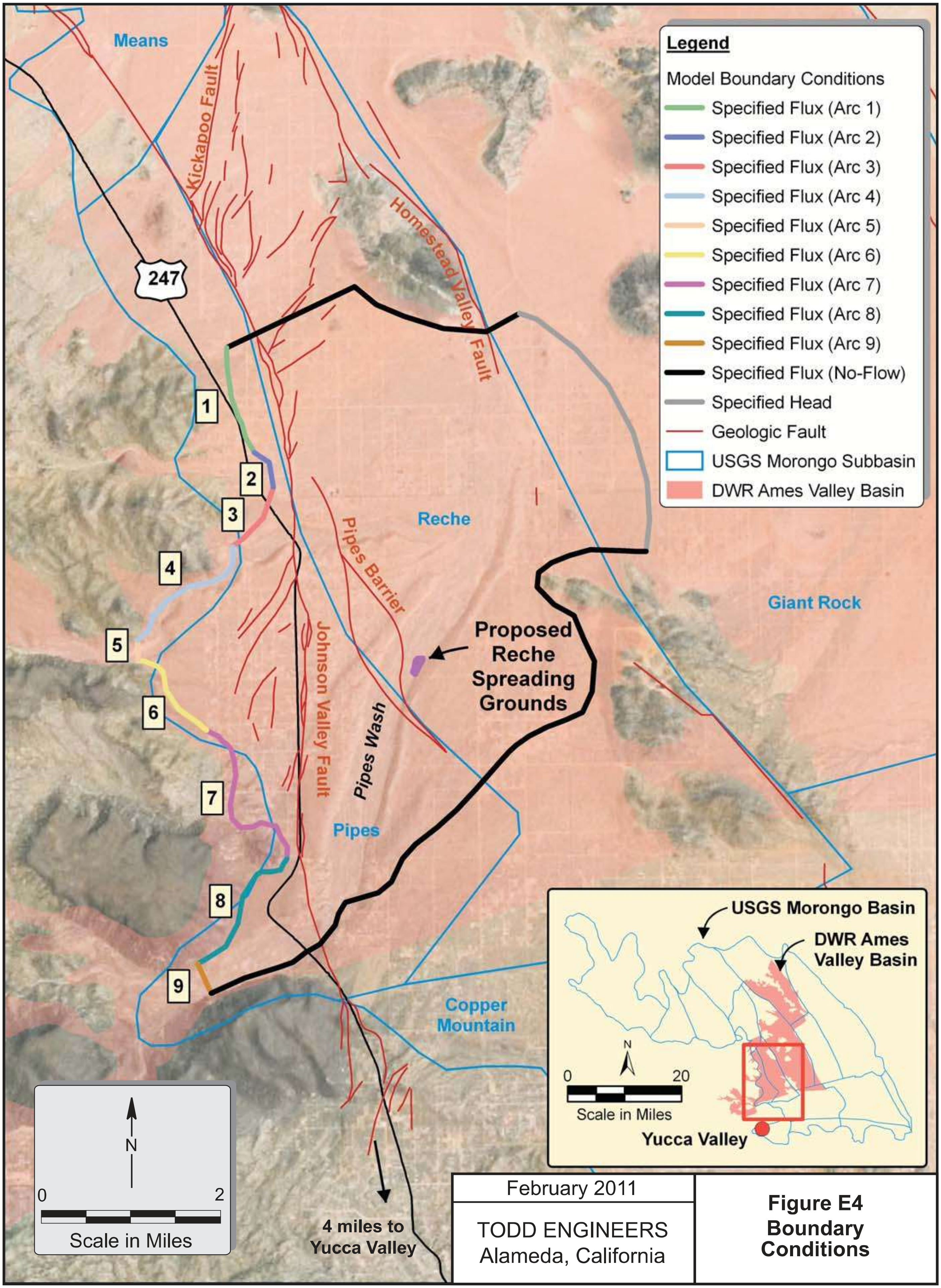
Legend
Model Boundary Conditions
Specified Flux (Arc 1)
Specified Flux (Arc 2)
Specified Flux (Arc 3)
Specified Flux (Arc 4)
Specified Flux (Arc 5)
Specified Flux (Arc 6)
Specified Flux (Arc 7)
Specified Flux (Arc 8)
Specified Flux (Arc 9)
Specified Flux (No-Flow)
Specified Head
Geologic Fault
USGS Morongo Subbasin
DWR Ames Valley Basin
Means
Kickapoo Fault
Homestead Valley Fault
247
1
2
3
4
5
6
7
8
9
Reche
Pipes Barrier
Johnson Valley Fault
Pipes Wash
Proposed Reche Spreading Grounds
Giant Rock
Pipes
Copper Mountain
USGS Morongo Basin
DWR Ames Valley Basin
N
0
20
Scale in Miles
Yucca Valley
N
0
2
Scale in Miles
4 miles to Yucca Valley
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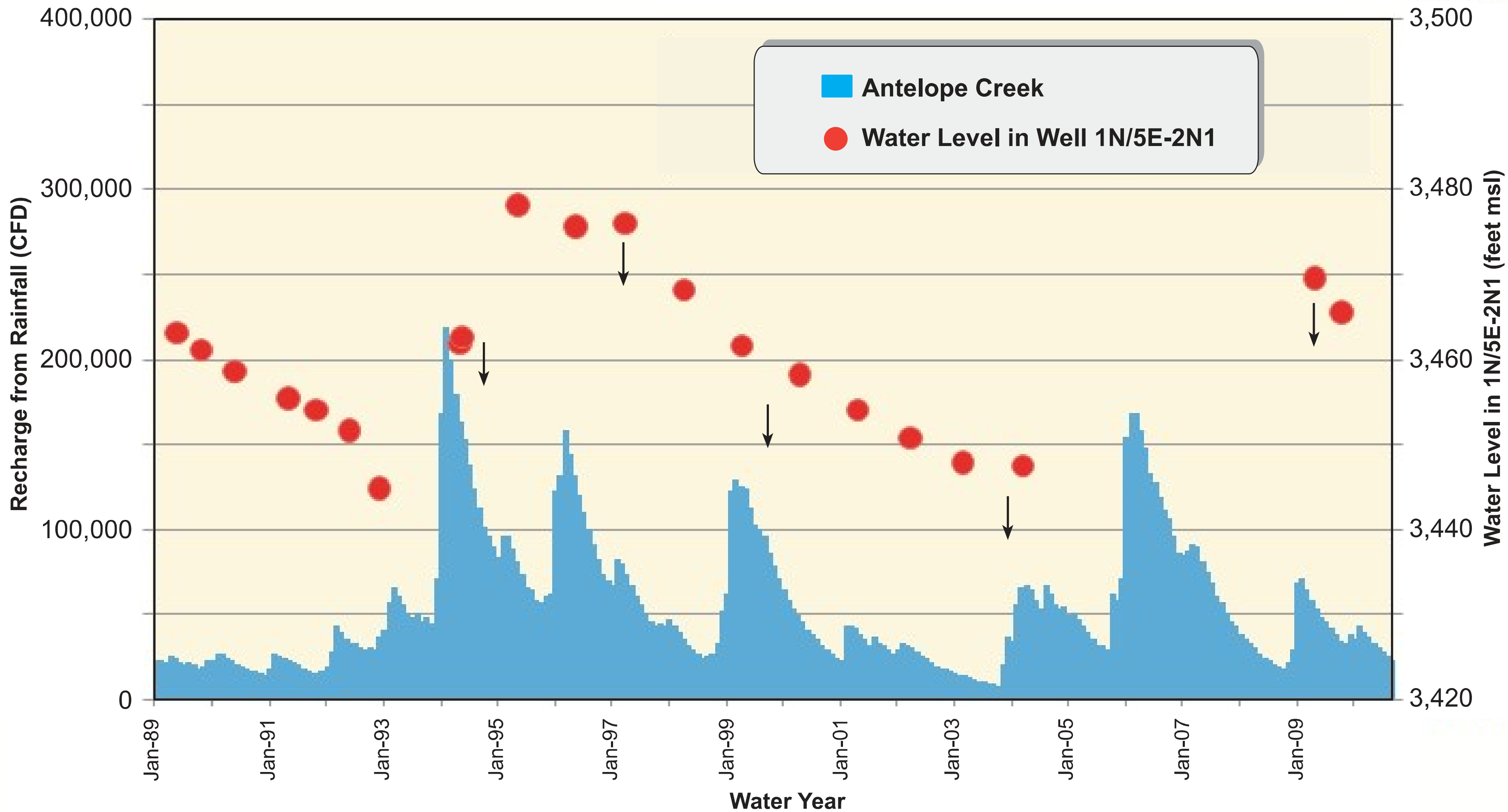
Figure E4
Boundary Conditions



February 2011	**Figure E5** **Relationship Between Rainfall and Water Levels in Pipes Wash Well 1N/5E-2N1**
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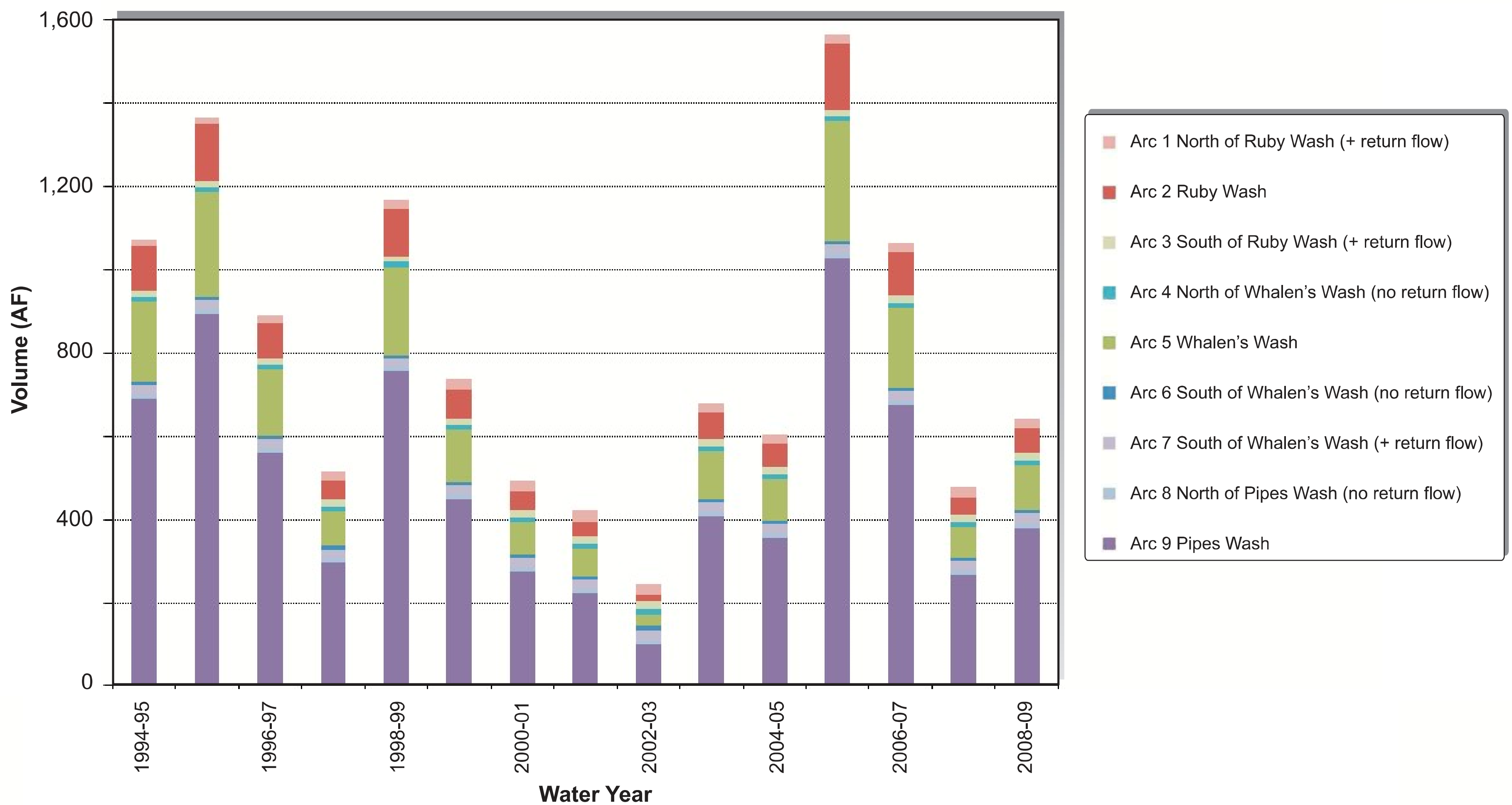

February 2011 TODD ENGINEERS Alameda, California	**Figure E6** **Recharge Rates** **for** **Western Flux Boundary**

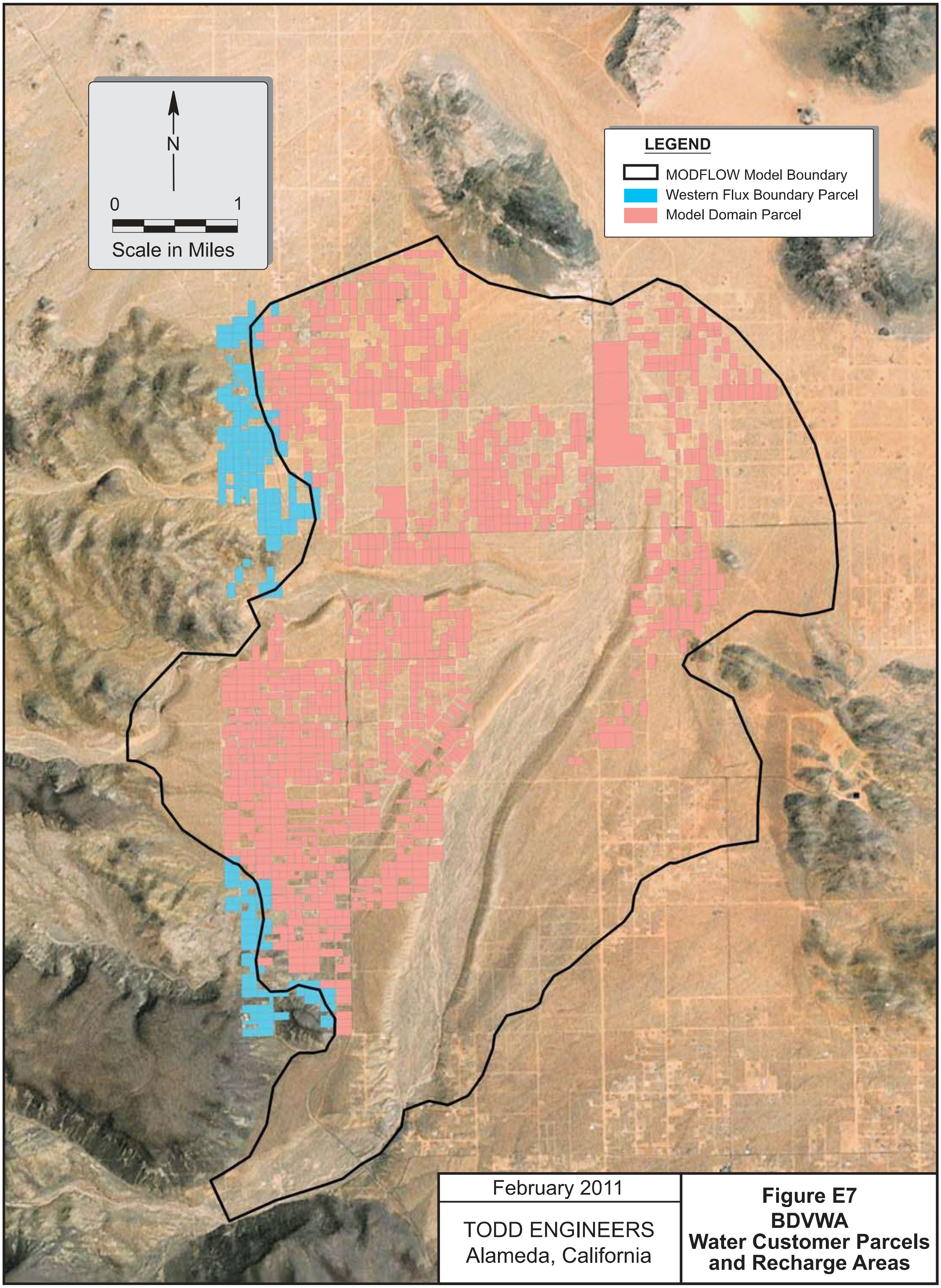

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Figure E7
BDVWA
Water Customer Parcels and Recharge Areas

