Aromas and Purisima Basin Management Technical Study

Santa Cruz Integrated Regional Water Management Planning Grant Task 4

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Prepared for:

Central Water District

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ABBREVIATIONS

Δs	drawdown
AFY	acre-feet per year
BAT	Best Available Technologies
bgs	below ground surface
BPD	basis of design
CO_2	carbon dioxide
CRM	Chemical Reductive Media
CWD	Central Water District
DPH	California Department of Public Health
ft	foot, feet
GIS	geographical information system
GMP	Groundwater Management Plan
gpm	gallons per minute
HRU	hydrologic response unit
IRWM	Integrated Regional Water Management
MAE	mean absolute error
MCL	maximum contaminant level
ME	mean error
mg/L	milligrams per liter
O&M	operation and maintenance
OEHHA	California Office of Environmental Health Hazard Assessment
PRMS	precipitation-runoff modeling system
PVWMA	Pajaro Valley Water Management Agency
Q	flow rate
RCF	Reduction, Coagulation, Filtration
RMSE	root mean squared error
S	storativity
SBA	Strong Base Anion Exchange
SqCWD	Soquel Creek Water District
STD	standard deviation of the errors
t	time
T	transmissivity
TAC	Technical Advisory Committee
TENORM	Technologically-Enhanced, Naturally-Occurring Radioactive Material
USEPA	United States Environmental Protection Agency

USGS ....United States Geological Survey
WBA ....Weak Base Anion Exchange
μg/L ....micrograms per liter

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EXECUTIVE SUMMARY

The Santa Cruz Integrated Regional Water Management (IWRM) region was awarded an IRWM Planning grant by the Department of Water Resources. On behalf of the IRWM region, the Regional Water Management Foundation (RWMF) served as grantee and worked in conjunction with local IRWM partner agencies (sub-grantees) to fund technical studies to inform future water resource management in the region. Central Water District (CWD), a sub-grantee, conducted a study of the redistribution of groundwater pumping between the Aromas and Purisima areas. Currently, CWD pumps approximately 96% of its water supply from the Aromas area and 4% from the Purisima area. However, groundwater pumped by CWD from the Aromas area has elevated levels of hexavalent chromium (chromium VI), a drinking water contaminant regulated under state and federal drinking water standards. In addition, the Aromas area is subject to seawater intrusion resulting from groundwater overdraft. This study evaluates the potential for CWD to redistribute groundwater pumping from the Aromas to inland portions of the Purisima in order to balance pumping and avoid chromium VI treatment costs.

CWD's existing wells in the Purisima area on Cox Road were evaluated for their ability to increase pumping. Tests evaluated well condition, well production performance, dry season well capacity, aquifer response to pumping and properties, and vertical flow and water quality profiles. Based on the age of the wells and documented degradation of well performance, it is recommended that existing wells be taken out of production and replaced with a single modern-designed production well. A new well would likely have a discharge capacity of 300 to 400 gallons per minute (gpm) and it is estimated that dry season production of approximately 160 acre-feet can be sustained. Based on groundwater modeling, the top of the screen should be at least 260 feet below ground surface and the well should be drilled 660 feet deep to screen the full depth of the Purisima F unit.

The potential to modify the Rob Roy #12 well to improve water quality was evaluated with a vertical flow and water quality profile. The results showed that the well's upper screen produces approximately 70% of the chromium VI mass but only 25% of the water flow. This indicates that modifying the well so that water is not produced from the upper screen could reduce chromium VI concentrations while maintaining the majority of flow. However, modifying Rob Roy #12 well is likely unnecessary as water quality samples indicate chromium

VI concentrations at the well are below the draft drinking water standard issued by the California Department of Public (DPH) Health in 2013.

In order to evaluate the sustainable yield for redistributing pumping to the Purisima, CWD's groundwater model was updated. The model was originally developed as a steady state model for use in CWD's Drinking Water Source Assessments (Johnson, 2009). The model was updated to simulated transient conditions from 1984-2009 for calibration to available groundwater level data and simulation of long-term groundwater level changes due to shifting pumping. Model inputs including pumping for private wells and some small water systems and return flow recharge were based on a spatial analysis of water use. Rainfall recharge and flow from upgradient watersheds were based on results from a watershed model using the Precipitation-Runoff Modeling System (HydroMetrics WRI, 2011). The western and eastern boundary conditions were based on groundwater level data from Soquel Creek Water District and Pajaro Valley Water Management Agency.

The updated groundwater model was calibrated to groundwater levels for 1984-2009 so that the model can be defensibly used to evaluate redistribution of pumping from the Aromas area to the Purisima area. Model calibration consisted of modifying the distribution and magnitude of horizontal hydraulic conductivity, vertical hydraulic conductivity, specific storage, and specific yield values. Values for seabed outcrop conductance were also modified for each layer. The model is well calibrated to groundwater levels at CWD's Rob Roy and Cox well fields as well as other wells in the Valencia Creek subbasin where the effects of shifting pumping is likely to be the greatest. The model calibration is also adequate for evaluating effects of shifting pumping in the Rio del Mar area, including coastal monitoring wells SC-A1 and SC-A8. However, the model should not be used to evaluate groundwater management in the Seascape area and areas to the south and east of the Seascape area as the model does not adequately simulate observed groundwater level trends in these areas.

The updated groundwater model was used to simulate three groundwater management scenarios for comparison with a baseline simulation:

Baseline Simulation: Current conditions projected into the future.
Scenario 1: Shift Rob Roy pumping to new Cox well to meet CWD demand
Scenario 2: Modify Rob Roy #12 to improve water quality
Scenario 3: Maximize Rob Roy and Cox pumping

Simulation results showed that the strategy to redistribute pumping to a new Cox well is within the sustainable yield of the Purisima Formation that supplies the Cox well field. Shifting pumping from the Aromas area to the Purisima area will also reduce system chromium VI concentrations while increasing CWD's reliability by diversifying its supply. Finally, CWD's increased inland pumping capacity potentially facilitates regional basin management if water in excess of CWD's demand can be used to help non-CWD pumpers reduce pumping closer to the coast to address seawater intrusion risk. The primary environmental effect of the strategy that may need further evaluation is the effect of predicted lower groundwater levels on the supply of private wells near the Cox well field.

Cost estimates for constructing and developing the new well and destroying two of the existing wells are provided. The cost estimates include preparation of a preliminary design report and technical specifications, the drilling contractor, and hydrogeologic oversight of the drilling contractor. The total estimated cost is approximately $700,000.

Cost estimates for modifying Rob Roy #12 to improve water quality are also provided. Modification strategies including lowering the pump and installing an inflatable packer to reduce flow from high concentration depth intervals are evaluated. Estimated costs range from $40,000 to $65,000. Modification of Rob Roy #12 is unlikely because chromium VI concentrations at the well are lower than the draft drinking water standard.

Conceptual design and cost estimates for water treatment at the Cox well field were prepared. Groundwater produced by the Cox wells is high in iron and manganese requiring treatment to meet drinking water standards. Three treatment alternatives were evaluated:

1. Single horizontal filter vessel with three cells
2. Three vertical pressure filter vessels
3. Package system in a skid mounted installation

The package system is the recommended alternative. The system is relatively easy to install and operate and is commonly used for treatment plants with low flow capacity such as provided by a new Cox well. The construction cost is estimated at approximately $2 million and the annual operations and maintenance costs are estimated at $140,000.

Treatment options for the removal of chromium VI were reviewed. Several treatment options are potentially available. Selection of a treatment process is complicated and depends on a number of site-specific factors including water quality, well capacity, footprint limitations, and cost. Pilot-scale testing conducted at Soquel Creek Water District's San Andreas well in spring 2013 showed highly promising results for the strong based anion exchange technology (Water Research Foundation Project #4488 *Hexavalent Chromium Treatment with Strong Base Anion Exchange and Chemical Reductive Media*).

In conclusion, the strategy of shifting pumping from the Aromas area to Purisima area is beneficial for CWD and regional basin management. Replacing the aging wells at the Cox well field with a new well and treating the groundwater for iron and manganese will improve CWD's system reliability and water quality. Increasing inland pumping capacity has the potential to facilitate regional partnerships that help non-CWD pumpers reduce pumping near the coast to reduce seawater intrusion risk. The estimated capital cost of the well replacement and treatment system installation is $2.7 million.

SECTION 1
BACKGROUND AND SCOPE

The Santa Cruz Integrated Regional Water Management (IRWM) Region was awarded a planning grant to complete key technical studies to guide sustainable management of local water resources. The grant was awarded by the California Department of Water Resources' (DWR) through the IRWM Program funded by Proposition 84. Task 4 of the planning grant was for Central Water District (CWD) to perform a planning study to evaluate redistribution of its groundwater pumping between the Aromas and Purisima areas.