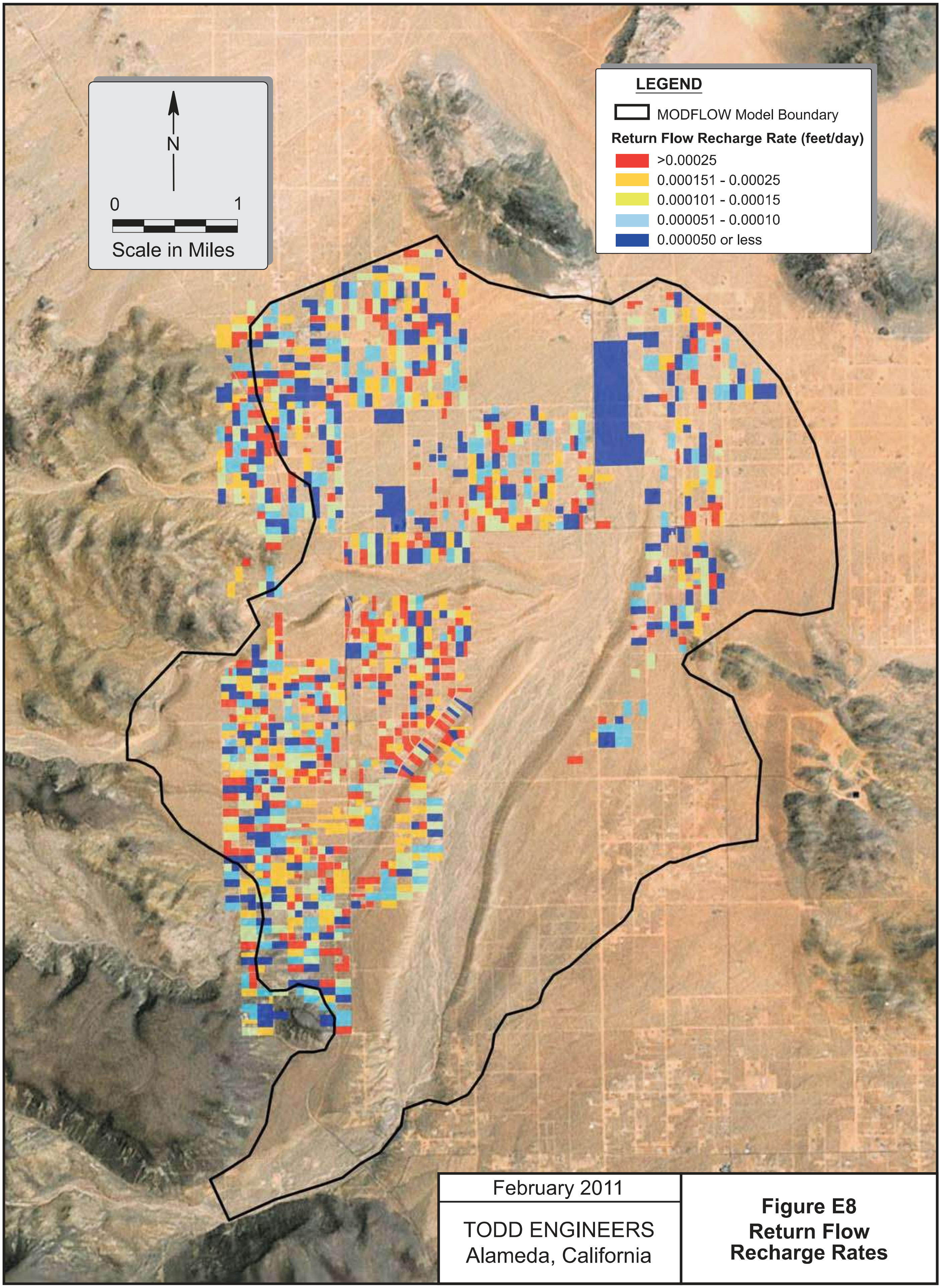

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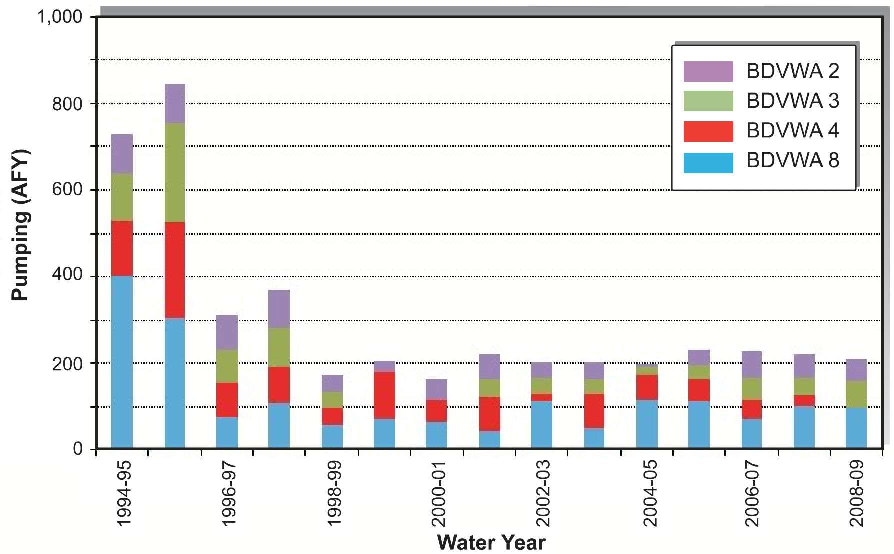
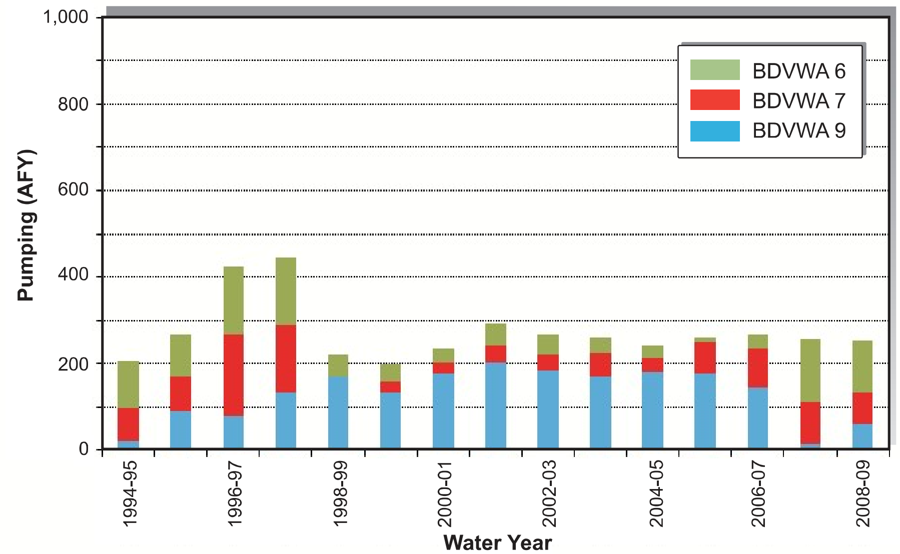
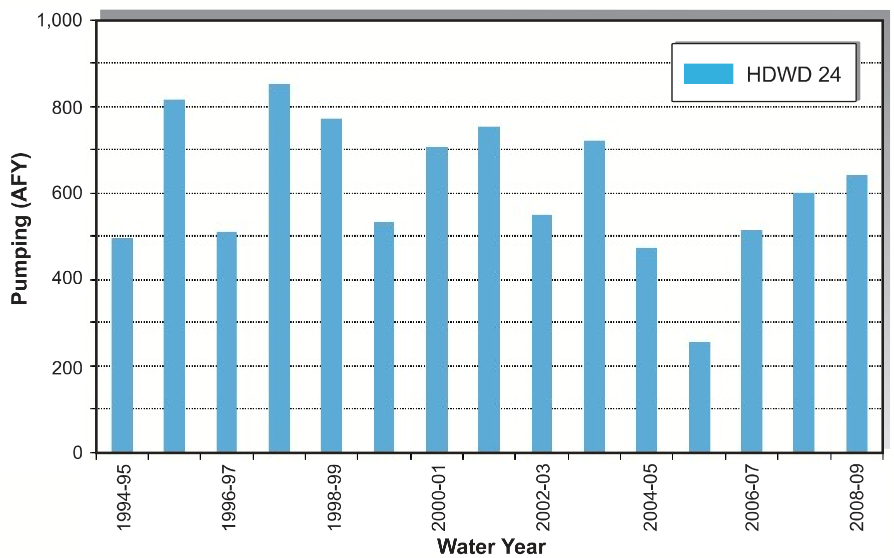
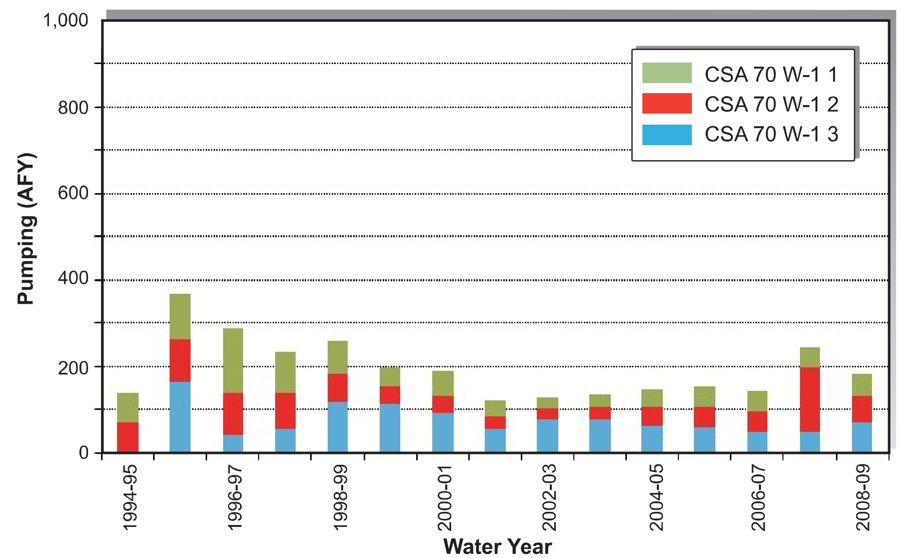
Figure E8
Return Flow Recharge Rates









February 2011	**Figure E9**
TODD ENGINEERS Alameda, California	**Production Well Pumping Rates**

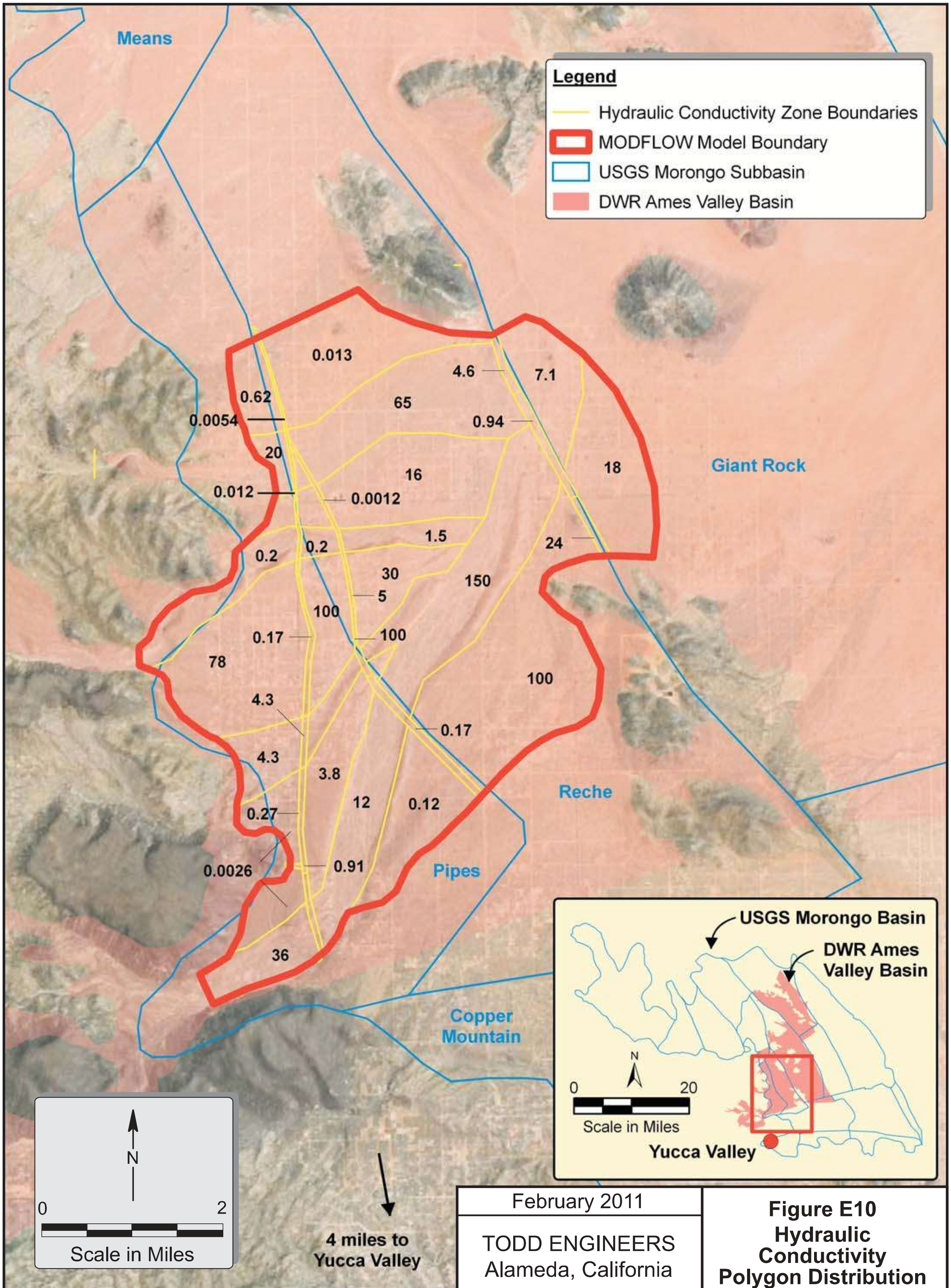


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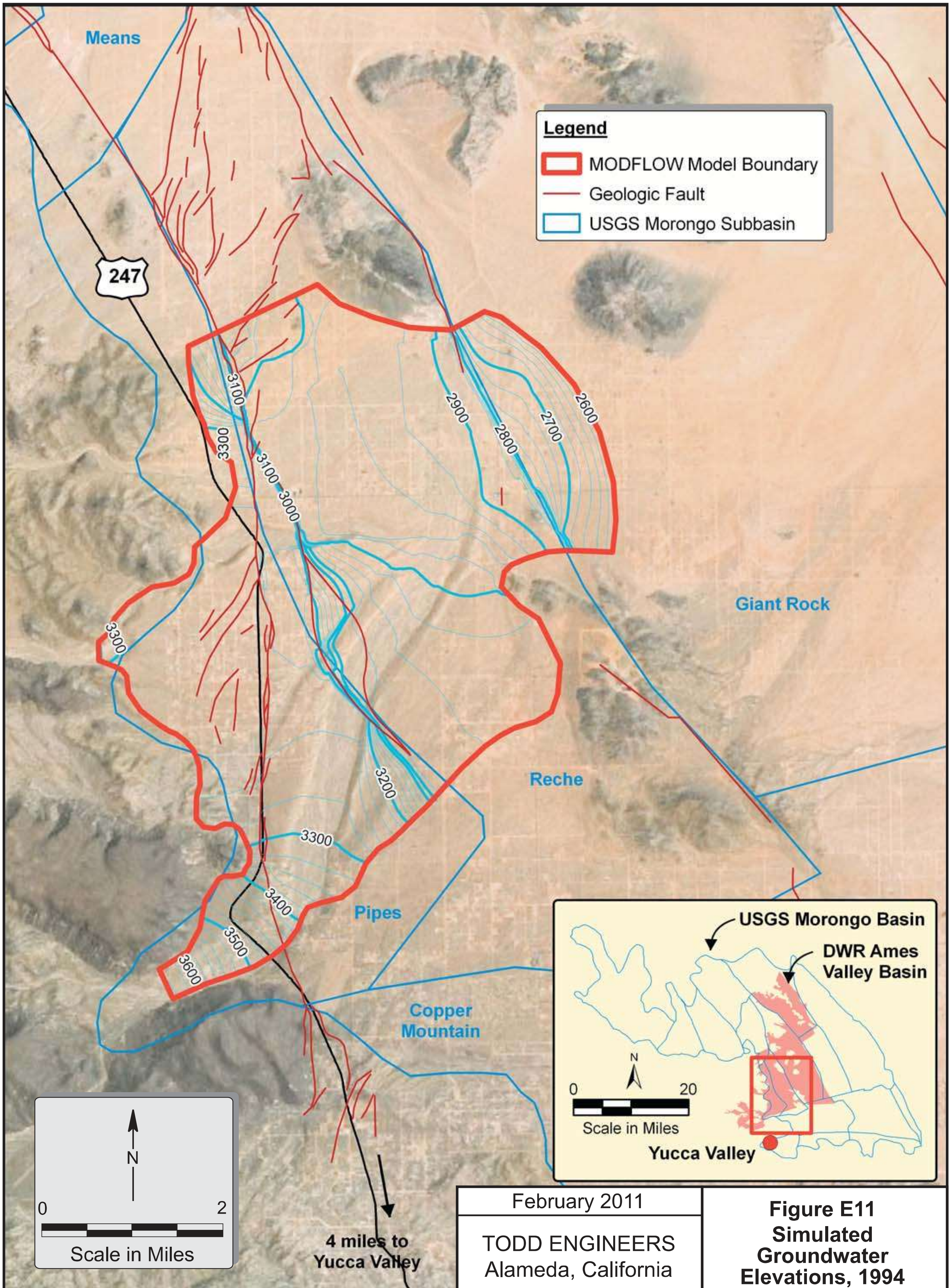
Figure E10
Hydraulic Conductivity Polygon Distribution



February 2011	Figure E11 Simulated Groundwater Elevations, 1994
TODD ENGINEERS Alameda, California	

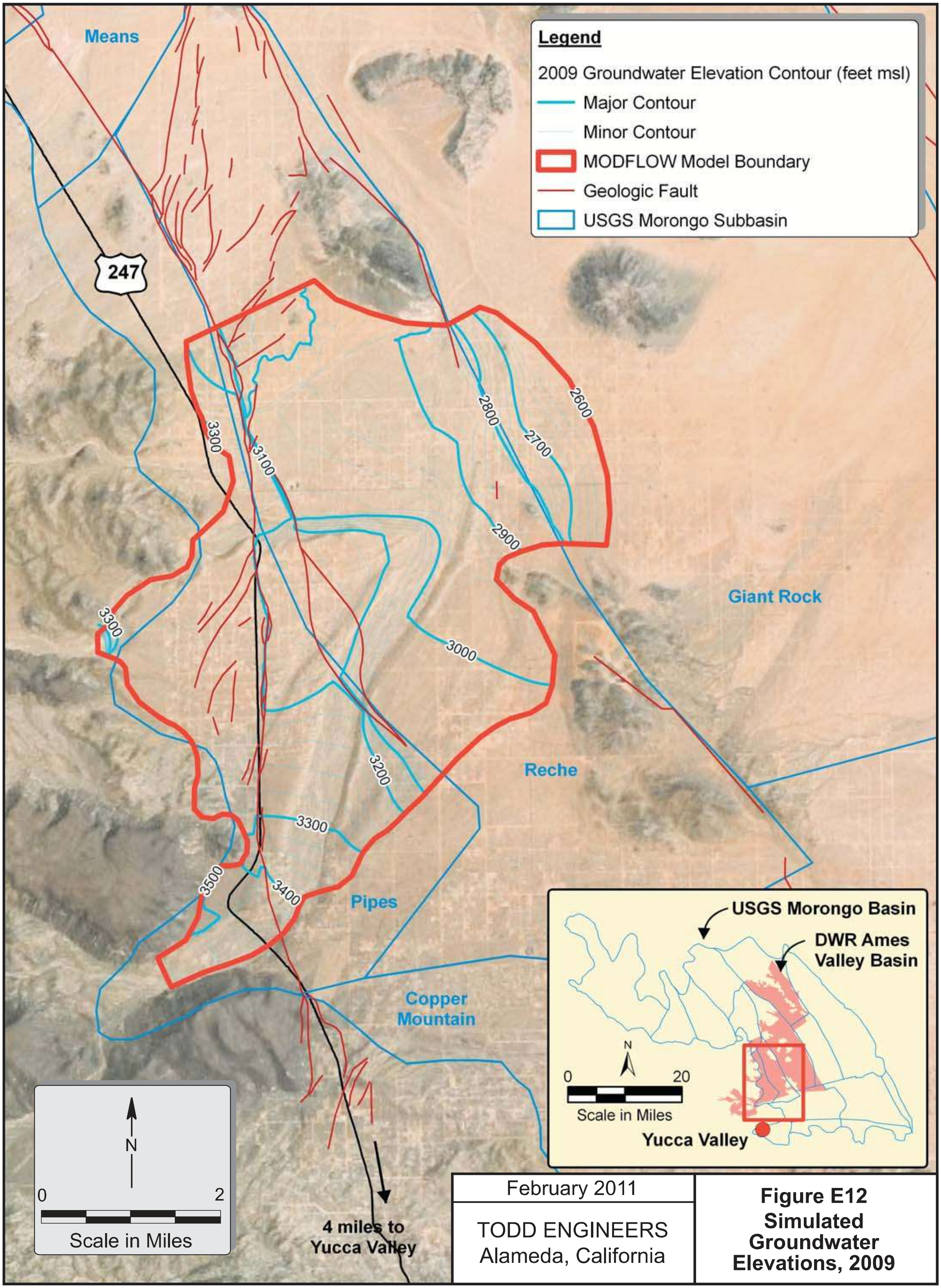


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Figure E12
Simulated Groundwater Elevations, 2009

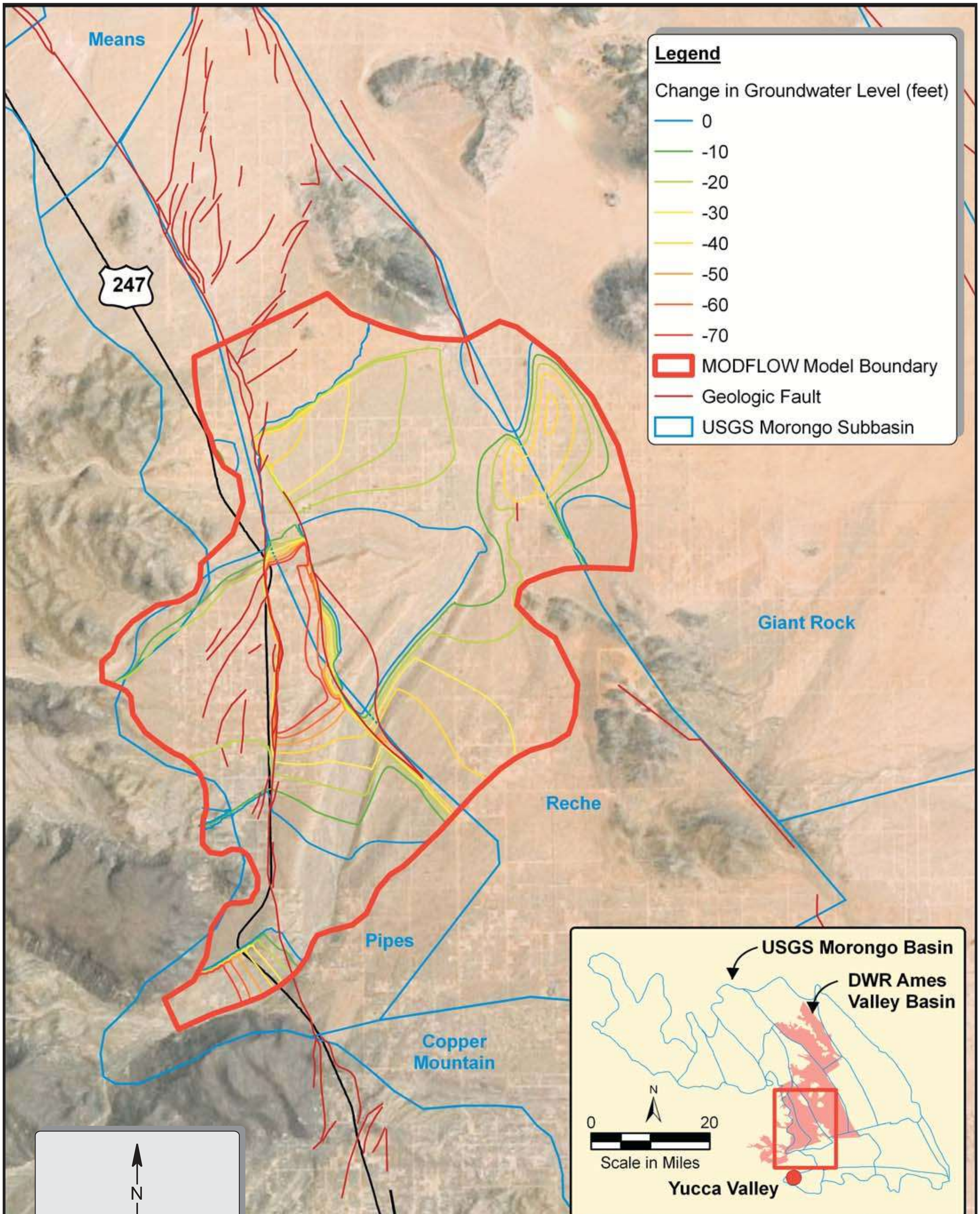

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Figure E13
Simulated Groundwater Elevation Change 1994 to 2009

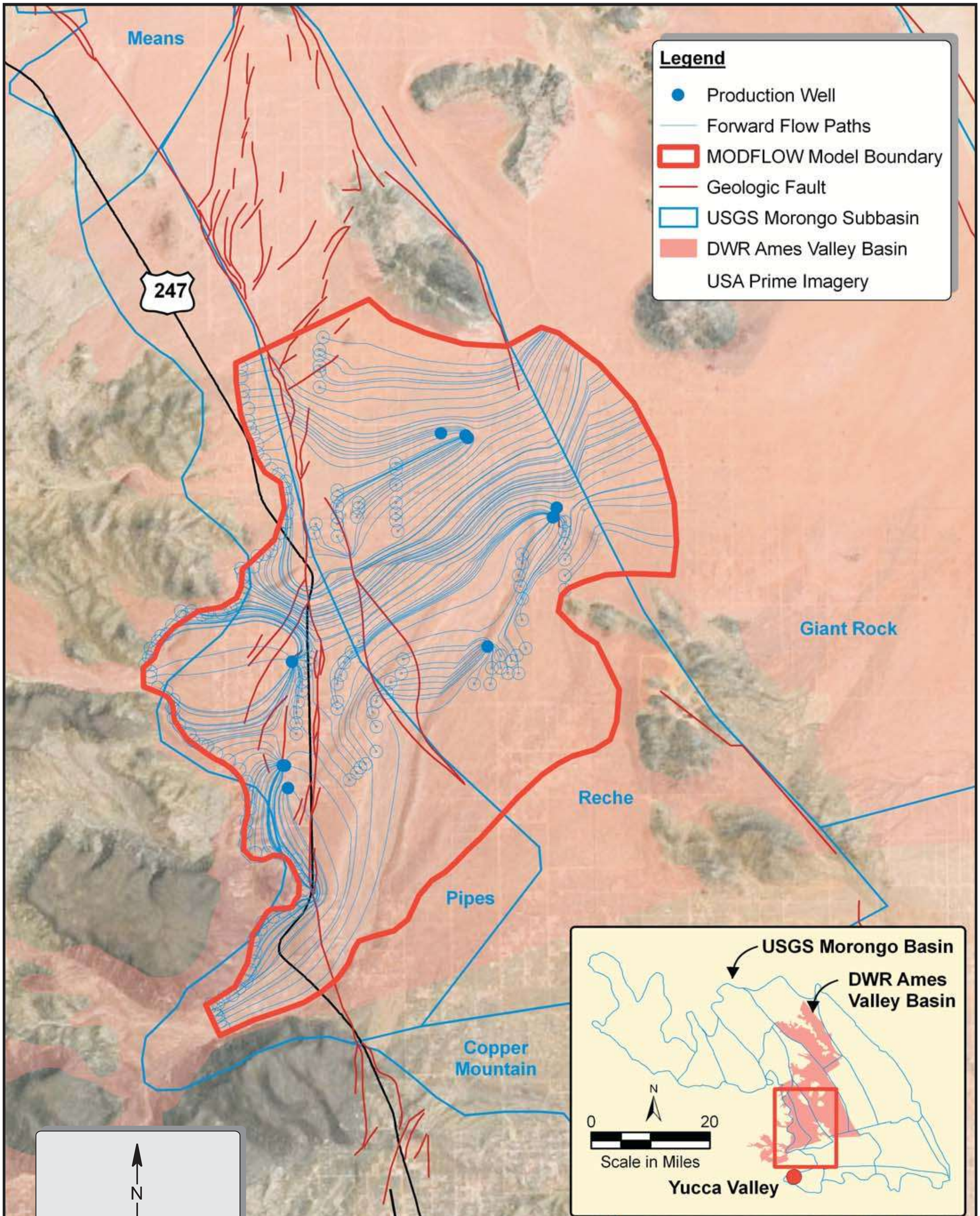

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Figure E14
Simulated Forward Flow Paths, 2009 Conditions