Currently, CWD pumps approximately 96% of its water supply from its Rob Roy well field in the Aromas area and 4% from its Cox well field in the Purisima area further inland. However, groundwater pumped by CWD from the Aromas area has elevated levels of chromium VI. In addition, the Aromas area is subject to seawater intrusion resulting from overdraft. Redistributing pumping from the Aromas to inland portions of the Purisima could allow CWD to avoid chromium VI treatment costs and help address the regional seawater intrusion risk.

The Purisima Formation that supplies the Cox well field has high iron and manganese concentrations exceeding the secondary drinking water standards. Increasing pumping from the inland Cox well field requires treatment for iron and manganese.

The potential strategy of redistributing pumping from the Aromas and Purisima while adding the capability to treat Purisima groundwater is evaluated based on the three groundwater management goals in the *Groundwater Management Plan - 2007: Soquel Aptos Area* (SqCWD and CWD, 2007) co-authored by CWD with neighboring Soquel Creek Water District:

1. Ensure water supply reliability for current and future beneficial uses
2. Maintain water quality to meet current and future beneficial uses
3. Prevent adverse environmental impacts.

To accomplish this evaluation, the following tasks were proposed for grant funding:

Task 4.1 Evaluate the sustainable yield of the local Purisima Formation
Task 4.2 Evaluate the condition and capacity of the Cox Road wells

Task 4.3 Prepare well rehabilitation/re-drill cost estimate
Task 4.4 Evaluate type and siting of a water treatment plant
Task 4.5 Groundwater management analysis
Task 4.6 Final report

This final report is a compilation of technical memorandums completed by HydroMetrics Water Resources Inc. and Kennedy Jenks Consultants summarizing work on these tasks. The technical memorandums were reviewed by a Technical Advisory Committee, which also met several times to discuss the evaluations.

Section 2 of the report includes the evaluation of the condition and capacity of the Cox Road wells for Task 4.2.

Section 3 of the report includes the recommendation for the Cox Road wells as part of Task 4.3.

Section 4 of the report includes evaluation of possible modification of Rob Roy #12 to improve water quality, scope that was added to Task 4.2.

Section 5 of the report documents the update to CWD's groundwater model for evaluating local Purisima Formation sustainable yield for Task 4.1.

Section 6 of the report documents the calibration of CWD's groundwater model for evaluating local Purisima Formation sustainable yield for Task 4.1.

Section 7 of the report documents the use of CWD's updated groundwater model to analyze groundwater management scenarios for Task 4.5.

Section 8 of the report includes cost estimates for implementing the recommendation of a replacement well at the Cox well field and modifying the Rob Roy #12 well as part of Task 4.3.

Section 9 of the report includes a conceptual design and cost estimate for a treatment plant at the Cox well field to address high iron and manganese concentrations. Kennedy-Jenks Consultants developed the design and cost estimate under Task 4.4.

Section 10 of the report reviews chromium VI treatment technologies. Kennedy-Jenks Consultants performed the review under scope added to Task 4.

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SECTION 2
EVALUATE THE CONDITION AND CAPACITY OF THE COX ROAD WELLS (TASK 4.2)

Central Water District's (CWD) wells were evaluated for their ability to meet CWD's customers' beneficial uses while improving groundwater management. As part of grant Task 4.2, tests at the Cox #3 and Cox #5 wells were conducted to evaluate well condition, well production performance, dry season well capacity, aquifer response to pumping and properties, and vertical flow and water quality profiles. The results from analyzing data collected from these tests (Task 4.2) were used to develop recommendations for the wells with an emphasis on the Cox well field, where an increase of production is the most straightforward way to implement the strategy of shifting pumping away from areas where hexavalent chromium naturally occurs (Section 3, Task 4.3). The aquifer properties estimated from the test data are also used in calibration of the groundwater model (Section 6, Task 4.1).

This report section documents the field test activities, data collected, and data analyses. The section was distributed to the Technical Advisory Committee (TAC) for review as part of a draft technical memorandum on January 9, 2013.

2.1 Relative Importance of Data for Cox Well Recommendation

This report is organized based on the types of information collected and analyses performed at Cox wells for Task 4.2. The order of the discussion is based on importance of the data and analyses to the well recommendations. The types of information are discussed in the following order:

1. Production well videos of Cox #3 and #5 used to assess condition of wells;
2. Drawdown data from Cox #3 aquifer test used to quantify performance of well;
3. Groundwater levels at observation wells during Cox #3 aquifer test used to estimate aquifer properties;
4. Drawdown and recovery data from Cox #3 aquifer test used to estimate well capacity over a dry season;

5. Flow and water quality profile of Cox #5 used to evaluate potential for well modification or new well design to address water quality issues;
6. Groundwater levels at private wells during Cox #5 well profile used to evaluate possible well interference; and
7. Other data collected but not used in the technical study.

An electronic copy of data collected is provided on a compact disk as Appendix A.

2.2 SUMMARY OF FIELD ACTIVITIES

Details of the field activities are described in the appropriate subsection listed above, but the following is a chronological summary of field activities performed for Task 4.2:

- On May 11, 2012, BESST Inc. performed chain-access surveys at Cox #3, Cox #5, and Rob Roy #12 to assess access for miniature camera and well profiling tools.
- On May 11, 2012, BESST Inc. conducted video survey of Cox #5 using miniature camera.
- On May 15-16, 2012, BESST Inc. conducted flow and water quality profile of Rob Roy #12.
- On the week of May 21, 2012, Maggiora Bros., Inc. pulled the pumps from Cox #3 and Cox #5, bailed the wells, and brushed the well screens.
- On May 29, 2012, Newman Well Surveys conducted video surveys of Cox #3 and Cox #5.
- Following the well surveys, Maggiora Bros., Inc. reinstalled the pumps in Cox #3 and Cox #5 with new drop pipe for Cox #5.
- On June 5-6, 2012, BESST Inc. conducted flow and water quality profile of Cox #5.
- On September 12, 2012, HydroMetrics WRI conducted aquifer test at Cox #3.

2.3 PRODUCTION WELL VIDEOS OF COX #3 AND #5

2.3.1 CHAIN ACCESS SURVEY

BESST, Inc. ran preliminary access surveys to make sure that the miniature video camera and well profiling tools would not get stuck in the well. The survey consists of lowering a dummy camera and a string of sausage weights on a chain down the well, to make sure that the well is open and their tools will not get stuck in the pump or hung up on any other unforeseen obstructions.

On May 11, 2012 chain access surveys were completed at Cox #3 and Cox #5. The interval between 195-205 feet below ground surface (bgs) was tight at Cox #3; a collar might be blocking access for video and profiling equipment. The second attempt at this well failed in the same place as the first attempt. Based on this, a video survey of Cox #3 was not run with the miniature camera. A chain access survey on Cox #5 found that the well had adequate access for the miniature video camera and profiling tools.

2.3.2 VIDEO OF COX #5 USING MINIATURE CAMERA

A video log of Cox #5 using BESST's miniature video camera was run on May 11, 2012. This survey revealed the well to be extremely encrusted/fouled, visibility was poor, and the camera did not get all the way to bottom as it was not able to advance below 233 feet at the approximate location of the pump and the constriction of well diameter from 12 to 8 inches. However, the video could not determine whether the well was sanded up to that point or the constriction prevented further access.

2.3.3 PUMP REMOVAL AND CLEANING

HydroMetrics WRI recommended removing the pumps and cleaning the wells at both Cox #3 and Cox #5. This would allow video surveys of the open holes to be performed as the miniature camera did not have access to Cox #3 and it was not clear why the video of Cox #5 could not advance below 233'. Based on a 1996 video of Cox #3 and the May 11, 2012 video of Cox #5 showing encrustation in the well, it was recommended that the wells be brushed. Bailing would remove any sediment accumulated at the bottom.

Maggiora Bros, Inc. removed the pumps from Cox #3 and Cox #5 and cleaned the wells the week of May 21, 2012. Bailing of the Cox #5 well showed only minimal accumulation of fill at the total depth of 260′, indicating that the progress of the miniature camera was limited by the pump and/or well constriction and not accumulation of fill.

2.3.4 VIDEO OF COX #3 IN OPEN HOLE

The video survey of Cox #3 was performed by Newman Well Surveys on May 29, 2012. A summary sheet of this most recent video is included in Appendix B. The video revealed no casing problems. The vertical milled slot perforations were moderately plugged in the upper interval between 135 and 230 feet depth; below this depth and to bottom the perforations were mostly open. The bottom in the most recent video was encountered at a depth of 282 feet.