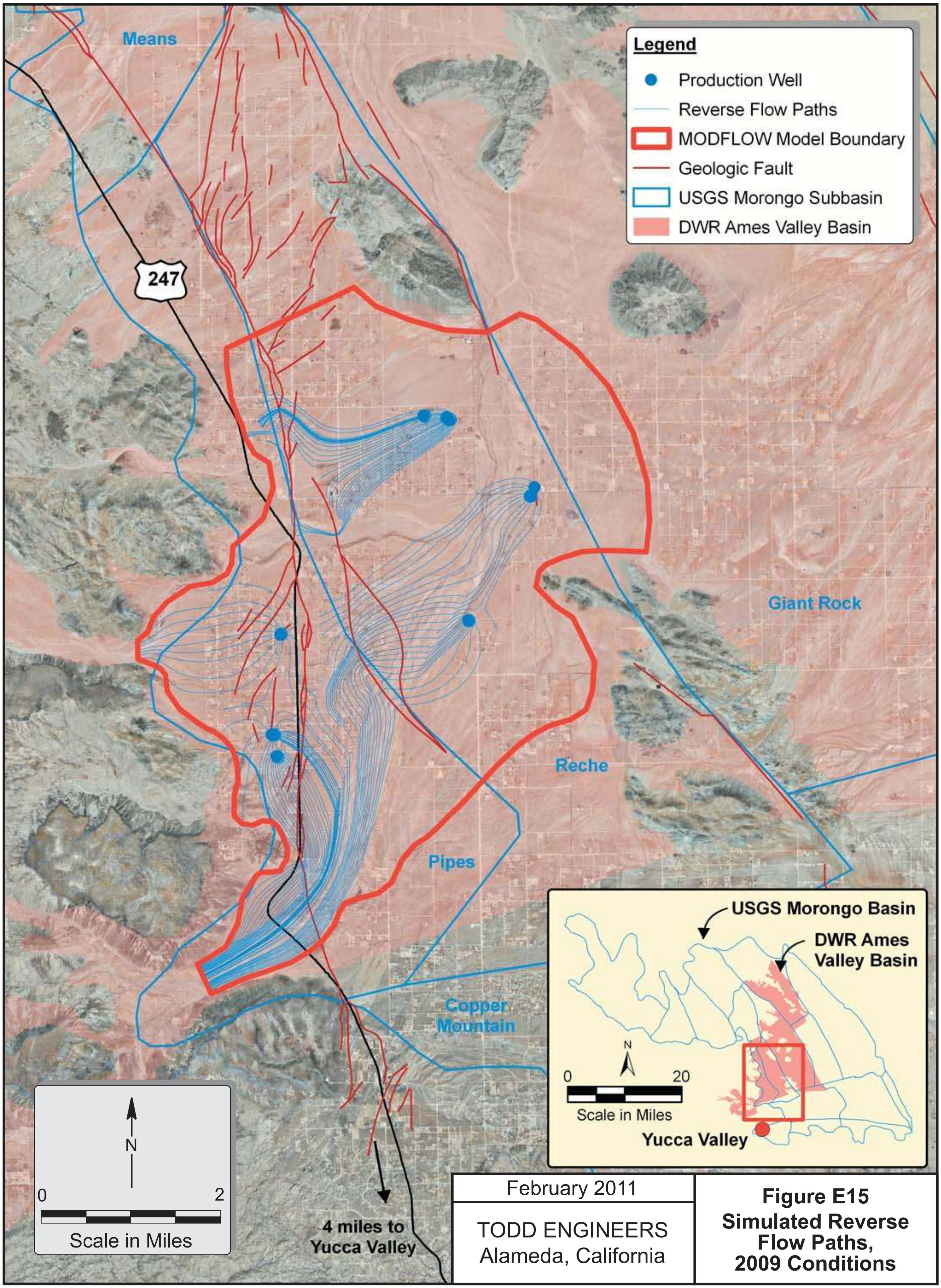

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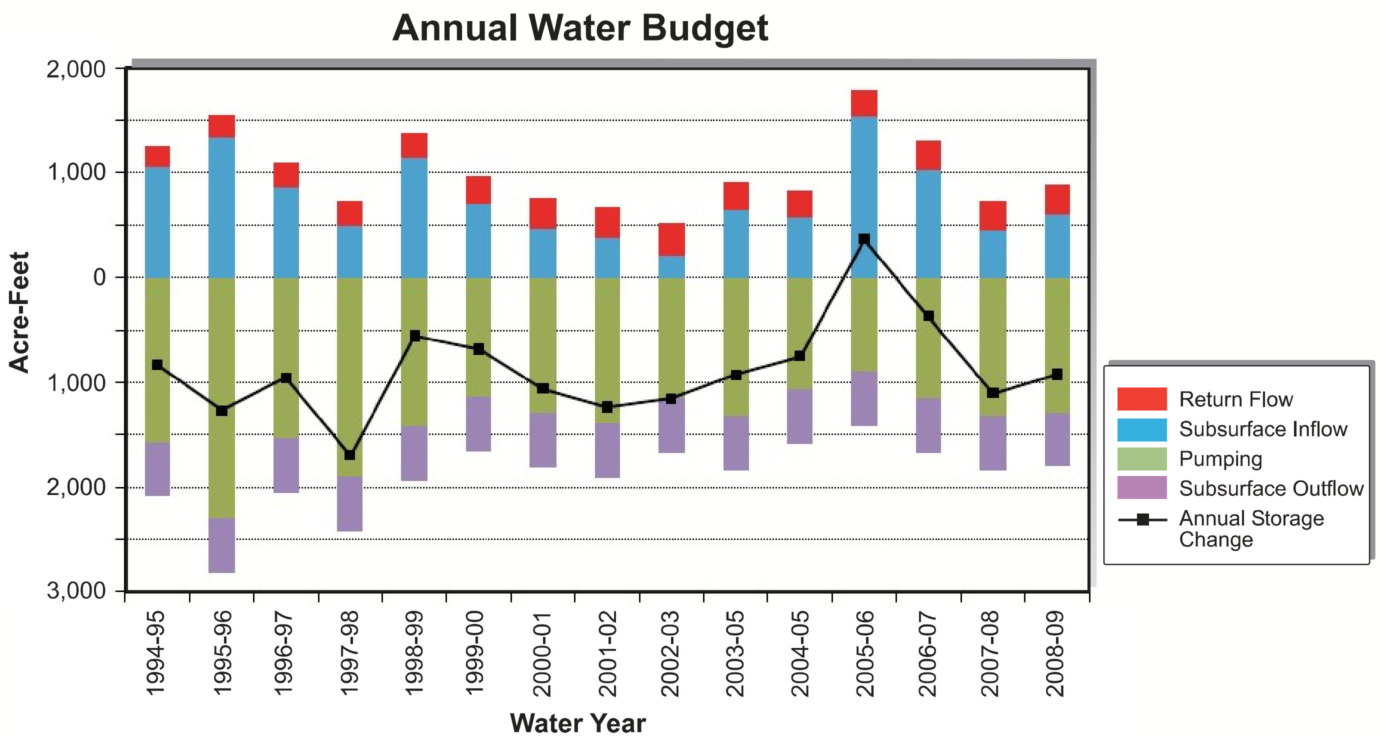
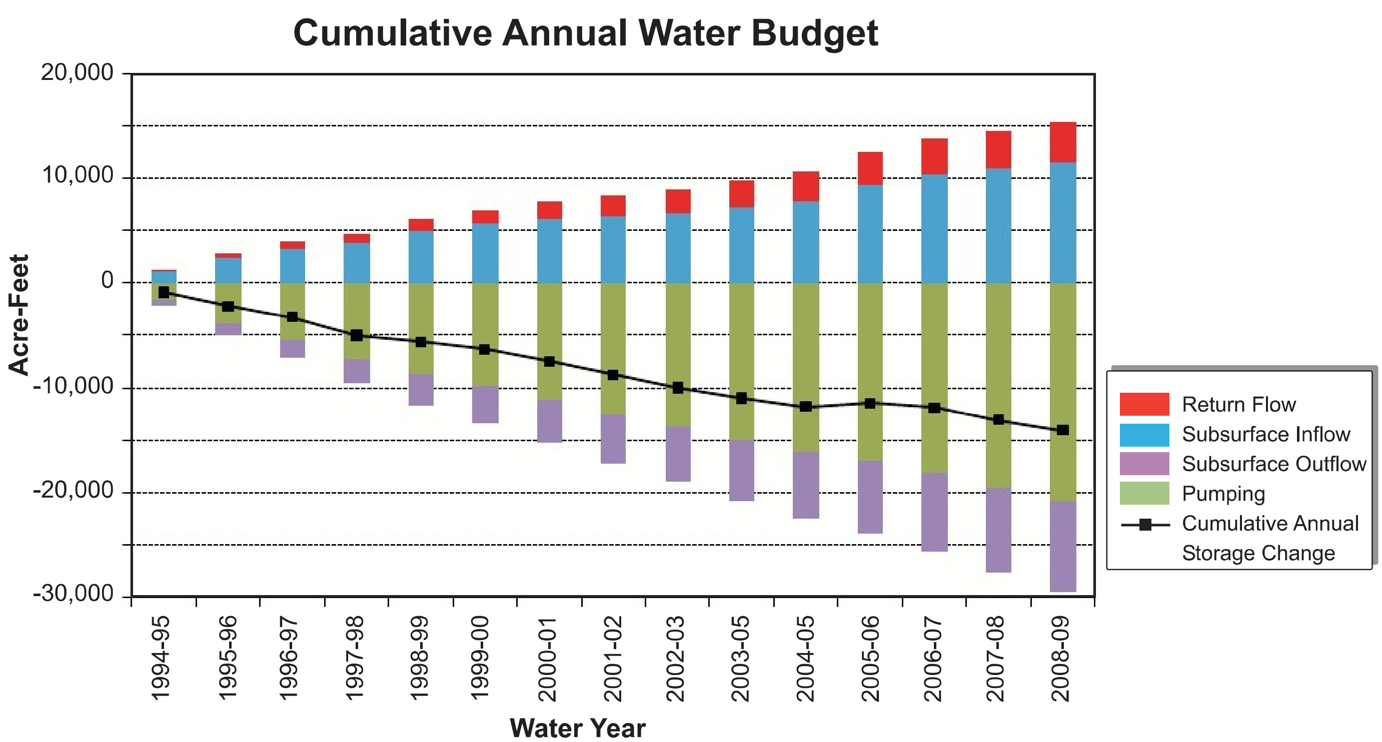
Figure E15
Simulated Reverse Flow Paths, 2009 Conditions





February 2011	**Figure E16**
TODD ENGINEERS Alameda, California	**Water Budget Summary 1994 - 2009**

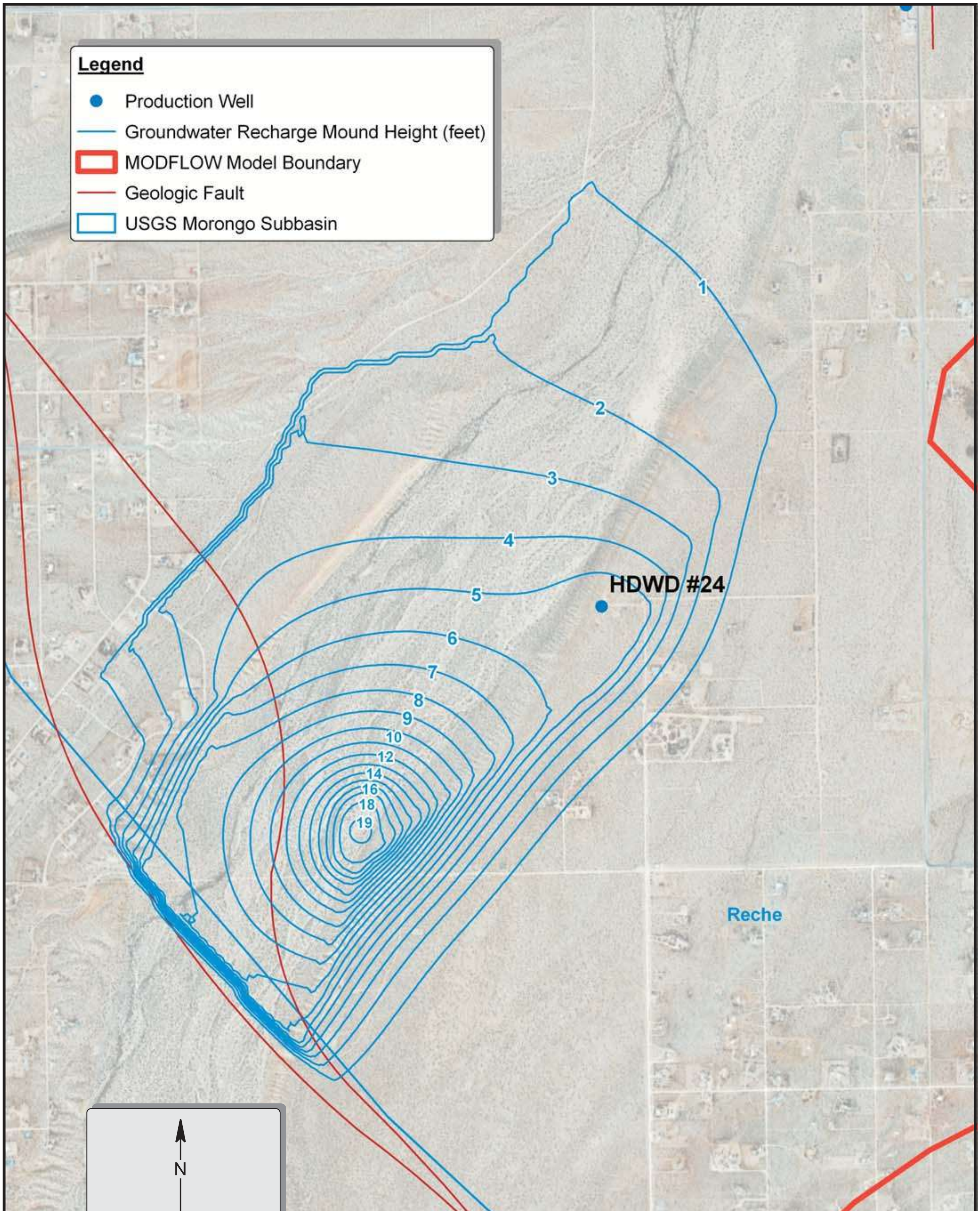


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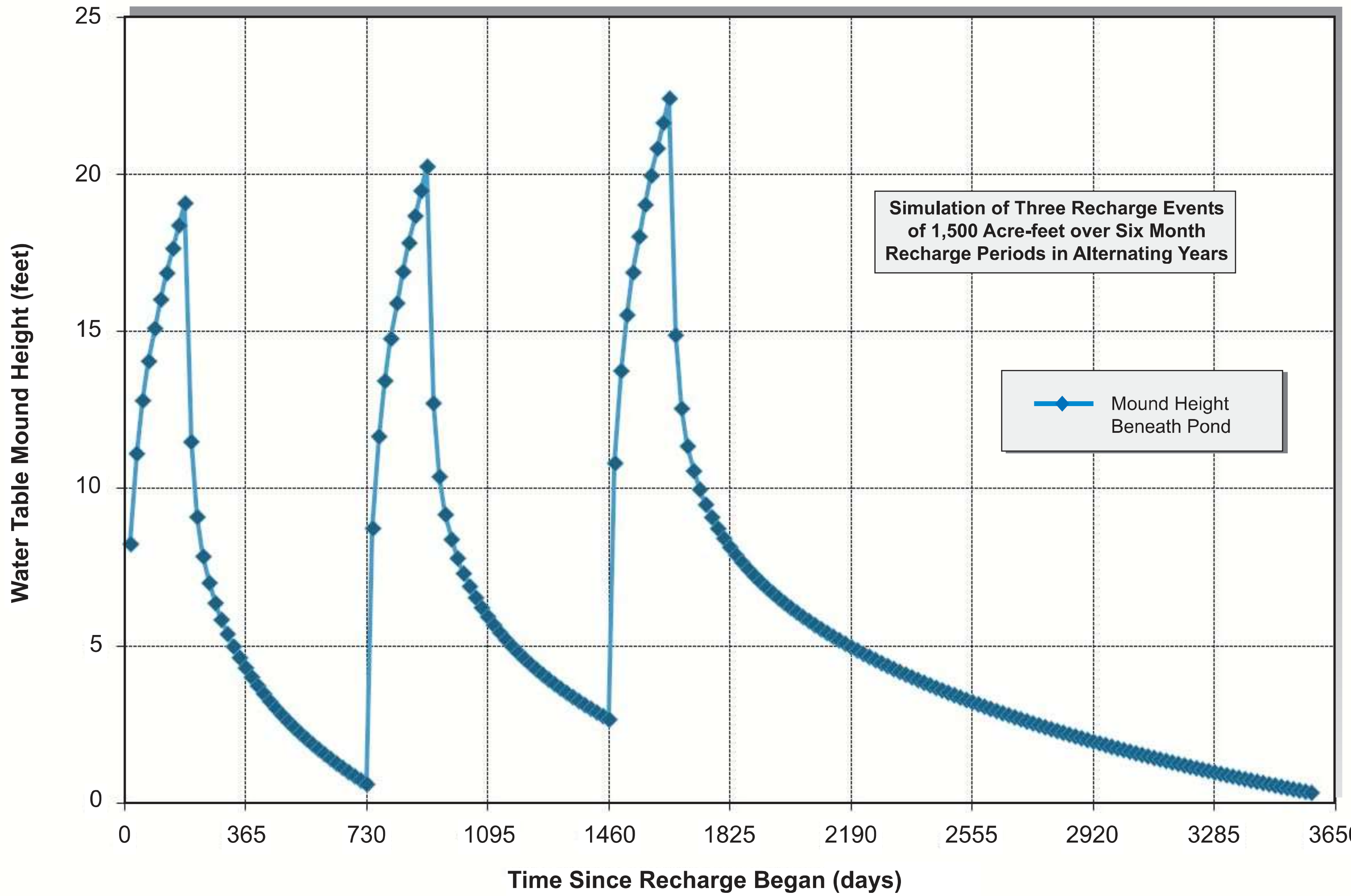
Figure E17
Simulated Water Table Mounding after Recharge of 1,500 AF After 6 months



February 2011	**Figure E18**
TODD ENGINEERS Alameda, California	**Simulated Water Levels over Time from Recharge of 1,500 AF in Alternating Years**

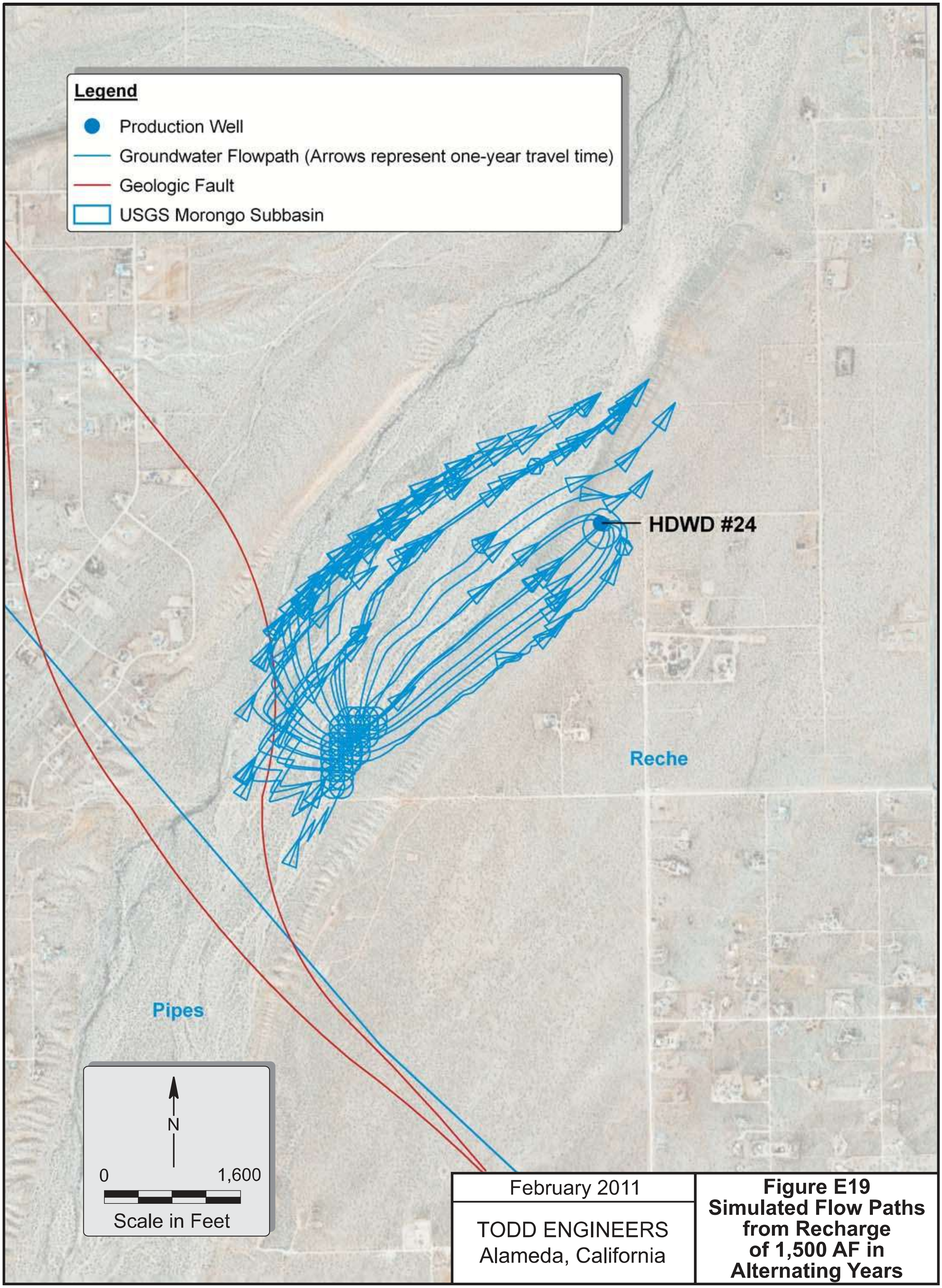
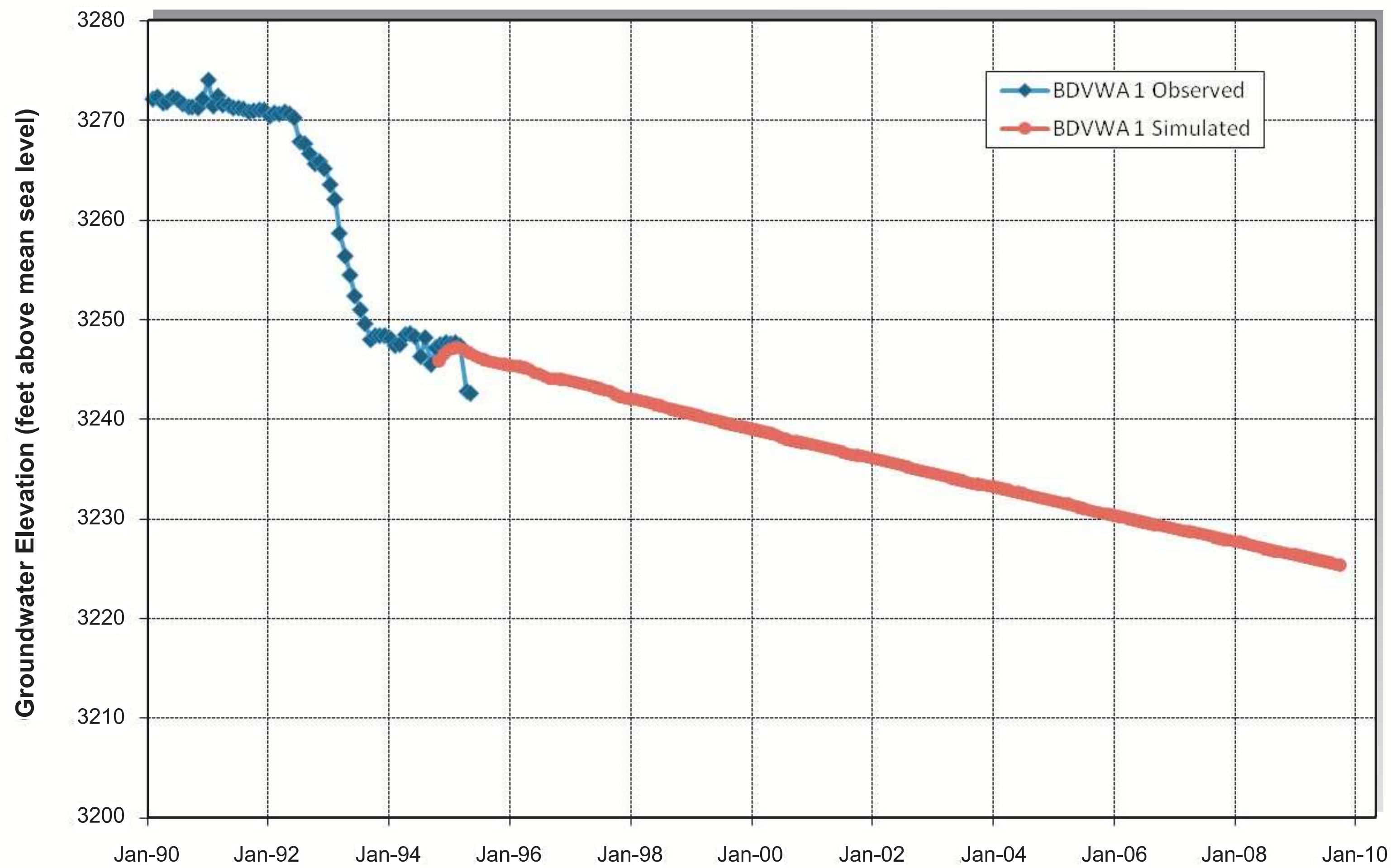
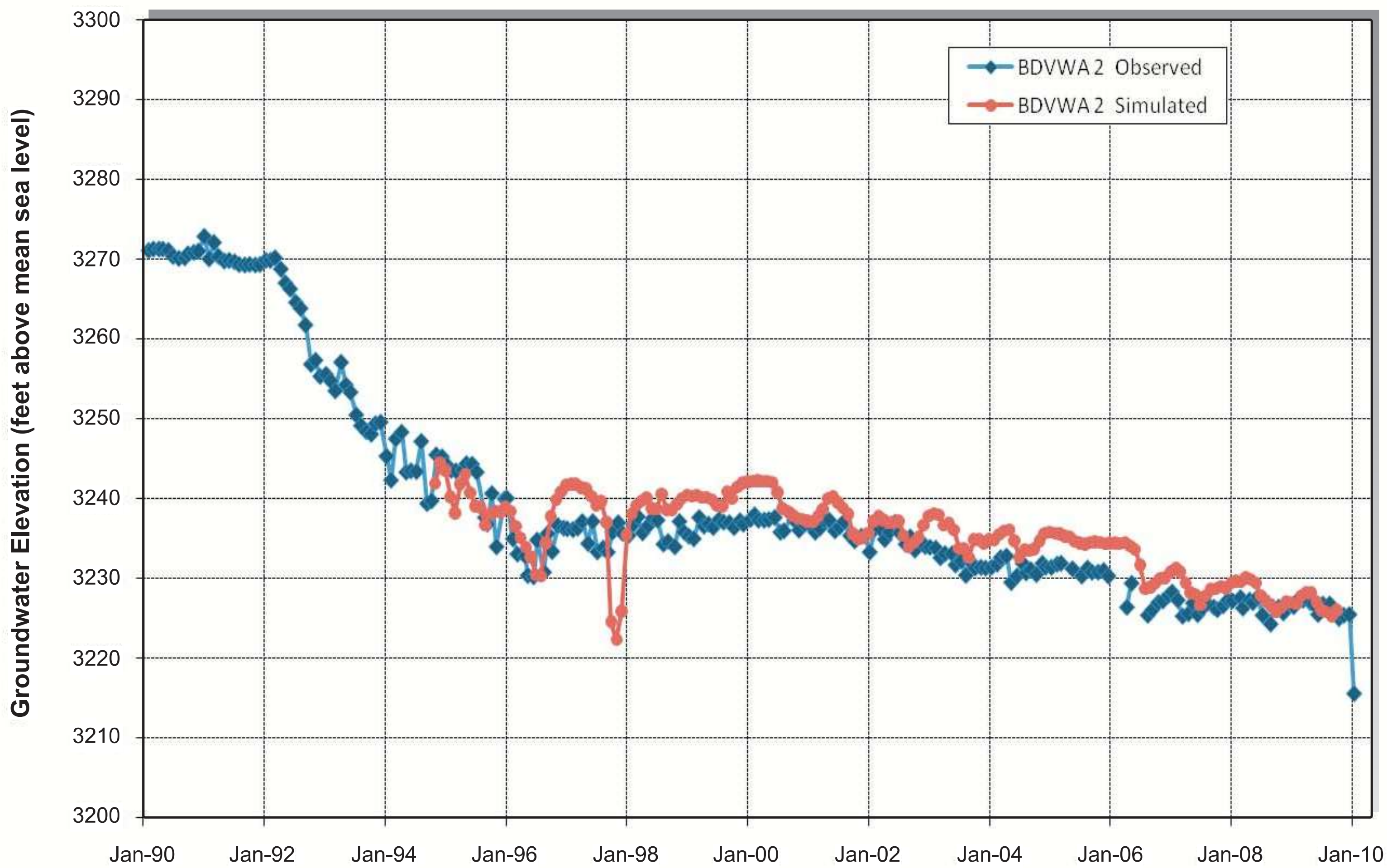