This survey is compared to a video from 1996 which shows several locations in the well (around 175 and 255 feet depth) with holes, enlarged perforations, or both, while, in other locations, the perforations look good. In some locations the perforations were obscured by encrustation, but were clearly visible at other locations. Where visible, gravel pack (or coarse material) could be seen bridging the slots. The bottom of the well was encountered at a depth of 291 feet. At the time of the 1996 video the well was considered to be in fair to poor condition.

The 1996 video advanced to 291 feet, which suggests that about 10 feet of fill has accumulated since 1996. Much of the present fill may be residual material not completely bailed out after brushing. With this exception, the comparison of the 1996 and 2012 videos reveals little additional degradation in condition.

2.3.5 VIDEO OF COX #5 IN OPEN HOLE

The video survey of Cox #5 was performed by Newman Well Surveys on May 29, 2012. This most recent video summary sheet is also included in Appendix B.

The visibility in the May 29, 2012 video, performed after brushing and bailing and flushing, was significantly improved from the May 11, 2012 video with the miniature camera. No apparent structural casing problems were noted. The video revealed that in the upper screen interval (195 to 210 feet depth) the screen was mostly plugged on one side of the casing and slightly plugged on the other. This likely results from the pump being set against the side of the casing. The uppermost and lowermost few feet of the lower screen were plugged; otherwise, the lower screen was relatively clean. The plugging on the top and bottom is likely galvanically-driven encrustation due to dissimilar metals – the suspected carbon steel blank and cellar juxtaposed with the stainless steel screen. The video survey bottomed at a depth of 258.5 feet, suggesting 3.5 feet of fill.

2.4 COX #3 WELL PERFORMANCE

2.4.1 COX #3 AQUIFER TEST DETAILS

On September 12, 2012, a constant-rate aquifer test was conducted at Cox #3, where the drawdown and recovery in the pumping well and observation wells near the pumping well were measured. The pump was turned on at 0916 hours and run at a constant rate of approximately 47.9 gallons per minute (gpm) until 1516 hours.

Drawdown was measured in the pumping well and also monitored at Cox #2 and Cox #5. Data were collected using the District's SCADA system for groundwater levels at Cox #3, a Geokon transducer in Cox #2, a sonic sounder, and a hand sounder. The transducer in Cox #3 was vented, and therefore directly provides the water level. Data from the non-vented Geokon transducer in Cox #2 represent absolute pressure that had to be barometrically compensated based on barometric pressure data from a Geokon barometer stored in the Cox #2 well house.

One hour of recovery data was manually collected for the pumping well and each monitoring well. The rest of the recovery period was recorded by the SCADA system at Cox #3 and logged by the Geokon transducer at Cox #2.

2.4.2 COX #3 SPECIFIC CAPACITY

24-hour specific capacity is a key measure of well performance. The drawdown data from the pumping well are extrapolated from the 6 hour test to 24 hours. This extrapolated drawdown is divided by flow rate to calculate 24 hour specific capacity. Figure 2-1 shows the drawdown data from Cox #3 well versus time on a semi-log plot. The extrapolated drawdown at 24 hours is 35.8 feet. For the flow rate of 47.9 gpm, the 24 hour specific capacity is 1.34 gpm/foot.

Prior performance data for Cox #3 are sparse. The California Department of Public Health (DPH) form dated 1970 documents a discharge rate of 300 gpm with static water level of 113 feet and pumping level of 215 feet for a specific capacity of about 3 gpm/foot. A hand notation on the well diagram lists a specific capacity of 4.31 gpm/foot in 1983 at 230 gpm. Therefore, specific capacity and well performance has declined significantly over the life of the well.

2.4.3 Cox #3 Well Efficiency

Well efficiency is a measurement of energy losses as groundwater flows through the well filter pack and the screen to the well bore, which causes drawdown in the well to be greater than drawdown in the surrounding aquifer. Drawdown in the aquifer just outside the well cannot be measured, but drawdowns in two or more observation wells at different distances from the pumping well can be extrapolated to estimate the drawdown just outside the well. Figure 2-2 shows the drawdown in Cox #2 and Cox #5 versus distance from Cox #3 at the end of the 6 hours of pumping on a semi-log plot. The distance-drawdown relationship is extrapolated back to 0.83 feet, the radius of the borehole (assuming a 20-inch borehole). This extrapolated drawdown of 20.4 feet is divided by drawdown of 33.75 feet observed in Cox #3 after 6 hours of pumping to calculate a well efficiency of 60%.

No previous estimates of well efficiency are available for comparison. However, 100% well efficiency would result in a specific capacity of 2.2 gpm/foot, which is lower than prior reported specific capacity values. It is possible that Cox #3 no longer accesses its full screen length due to buildup on the screen or clogging in the filter pack, limiting its current specific capacity.

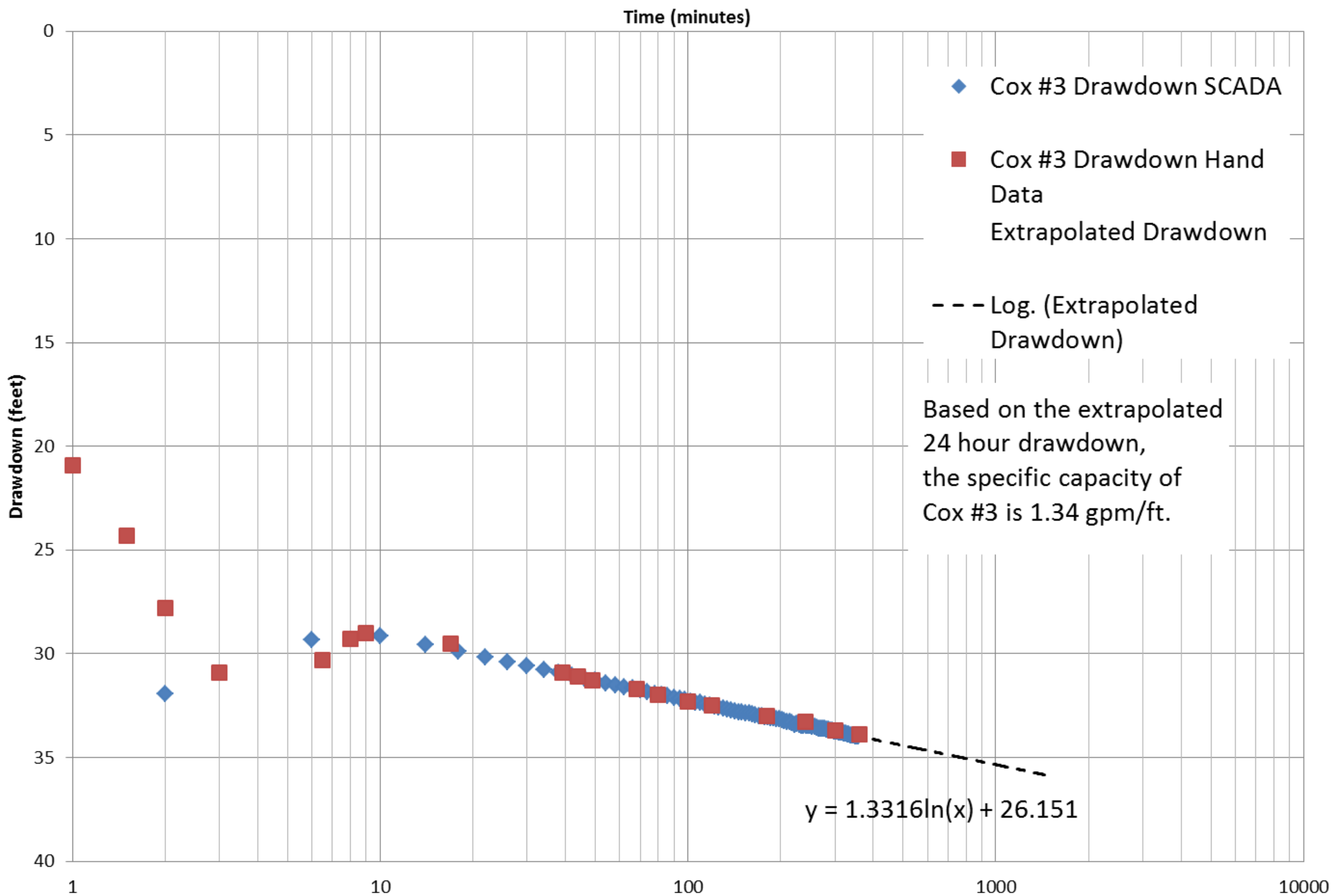

Figure 2-1. Pumping Well Drawdown Data for Cox #3 Pumping of 47.9 gpm

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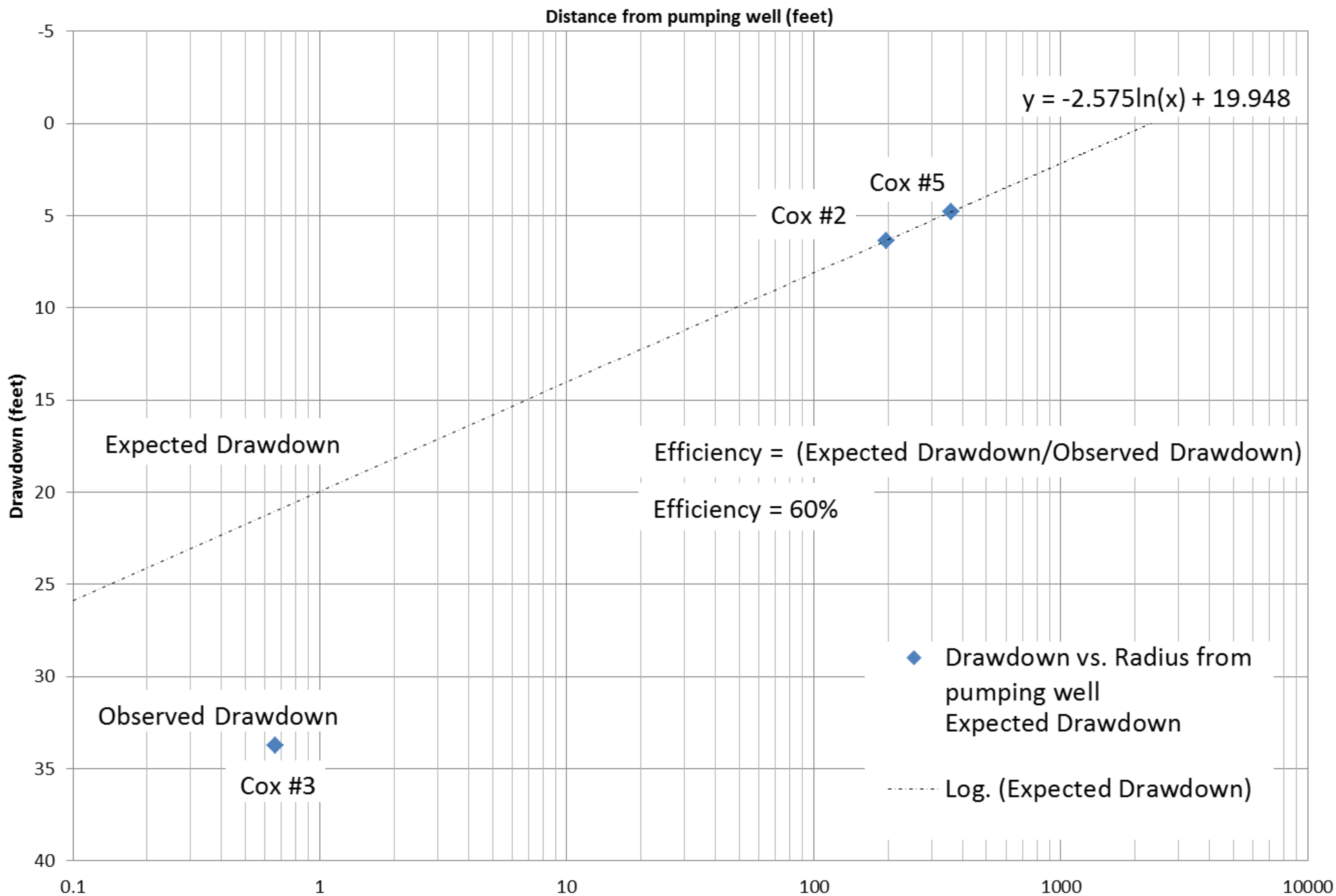

Figure 2-2. Distance-Drawdown Data at End of 6 Hours of Pumping at Cox #3 (47.9 gpm)

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2.5 ESTIMATED AQUIFER PROPERTIES FROM COX #3 TEST

During the September 12, 2012 Cox #3 aquifer test, drawdown was recorded at Cox #2 and Cox #5. The drawdown curves from these observation wells are used to estimate aquifer properties in the Cox well field area. These estimates will be used in calibrating the groundwater model being developed for Task 1.1.

2.5.1 COOPER-JACOB METHOD

The Cooper-Jacob solution (1946) is used to estimate transmissivity and storativity from the drawdown versus time data for Cox #2 and Cox #5. The Cooper-Jacob solution is an approximation of the Theis method, which makes the following assumptions:

- Aquifer areal extent is infinite;
- Aquifer is homogeneous, isotropic, and of uniform thickness;
- Pumping well fully penetrates the aquifer;
- Flow to pumping well is horizontal;
- Aquifer is confined;
- Storage releases water instantaneously with decline in head; and
- Storage in pumping well is neglected

These assumptions are reasonable for the short-term (6 hour) Cox #3 aquifer test that monitors wells within 360.5 feet (the distance between Cox #3 and Cox #5.)

The Cooper-Jacob solution is based on a semi-log plot of drawdown versus time. After early time, drawdown increases linearly with the logarithm of time, and the Cooper-Jacob solution can be applied. Drawdown from flow rate Q over one log cycle of time, Δs, is used to calculate transmissivity, T, based on the following relationship:

$$T = \frac{2.3Q}{4\pi\Delta s}$$

The log-linear time-drawdown relationship is extrapolated to the time of zero drawdown, t_o. This value is used to calculate storativity, S, using the following equation:

$$S = \frac{2.25Tt_o}{r^2}$$

where: r is the distance from the pumping well.

2.5.2 TRANSMISSIVITY AND STORATIVITY BASED ON COX #2 AND COX #5 DATA

The Copper-Jacob solution is applied to the barometrically-compensated logger drawdown data from Cox #2 and the hand-sounded drawdown data from Cox #5. Cox #2 is 197 feet and Cox #5 is 360.5 feet away from the Cox #3, which was pumping at 47.9 gpm. The results are summarized in Table 2-1 and the solutions are shown in Figure 2-3 and igure 2-4.

Table 2-1. Transmissivity and Storativity Based on Cox #2 and Cox #5 Data Using Cooper-Jacob Solution

Observation Well	R (feet)	Δs (feet)	T (feet²/day)	t_o (days)	S
Cox #2	197	3.6	470	0.0037	0.00010
Cox #5	360.5	3.5	488	0.0087	0.00025

2.5.3 COMPARISON OF TRANSMISSIVITY WITH HISTORICAL SPECIFIC CAPACITY

Specific capacity (Q/s) can be estimated from transmissivity T and storativity S using the following equation developed by Theis, Brown, and Meyers (1963):

$$\frac{Q}{s} = \frac{4\pi T}{\ln \frac{4\pi t}{S r_w^2} - 0.5772}$$

This equation assumes no well loss (100% efficiency) so the result must be multiplied by the well efficiency to estimate the actual specific capacity. For a casing radius, r_w, of 6 inches, the equation estimates the maximum specific capacity as 1.75-1.9 gpm/foot. This is lower than prior reported values of specific capacity (3 to 4.3 gpm/foot). As mentioned above, it is possible that Cox #3 no longer accesses its full screen length, limiting its current specific capacity. We will use the lower historical value of 3 gpm/foot as a specific capacity goal for future well performance.

2.5.4 AQUIFER PARAMETER TO USE IN NUMERICAL MODEL CALIBRATION

Transmissivity is equal to the product of horizontal hydraulic conductivity and aquifer thickness. Therefore, transmissivity estimates can be translated to horizontal hydraulic conductivity, a parameter required by the numerical model. Since the Cooper-Jacob solution assumes that the pumping well fully penetrates the aquifer, the Cox #3 screen interval is used to estimate the aquifer thickness. Based on the video, Cox #3 is screened from a depth of 139 to 282 feet for an aquifer thickness of 143 feet. Dividing the average transmissivity from the Cooper-Jacob solution (478 ft^2/day) by this thickness results in a hydraulic conductivity of 3.4 feet/day. This value will be used to guide calibration of the numerical model for the Purisima F aquifer in the area of the Cox well field. Higher values for hydraulic conductivity may also be supported by the data, as it is possible that Cox #3 no longer accesses its full screen length, meaning that the transmissivity indicated by the aquifer tests is lower than the actual aquifer transmissivity.

Storativity values calculated from aquifer test data will not be used in numerical model calibration. The mechanisms for releasing groundwater from storage during such a short-term test as the 6 hour aquifer test are different from those that drive storage release during longer-term well usage such as that simulated by the numerical model, which uses monthly stress periods.