Figure E19
Simulated Flow Paths from Recharge of 1,500 AF in Alternating Years

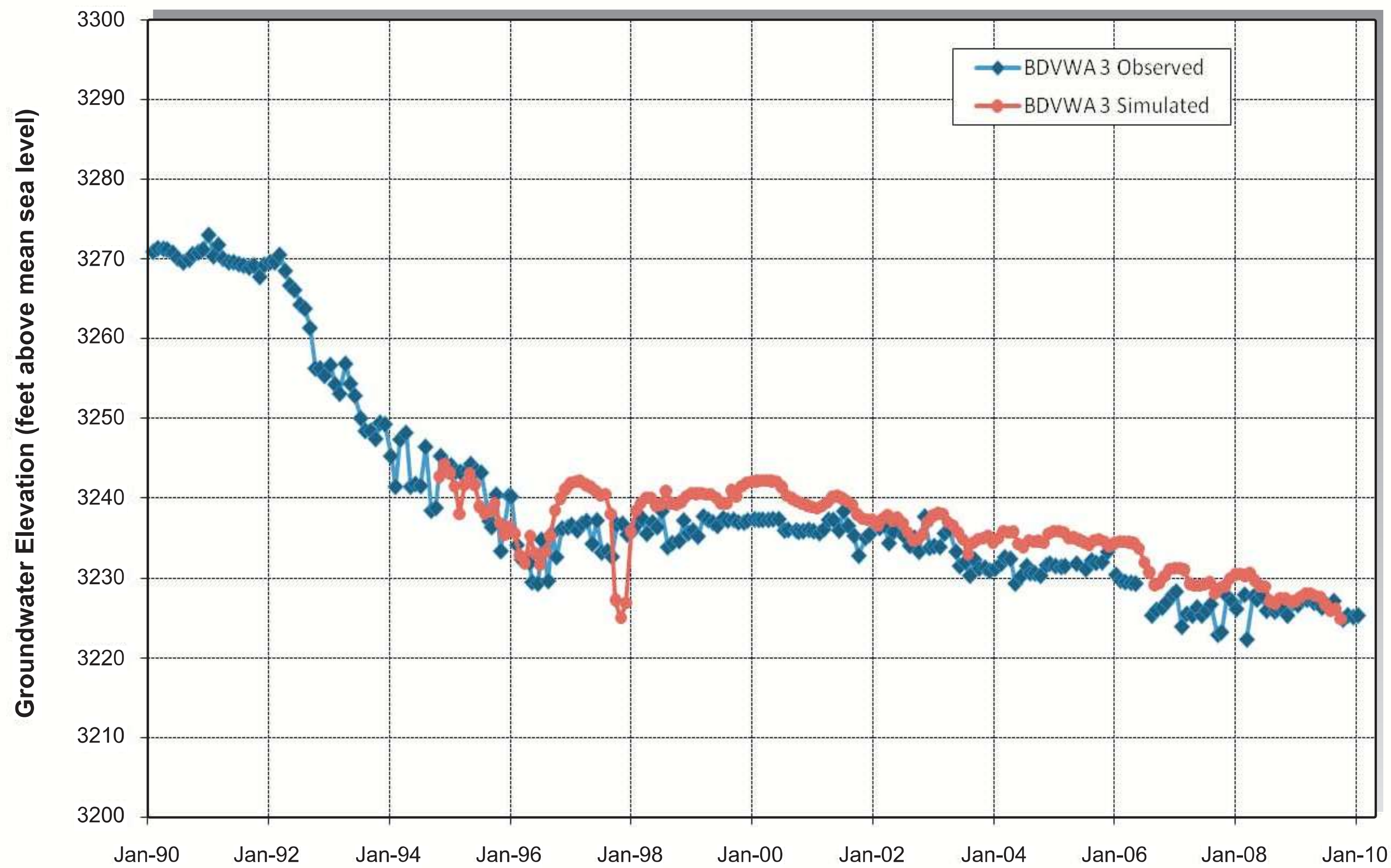
Charts



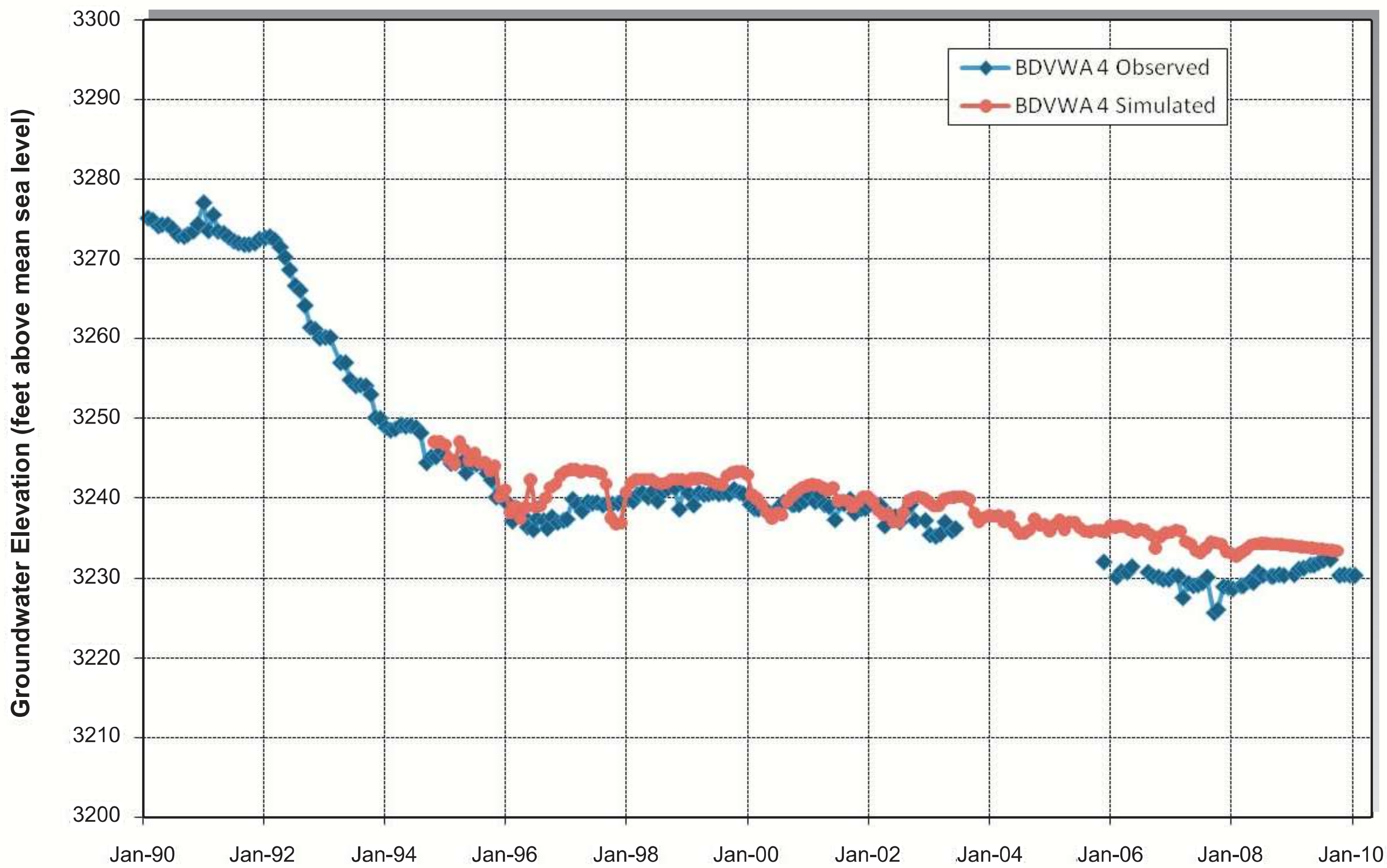
February 2011	**Chart E1**
TODD ENGINEERS Alameda, California	**1994 - 2009 Observed and Simulated Groundwater Elevations, Well BDVWA 1**



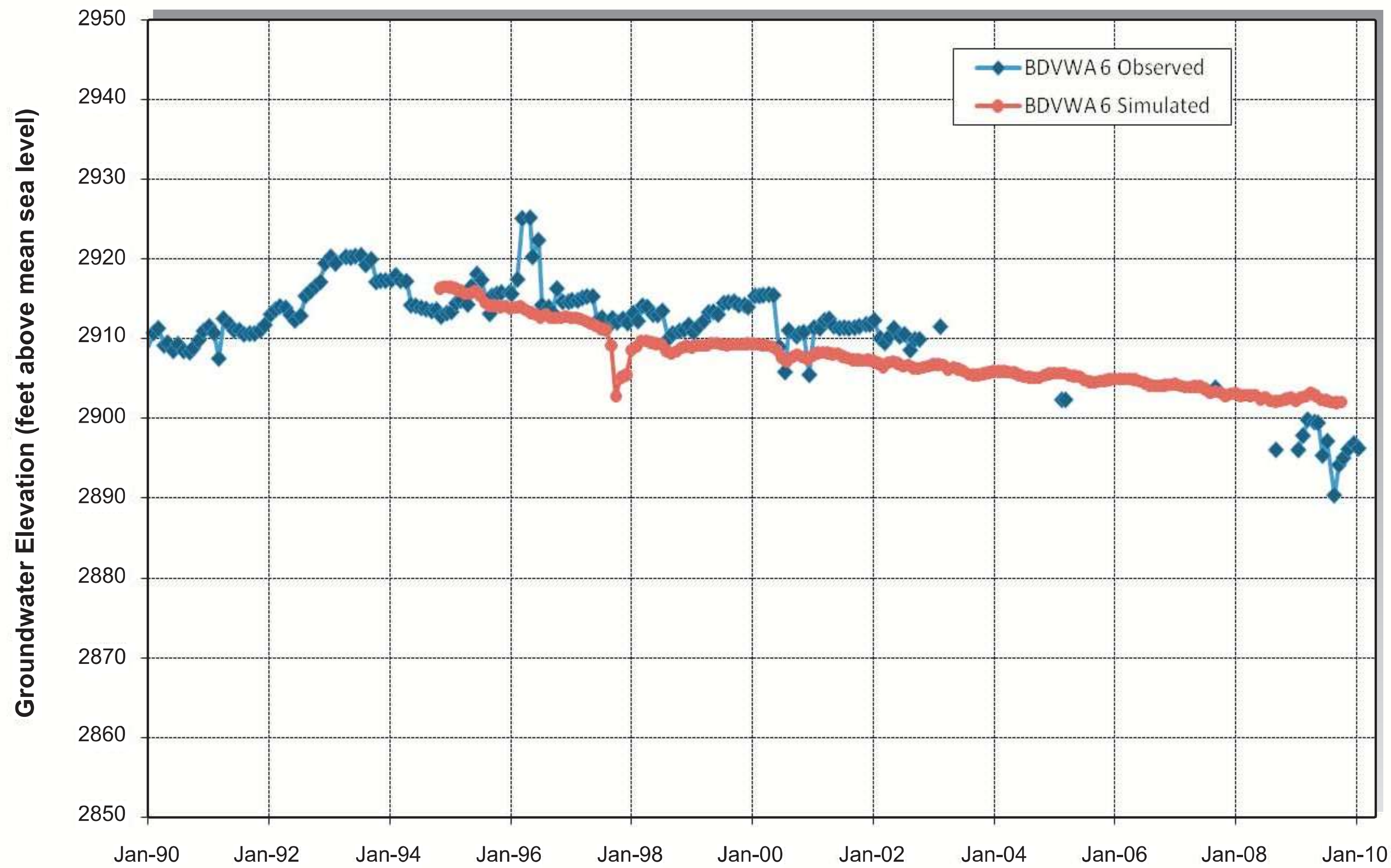
February 2011	Chart E2 1994 - 2009 Observed and Simulated Groundwater Elevations, Well BDVWA 2
TODD ENGINEERS Alameda, California	



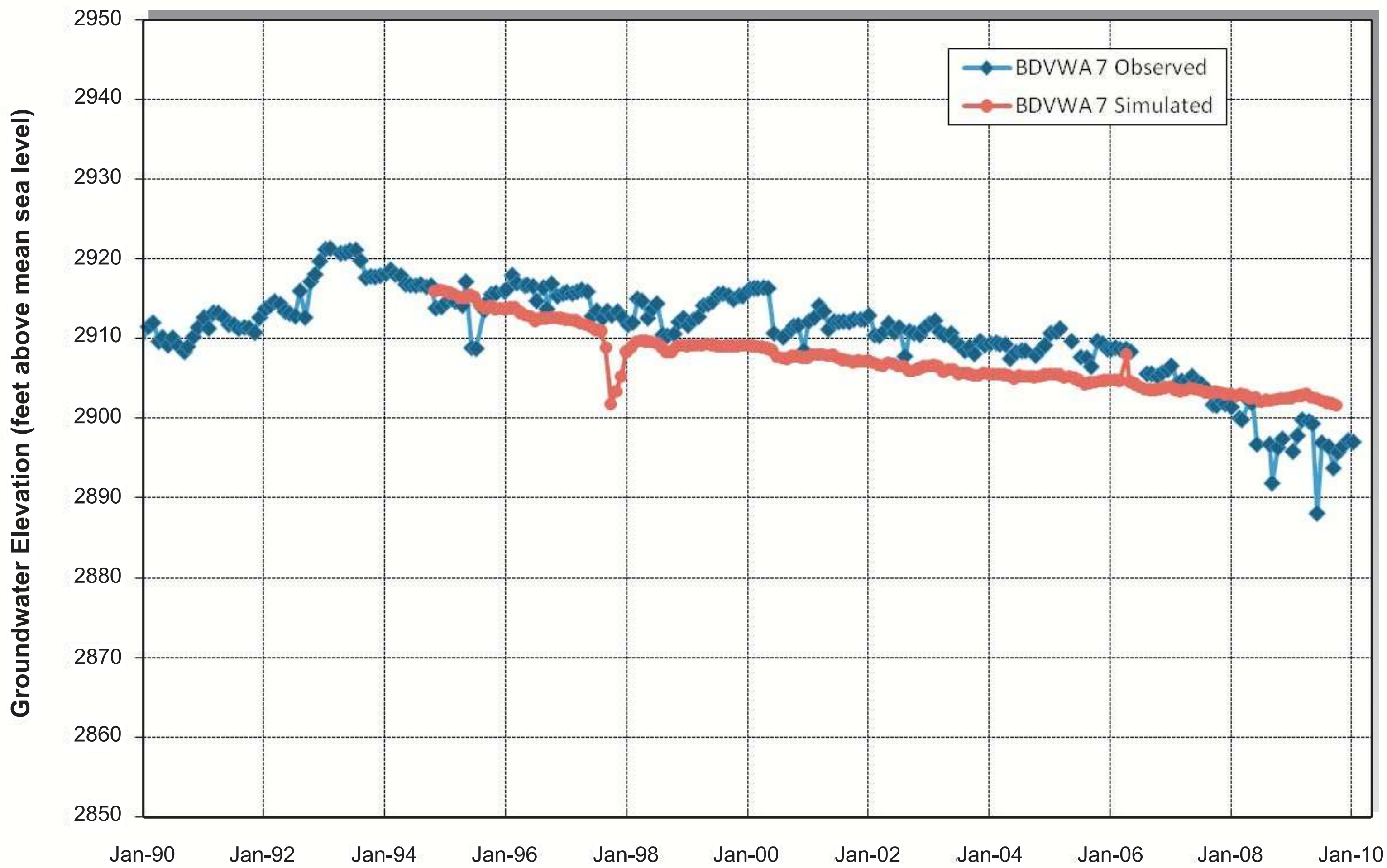
February 2011	Chart E3 1994 - 2009 Observed and Simulated Groundwater Elevations, Well BDVWA 3
TODD ENGINEERS Alameda, California	