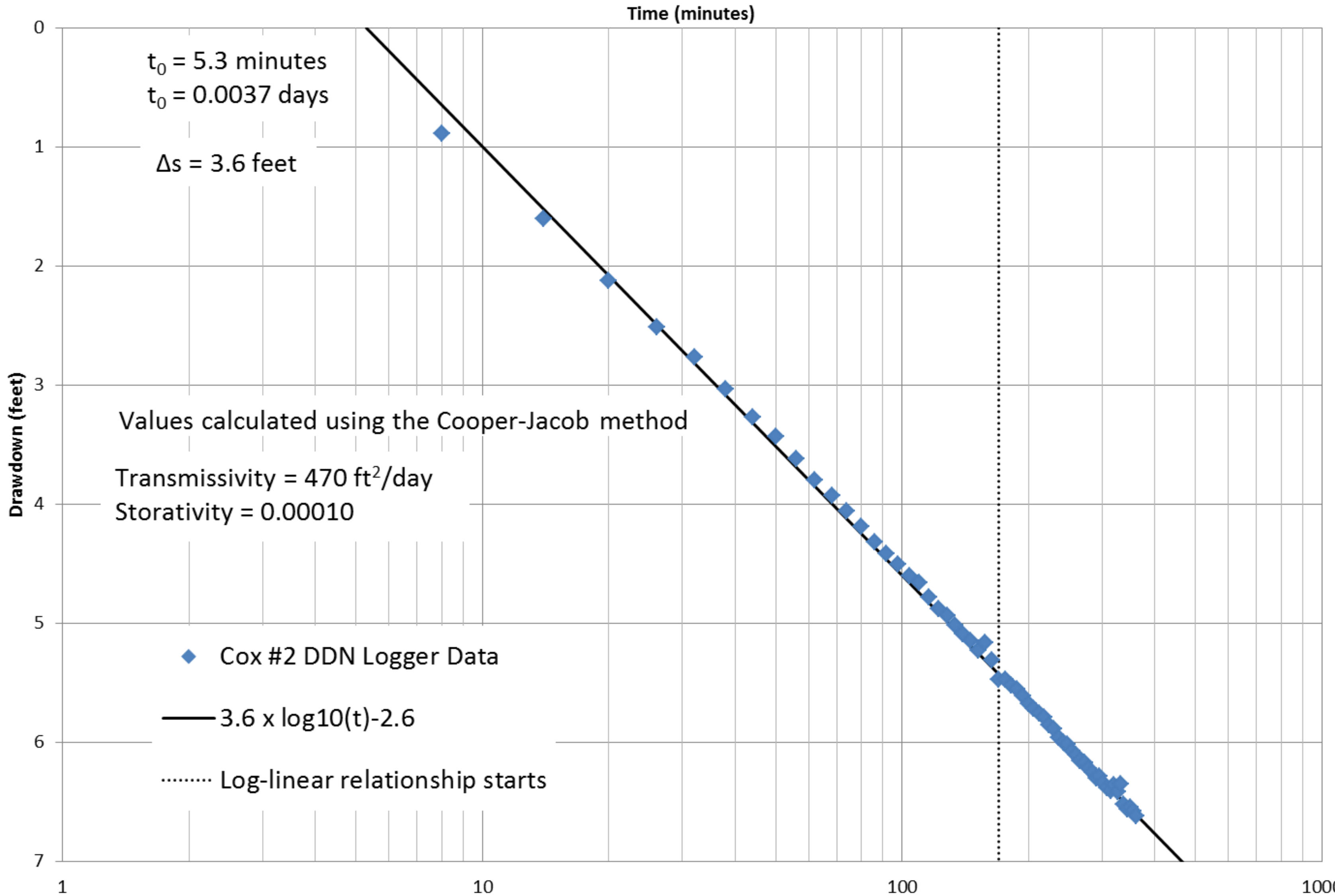
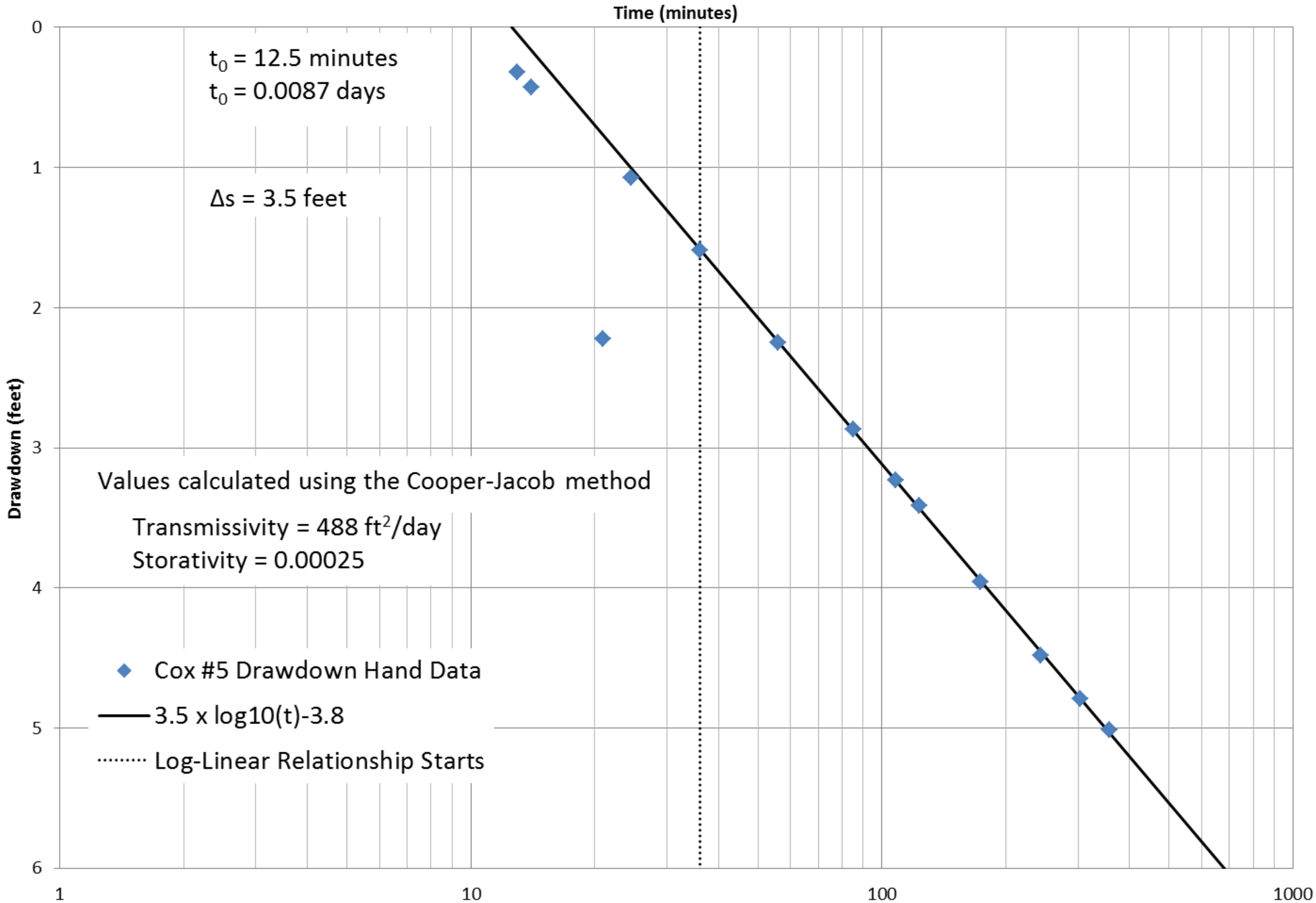


Figure 2-3. Cooper-Jacob Solution for Cox #2 Drawdown



igure 2-4. Cooper-Jacob Solution for Cox #5 Drawdown

2.6 Cox #3 Well Capacity over Dry Season

During seasons of low recharge, groundwater levels in pumping wells do not completely recover between pumping cycles. As a result, drawdown over multiple cycles is greater than drawdown in one cycle. Drawdown and recovery data from the Cox #3 aquifer test are used to estimate drawdown over multiple cycles. This calculation can be extended to identifying the combinations of pumping rate and daily pumping cycle duration that will keep drawdown within the well's allowable drawdown. Allowable drawdown can be defined to keep pumping water levels above a specific level such as a level above the pump bowls (to prevent cavitation) or the top of the screen (to prevent aeration and risk of well damage). The well capacity for the dry season is determined from the calculated pumping rate and daily pumping cycle durations over the dry season.

2.6.1 Estimating Drawdown over Multiple Cycles from Cox #3 Aquifer Test

Estimating drawdown over multiple cycles includes two components. One component is the expected drawdown from short-term (daily) pumping of Cox #3. The second component is the anticipated seasonal drop in well water levels. The anticipated seasonal drop in well water levels is estimated using the concept of residual drawdown. The residual drawdown after 180 days of daily pumping is extrapolated from the recovery data in Cox #3. This calculation assumes no recharge to aid recovery. The analysis is conducted using the following steps.

1. Extrapolate drawdown data from Cox #3 (Figure 2-1) to identify drawdown for daily pumping durations of 8, 10, 12, and 15 hours of pumping at the same pumping rate as for the aquifer test (47.9 gpm).

2. Extrapolate recovery data from Cox #3 (Figure 2-5) to estimate one daily pumping cycle's residual drawdown (i.e. the amount of drawdown not recovered before the start of the next pumping cycle) after each day between 1 and 179 days. The x-axis in Figure 2-5 is t/t' where t is the time since start of pumping and t' is the time since end of pumping. For example, after 1 day of recovery for a pumping duration of 8 hours, t/t' is 32/24 or 1.33 and can be used to extrapolate the residual drawdown at the end of pumping on the 2^{nd} day.

3. Sum the residual drawdowns calculated in step #2 for all 179 pumping cycles to estimate the total residual drawdown after 179 days of recovery (Figure 2-6).

4. Add the drawdown due to pumping the well between 8 and 15 hours (step #1) to the 179-day total residual drawdown (step #3) to estimate total drawdown after 180 days.

Table 2-2 shows the results of this analysis.

Table 2-2. Estimated Drawdown after 180 Days Pumping Cox #3 Well 47.9 gpm

	Daily Pumping Durations			
	8 hours	**10 hours**	**12 hours**	**15 hours**
First Day Recovery Residual Drawdown	0.31	0.38	0.44	0.53
Total Residual Drawdown	2.0	2.5	3.0	3.6
Daily Drawdown	34.4	34.7	34.9	35.2
Total Drawdown	36.4	37.1	37.9	38.8