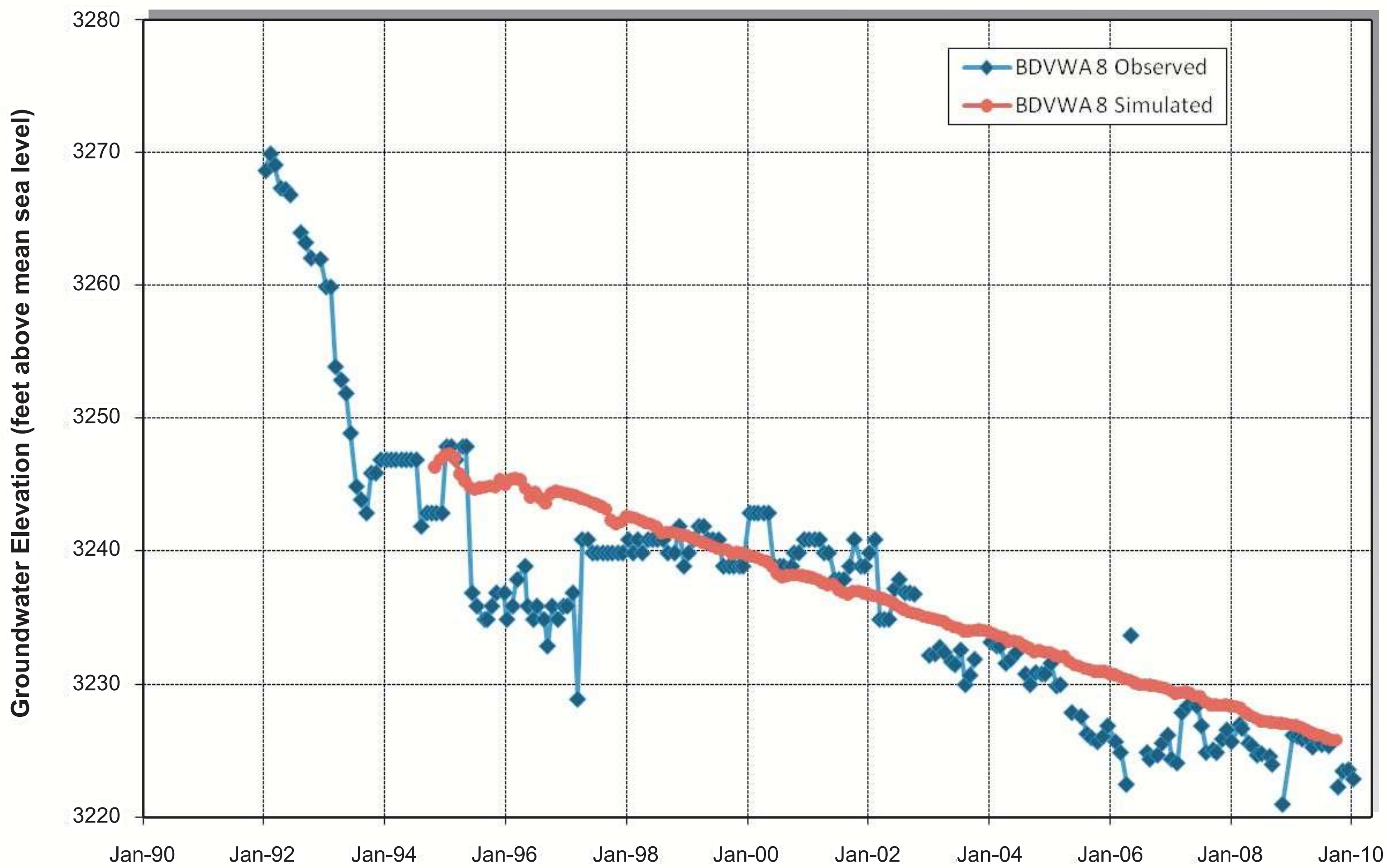
February 2011	**Chart E4**
TODD ENGINEERS Alameda, California	**1994 - 2009 Observed and Simulated Groundwater Elevations, Well BDVWA 4**



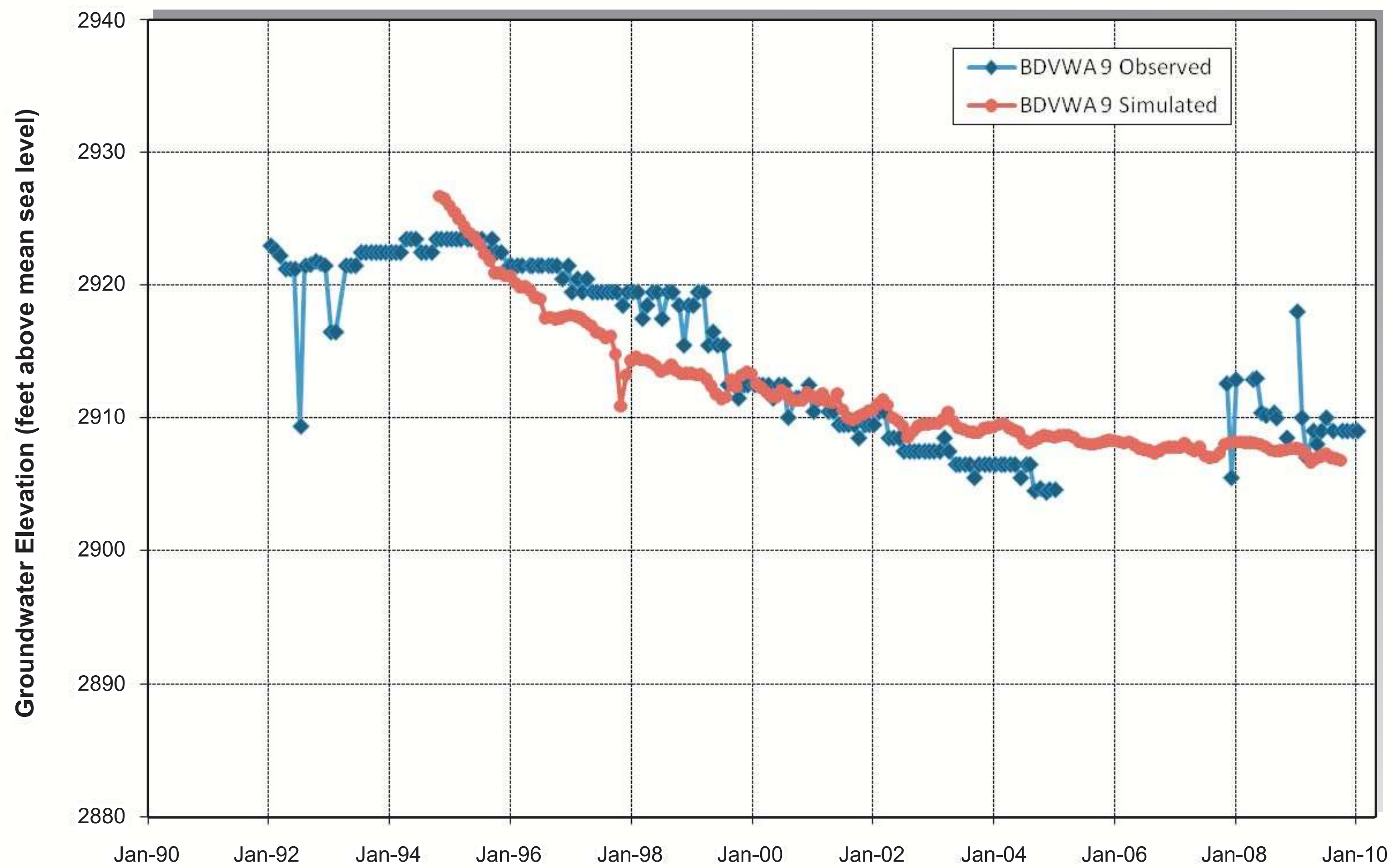
February 2011	**Chart E5**
TODD ENGINEERS Alameda, California	**1994 - 2009 Observed and Simulated Groundwater Elevations, Well BDVWA 6**



February 2011	**Chart E6**
TODD ENGINEERS Alameda, California	**1994 - 2009 Observed and Simulated Groundwater Elevations, Well BDVWA 7**



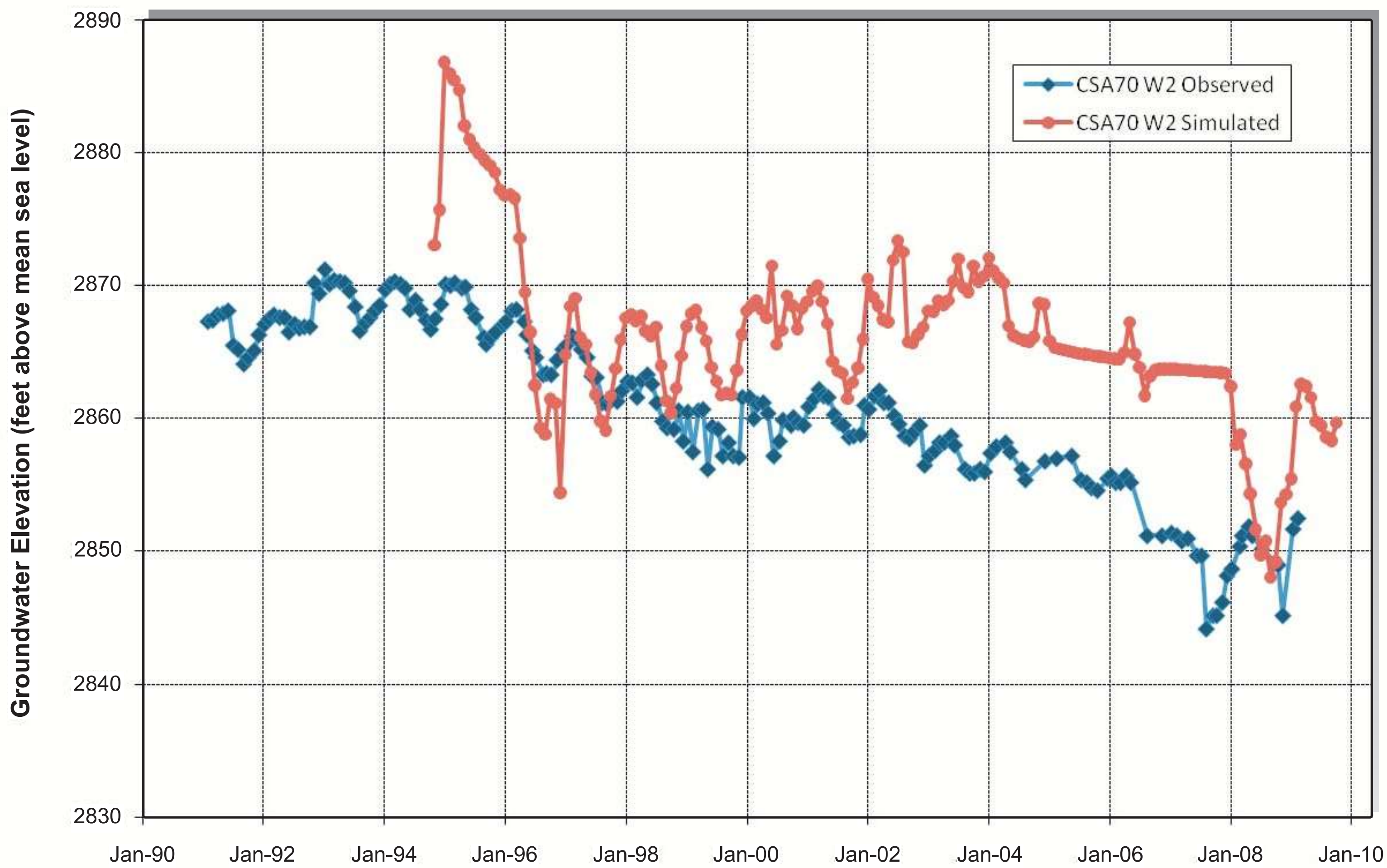
February 2011	**Chart E7**
TODD ENGINEERS Alameda, California	**1994 - 2009 Observed and Simulated Groundwater Elevations, Well BDVWA 8**