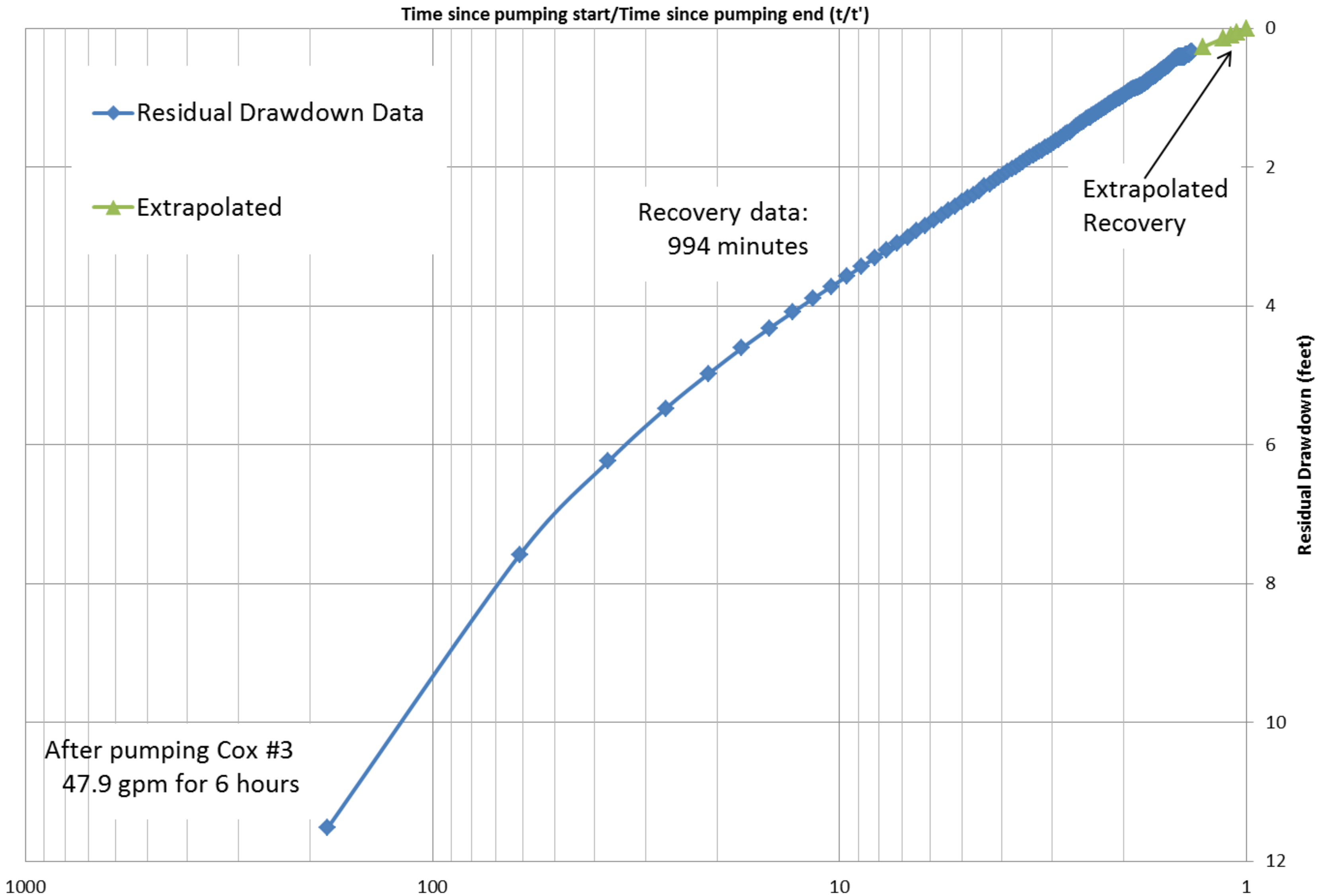


Figure 2-5. Cox #3 Recovery Data after 6 Hours Pumping Cox #3 at 47.9 gpm

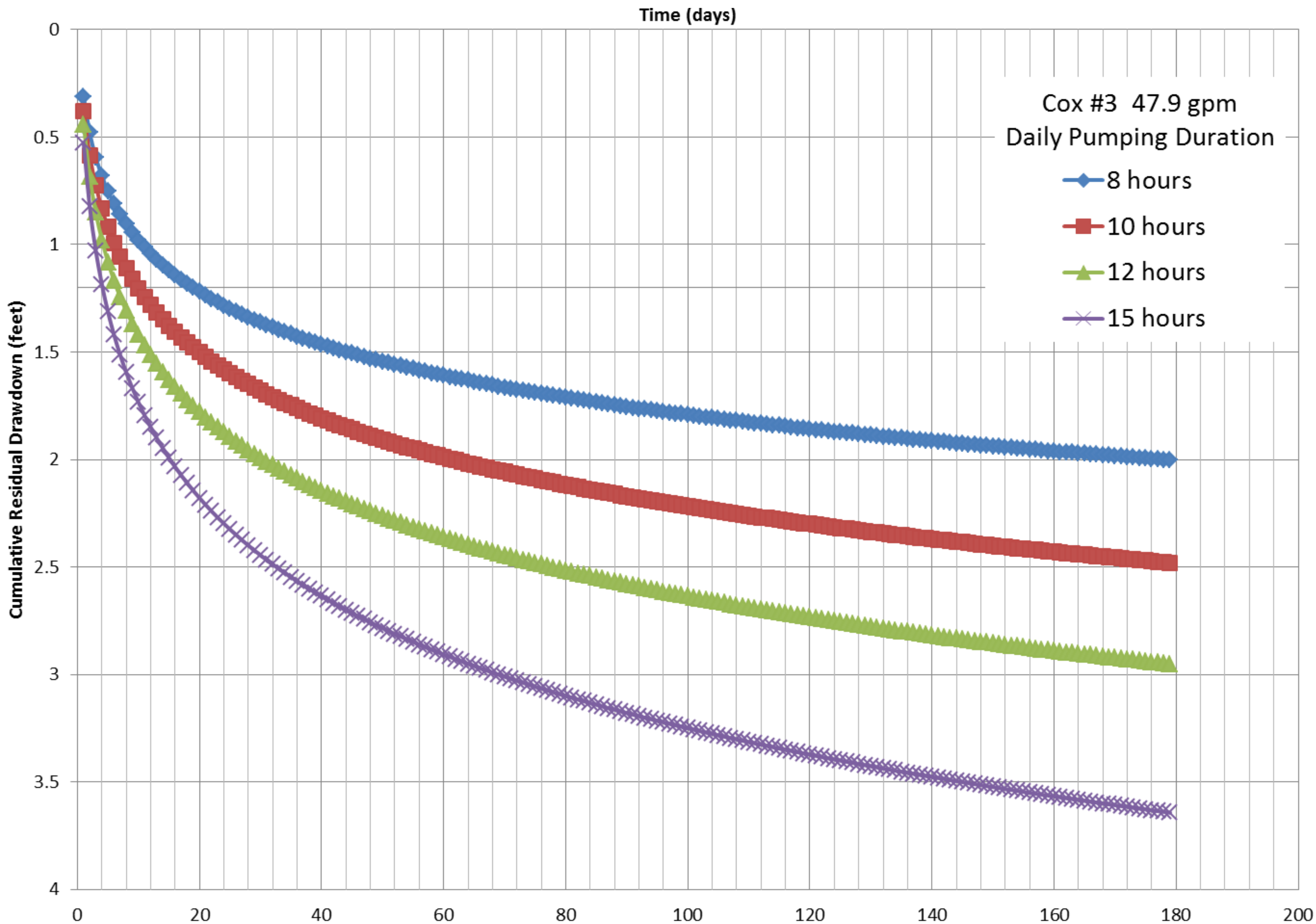


Figure 2-6. Residual Drawdown for Cox #3 for Days without Recharge Recovery

2.6.2 Cox #3 Allowable Drawdown and Capacity Over Dry Season

As noted above, allowable drawdown can be determined based either on the depth of the pump bowls or the depth of the screen. At Cox #3, allowable drawdown is based on the top of the screen because the pump bowls in Cox #3 are below the top of the screen, at a depth of 189 feet. Although wells can operate in practice with groundwater levels dropping below the top of the screen, it is considered best practice to prevent this from occurring. The videos show that the perforations begin at a depth of 139 feet, as opposed to 172 feet as documented in the well drawing. The static depth to water prior to the aquifer test (97.5 feet) is consistent with monthly static groundwater levels measured at Cox #3. The allowable drawdown is therefore 41.5 feet (139 feet minus 97.5 feet).

In order to calculate the pumping rate that will keep total drawdown within allowable drawdown, the allowable drawdown is divided by the total drawdown in Table 2-2 and multiplied by the test pumping rate of 47.9 gpm. Total capacity over the dry season is calculated by multiplying the allowable pumping rate for each pumping duration by 180 days (Table 2-3).

Table 2-3. Allowable Pumping Rate and Capacity over Dry Season for Cox #3

	Daily Pumping Durations			
	8 hours	**10 hours**	**12 hours**	**15 hours**
Allowable Drawdown/Total Drawdown from Table 2-1	1.14	1.11	1.09	1.07
Allowable Rate (gpm)	54.4	53.5	52	51
Dry Season Capacity (acre-feet)	14	18	21	25

These capacities are greater than recent production at Cox #3 as total annual pumping from the well has ranged from 5 acre-feet to 25 acre-feet since 1999. Meanwhile, pumping water levels have been maintained above the top of the screen (depth of 139 feet) since 2003. However, these capacities are far below what would be required to implement the groundwater management strategy of shifting pumping from the Rob Roy well field to the Cox well field, as May to October pumping at the Rob Roy well field has ranged from 305-415 acre-feet since 1999.

2.6.3 Potential Dry Season Capacity for New Well

A replacement well for the Cox well field could have a higher dry season capacity in two ways. First, the new well is expected to have a higher specific capacity than the existing well, pumping at a higher rate for the same amount of drawdown. Second, the new well could be designed to increase the allowable drawdown by having a lower top of screen than in the existing well.

In order to estimate the potential dry season capacity for a new Cox well, we make the following assumptions:

1. The new well meets the specific capacity goal of 3.0 gpm/foot based on historical reports at Cox #3, or 2.2 times the current specific capacity.

2. The drawdown and recovery data observed for Cox #3 in the aquifer test apply to the new well; based on the specific capacity goal, it is assumed that the new well can pump at 105 gpm, or 2.2 times the aquifer test rate of 47.9 gpm.

With these assumptions, we can use the same methods as above to calculate pumping rates that will keep total dry season drawdown within allowable drawdown for different daily pumping durations. Two different allowable drawdowns are tested. An allowable drawdown of 77.5 feet is calculated based on keeping the pumping water level above a depth of 175 feet at the Cox #3 location (ground surface elevation 290 feet). Assuming a top of screen at 200 feet depth, this allowable drawdown would leave 15 feet of head above an assumed pump intake depth of 190 feet to prevent cavitation. An allowable drawdown of 117.5 feet is calculated based on keeping the pumping water level above a depth of 215 feet at the Cox #3 location. Assuming a top of screen at 240 feet depth, this allowable drawdown would leave 15 feet of head above an assumed pump intake depth of 230 feet to prevent cavitation. Table 2-4 shows the calculated dry season capacities for these two allowable drawdowns. These capacities of approximately 60-160 acre-feet could shift 15-50% of the range of Rob Roy dry season pumping since 1999.

Table 2-4. Potential Dry Season Capacity for New Well

Allowable Drawdown		Daily Pumping Durations 8 hours	10 hours	12 hours	15 hours
77.5 feet	Allowable Drawdown/Total Drawdown from Table 2-1	2.13	2.09	2.05	1.99
	Allowable Rate (gpm)	228	224	219	214
	Dry Season Capacity (acre-feet)	61	74	87	106
117.5 feet	Allowable Drawdown/Total Drawdown from Table 2-1	3.23	3.16	3.10	3.02
	Allowable Rate (gpm)	346	339	333	324
	Dry Season Capacity (acre-feet)	92	112	132	161

2.7 COX #5 FLOW AND WATER QUALITY PROFILE

2.7.1 COX #3 ACCESS

Although the May 11, 2012 chain access survey showed lack of access for BESST Inc.'s flow and water quality profiling tools at Cox #3, a second chain access survey was conducted on Cox #3 on June 5, 2012. The pump had been removed and replaced in the meantime, possibly improving access. However, during this test the dummy tool and weights got permanently stuck in the access tube at a depth of approximately 180', near the pump intake. The dummy tool was left in the well and tied off after the line was cut. This effectively blocked access to Cox #3. At this point the decision was made to profile Cox #5 due to the accessibility issues at Cox #3.

2.7.2 PROFILE DETAILS

BESST, Inc. profiled CWD Cox #5 well June 5th and 6th, 2012.

On June 5, the Cox #5 pump was turned on at 1130 hours. The pumping rate was adjusted to be approximately 100 gpm during the profiling. Though the pumping rate was only recorded once per day, it was checked and adjusted multiple times to maintain a steady pumping rate of about 100 gpm. Once the water level in this well had stabilized at around 185 feet (for an approximate drawdown of 60 feet), BESST, Inc. began to conduct its injections of Rhodamine dye and to record the return times from various depths using a fluorometer.

Injections were performed from 1200 to 1500 hours. The injection depths used for the profiling were 200, 204, 208, 213, 217, 221, 225, 227, 230, 232.5, 235, 240, 245, 247.5 and 254 feet. Following the dye tracer study, depth-discrete water quality samples were collected from this well. Samples were collected on June 5 at the well head and at depths of 229, 237.5, 247.5 and 254 feet. The Cox #5 pump was turned off at 1730 hours, with a final pumping water level depth of around 185 feet.

On June 6, the Cox #5 pump was turned on at 1100 hours after a static water level was measured at a depth of 126 feet. The pumping rate was again approximately 100 gpm. This rate was checked and adjusted throughout the day to maintain a flow of about 100 gpm. After the pumping water level depth fell below 178 feet, BESST, Inc. collected discrete-depth water quality samples at depths of 198 and 215 feet. The Cox #5 pump was turned off at 1430 hours, with a final pumping water level depth of around 185 feet.

The samples were sent to Monterey Bay Analytical Services to be analyzed for chromium VI, General Water Quality parameters (including Iron and Manganese), and Title 22 inorganics as shown in Table 2-5.

Table 2-5. Cox #5 Depth-Discrete Sample Analyses

Sample	chromium VI	General Water Quality	Title 22 Inorganics
Well Head	X		X
198		X	X
215	X		X
229		X	
237.5	X		X
247.5		X	
254		X	

2.7.3 Flow Profile Results

The return time of the Rhodamine dye (as determined as the peak in Rhodamine concentration measured by groundwater fluorescence) indicates the amount of time required for groundwater at the injection site to reach the well head. The difference in return times of the Rhodamine dye between two different injection depths is divided by the difference in the depths to calculate the average in-well velocity between the two depths. Average volumetric flow in each interval is calculated by multiplying the interval velocity by the cross-sectional flow area (accounting for the casing diameter and the pump column, if present). Casing diameter is estimated to reduce from 12 to 10 inches at 224 feet, and from 10 to 8 inches at 240 feet based on the well video and well construction diagram). The pump intake was at a depth of 237 feet. Flow is assumed to travel downwards from the upper screen and the top of the second screen to the pump intake. Flow is assumed to travel upwards from the bottom of the well to the pump intake. Table 2-6 shows the flow profile results with the blank section highlighted, and the uncorrected average flow is plotted against depth in Figure 2-7. The calculated flow is considered uncorrected, as it is not adjusted to the total flow measured at the well head. This is due to the presence in the intervals near the pump intake of both measurement noise and the presence of the reduction in casing diameter to about 8 inches.