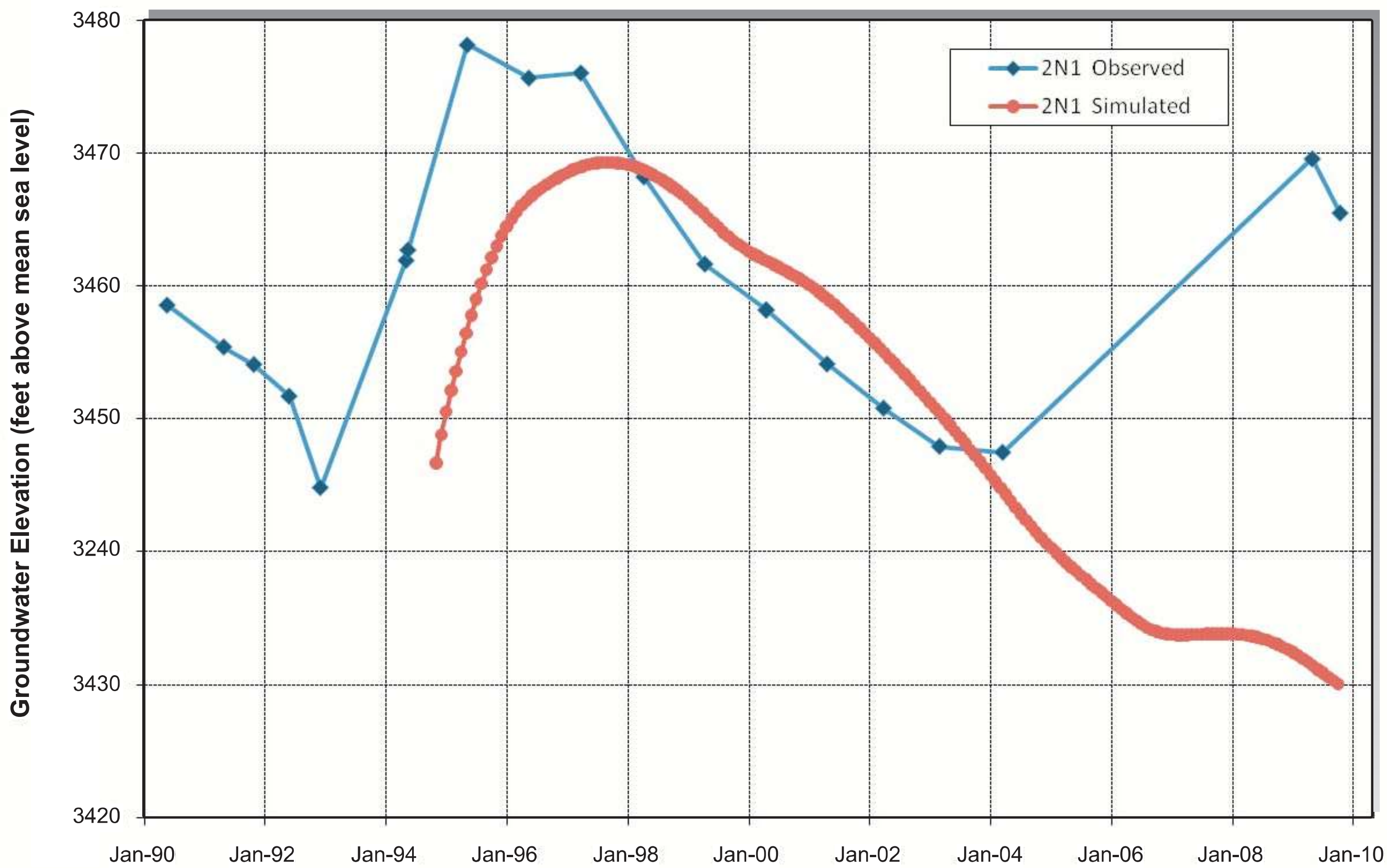
February 2011	**Chart E8**
TODD ENGINEERS Alameda, California	**1994 - 2009 Observed and Simulated Groundwater Elevations, Well BDVWA 9**

February 2011	**Chart E9**
TODD ENGINEERS Alameda, California	**1994 - 2009 Observed and Simulated Groundwater Elevations, Well HDWD 24**

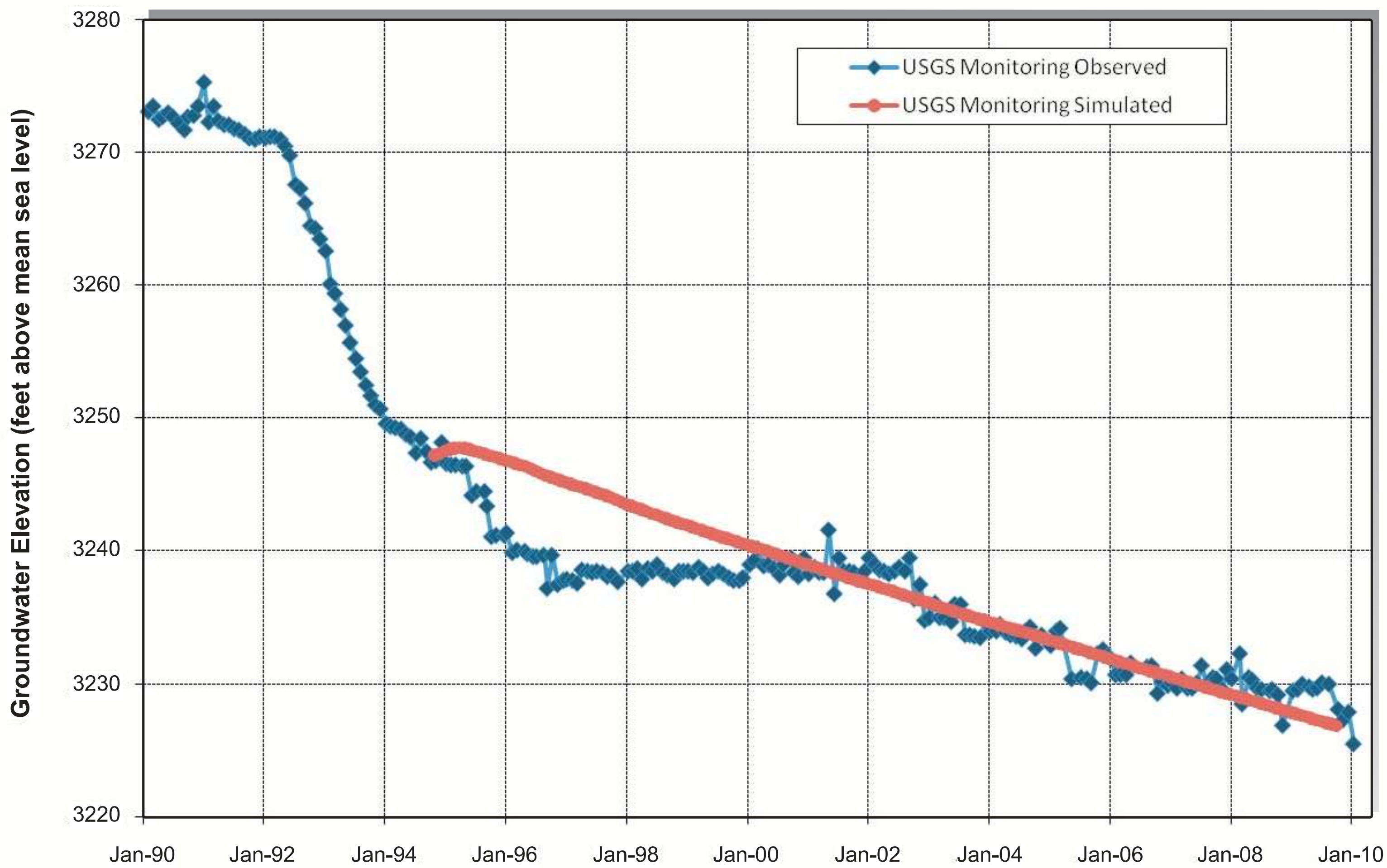
February 2011	**Chart E10** **1994 - 2009 Observed and Simulated Groundwater Elevations, Well CSA 1**
TODD ENGINEERS Alameda, California	



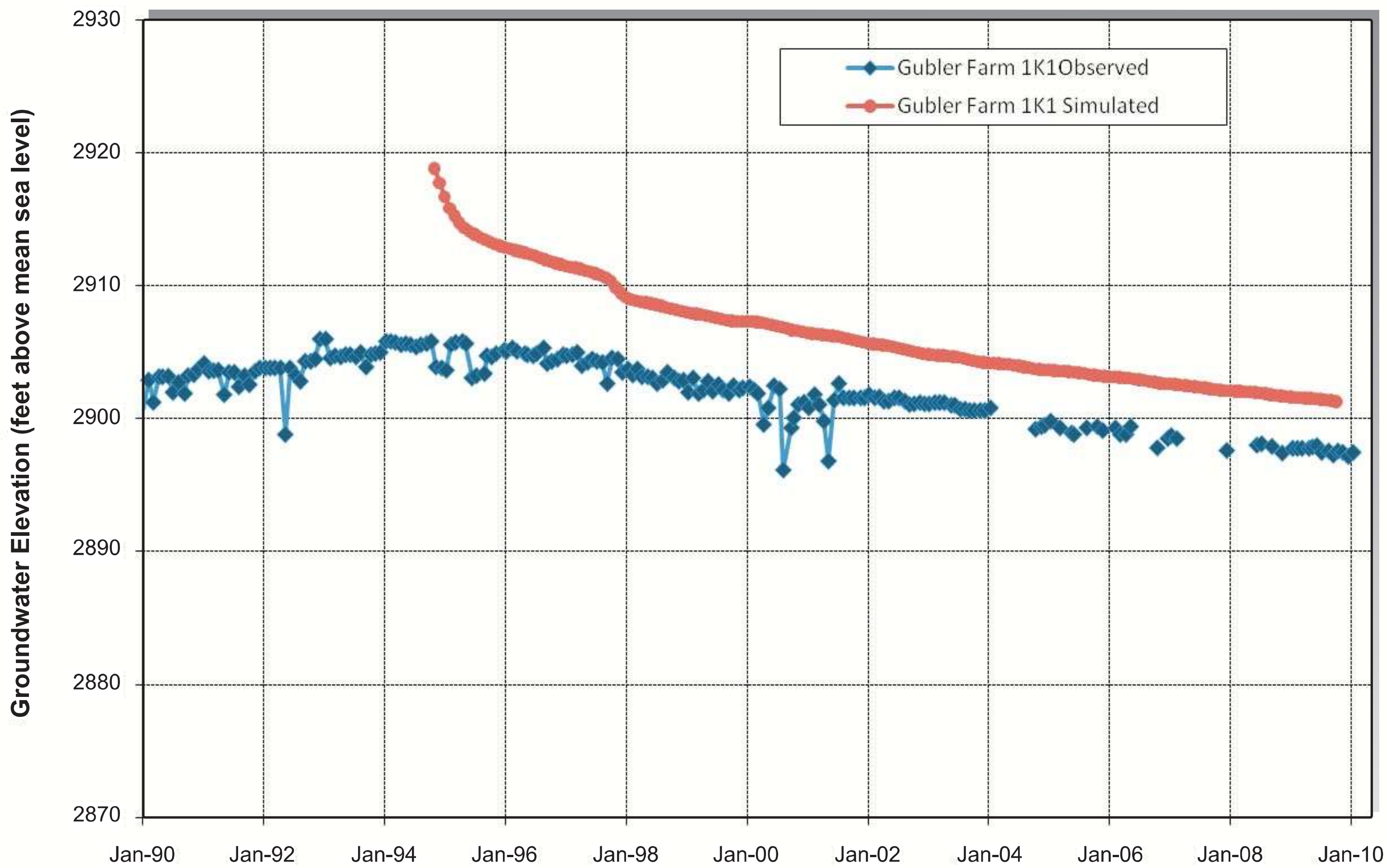
February 2011	**Chart E11** **1994 - 2009 Observed and Simulated Groundwater Elevations, Well CSA 2**
TODD ENGINEERS Alameda, California	



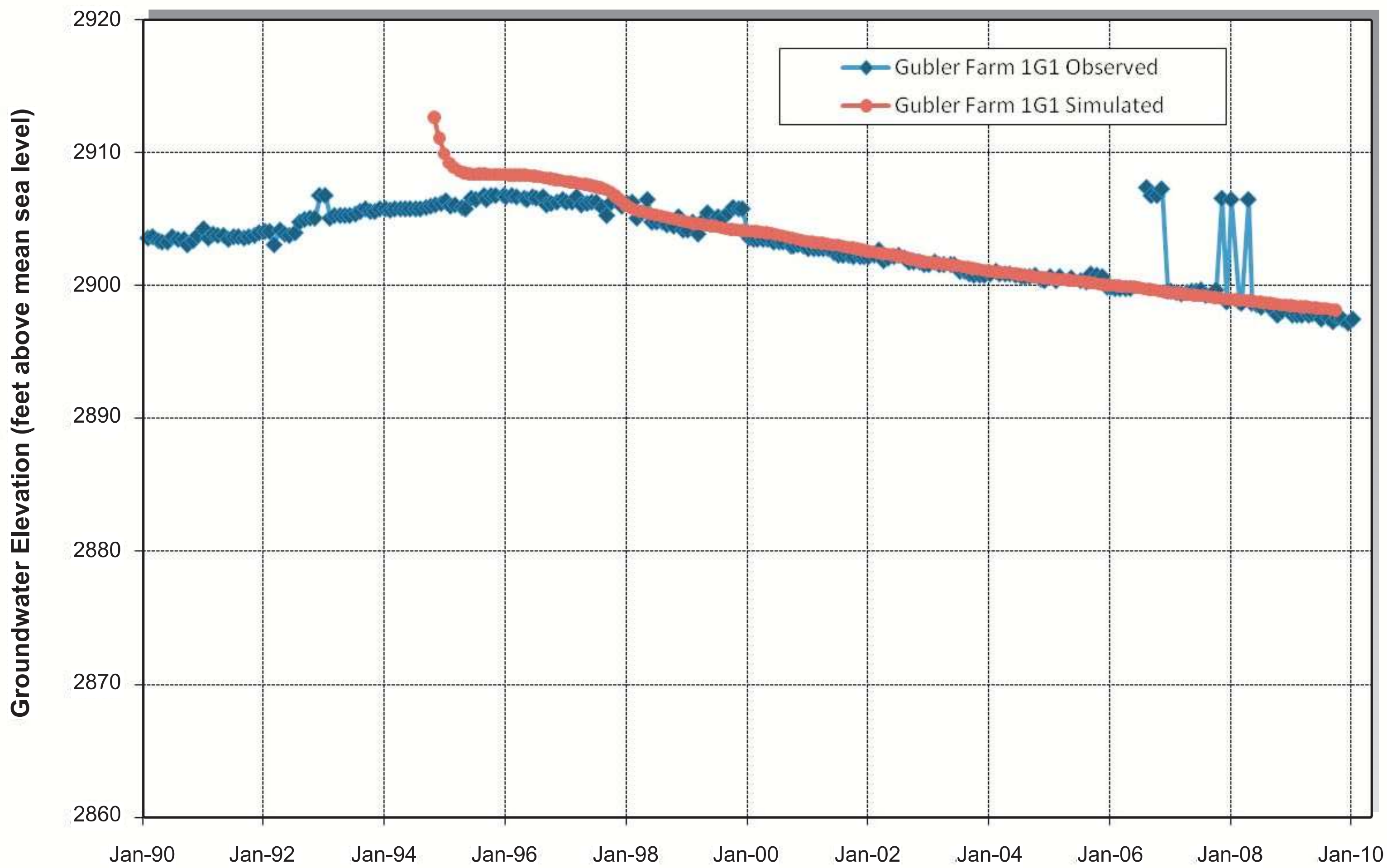
February 2011	**Chart E12**
TODD ENGINEERS Alameda, California	**1994 - 2009 Observed and Simulated Groundwater Elevations, Well 2N1**



February 2011	**Chart E13** **1994 - 2009 Observed and Simulated Groundwater Elevations, Well USGS Monitoring**
TODD ENGINEERS Alameda, California	



February 2011	**Chart E14** **1994 - 2009 Observed and Simulated Groundwater Elevations, Well Gubler 1K1**
TODD ENGINEERS Alameda, California	



February 2011	**Chart E15** **1994 - 2009 Observed and Simulated Groundwater Elevations, Well Gubler 1G1**
TODD ENGINEERS Alameda, California	