Precise flows at injection points or any other points within the screened interval should not be calculated with a simple flow balance so flow contributions by the aquifer within each injection interval are not calculated. However, the average well flow for an interval in the blank is an estimate for the constant flow through the blank section. The interval between the depths of 213 and 217 feet is between the two screen sections of the well. The calculated flow of 20 gpm for this interval shows that only a small percentage of flow comes from the upper screen.

Further analysis of aquifer flow contributions within screen intervals requires a more advanced tool such as an axi-symmetric model or the US Geological Survey program AnalyzeHole. Using an advanced tool for further analysis will be necessary to predict flow for a well with a different design.

Table 2-6. Cox #5 Flow Profile Results

Interval Depth (feet)		Return Time (seconds)		Avg. Absolute Velocity (feet per second)	Diameter (inches)		Uncorrected Avg.Flow (gpm)
Top	Bottom	From Top	From Bottom		Casing	Pump Column	
200	204	536	408	1.9	12	8	6
204	208	408	322	2.8	12	8	9
208	213	322	258	4.7	12	8	15
213	217	258	219	6.2	12	8	20
217	221	219	194	9.6	12	8	31
221	225	194	179	16.0	12	8	52
225	227	179	176	40.0	10	8	59
227	230	176	171	36.0	10	8	53
230	232.5	171	167	37.5	10	8	55
232.5	235	167	155	12.5	10	8	18
235	240	Pump Intake at 237 feet					
240	245	148	141	42.9	8	0	112
245	250	141	146	60.0	8	0	157
250	254	146	174	8.6	8	0	22

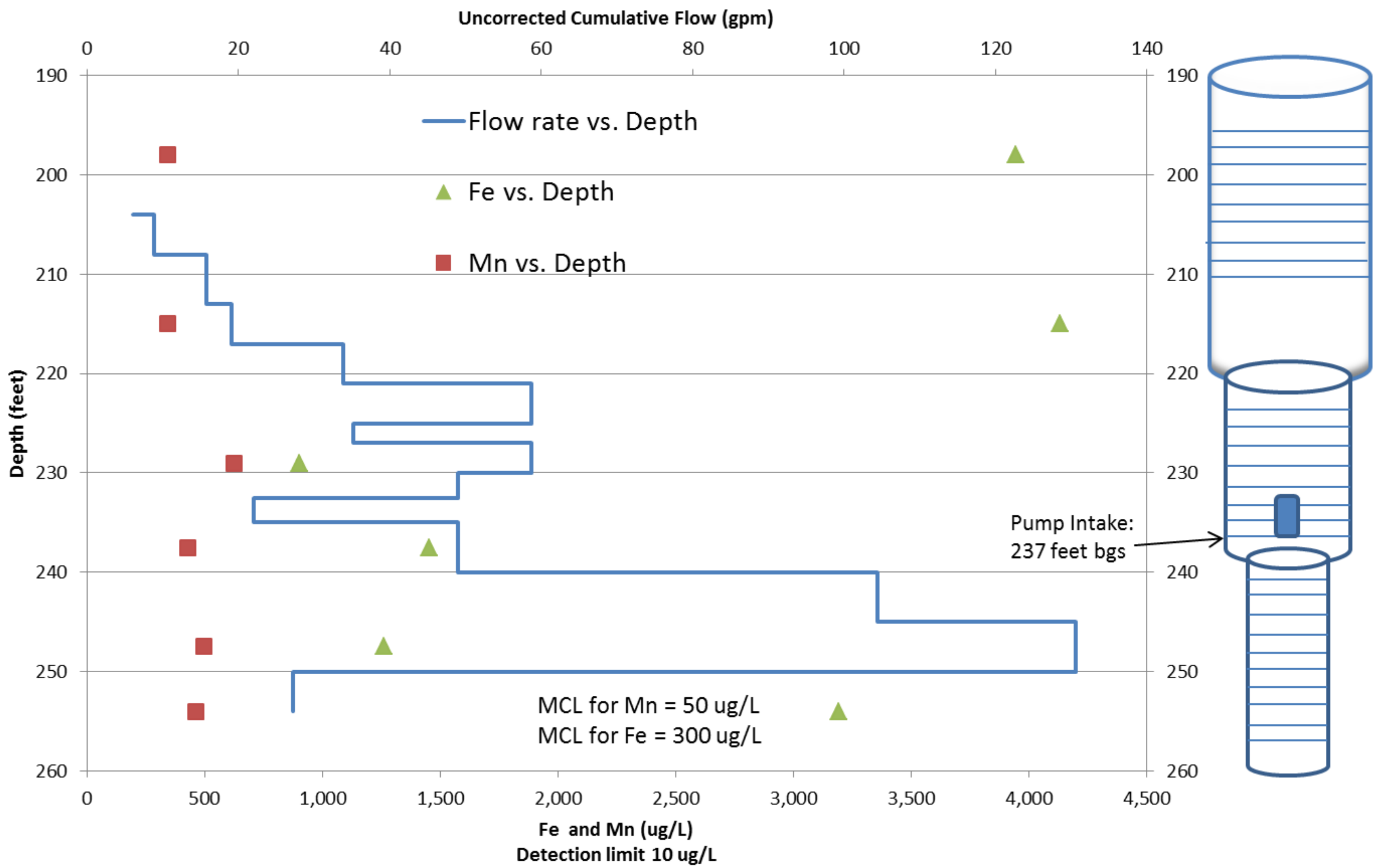


Figure 2-7. Cox #5 Flow and Water Quality Profile

2.7.4 Water Quality Profile Results

Iron and manganese are the water quality constituents of primary concern at Cox #5, as well head concentrations are greater than the secondary drinking water standards of 300 μg/L for iron and 50 μg/L for manganese. Some of the depth-discrete samples provided concentrations for iron and manganese in the well while Cox #5 was pumping. Table 2-7 and Figure 2-7 show iron and manganese concentrations with depth and at the well head. The blank section is highlighted on Table 2-7.

Table 2-7. Iron and Manganese Concentrations with Depth in Cox #5

Sample	Iron (μg/L)	Manganese (μg/L)
198	3,943	345
215	4,129	342
229	904	625
Well Head (Intake at 237)	1,202	518
237.5	1,455	427
247.5	1,260	497
254	3,190	465

Since it is not recommended to use a simple flow balance to calculate flow contribution by the aquifer in each interval, it is not recommended to use a simple mass balance to calculate iron and manganese contribution by the aquifer in each interval. However, an approximate estimate of the contribution of the upper screen can be calculated by using the 215 foot sample from the blank below the upper screen, the uncorrected flow in the blank section (20 gpm) and the well head flow (100 gpm). This calculation estimates that approximately 310,000 out of 460,000 μg/min (or 70%) of iron is produced from the upper screened interval, but only approximately 26,000 out of 200,000 μg/min (or 10%) of manganese is produced from the upper screen.

These results indicate that a new well with a top of screen below the elevation of the bottom of the upper screen may reduce iron concentrations at the well head while potentially increasing the manganese concentrations at the well head. A deeper top of screen for a new well would also increase the well capacity by increasing allowable drawdown, as discussed above.

Chromium VI was non-detect at a detection limit of 1.0 μg/L for the well head sample or the samples at 215 and 237.5 feet.

2.8 PRIVATE WELL MONITORING DURING COX #5 WELL PROFILE

2.8.1 MONITORING DETAILS

During the June 5 and 6 profile of Cox #5, groundwater levels were monitored at two nearby private wells. The two wells were at 344 Possumwood Ridge Road (approximately 1,300 feet from Cox #5) and 450 Cox Road (approximately 315 feet from Cox #5). Due to installation of pumps in the wells, sonic sounders were used to measure well water levels. These sounders measure water levels by emitting a sound wave and measuring the return time of sound waves reflected from the water surface. These sounders are susceptible to interference, so filtering of the data is required.

2.8.2 POSSUMWOOD RIDGE ROAD PRIVATE WELL

An Enosounder sonic sounder was installed at the Possumwood Ridge Road wellhead. The Enosounder recorded water measurements from May 19-20, May 21-22, and during the Cox #5 well profiling June 5-6. Based on recorded error codes and visual inspection, anomalous data were removed. The data for May 21-22 and June 5-6 show short but frequent water level cycles from pumping at the private well.

Figure 2-8 shows recorded well water levels during June 5 and 6. Groundwater level variation caused by pumping at the Possumwood Ridge Road well masks any drawdown caused by pumping Cox #5, limiting the usefulness of this well in analyzing the effect of pumping at Cox #5.

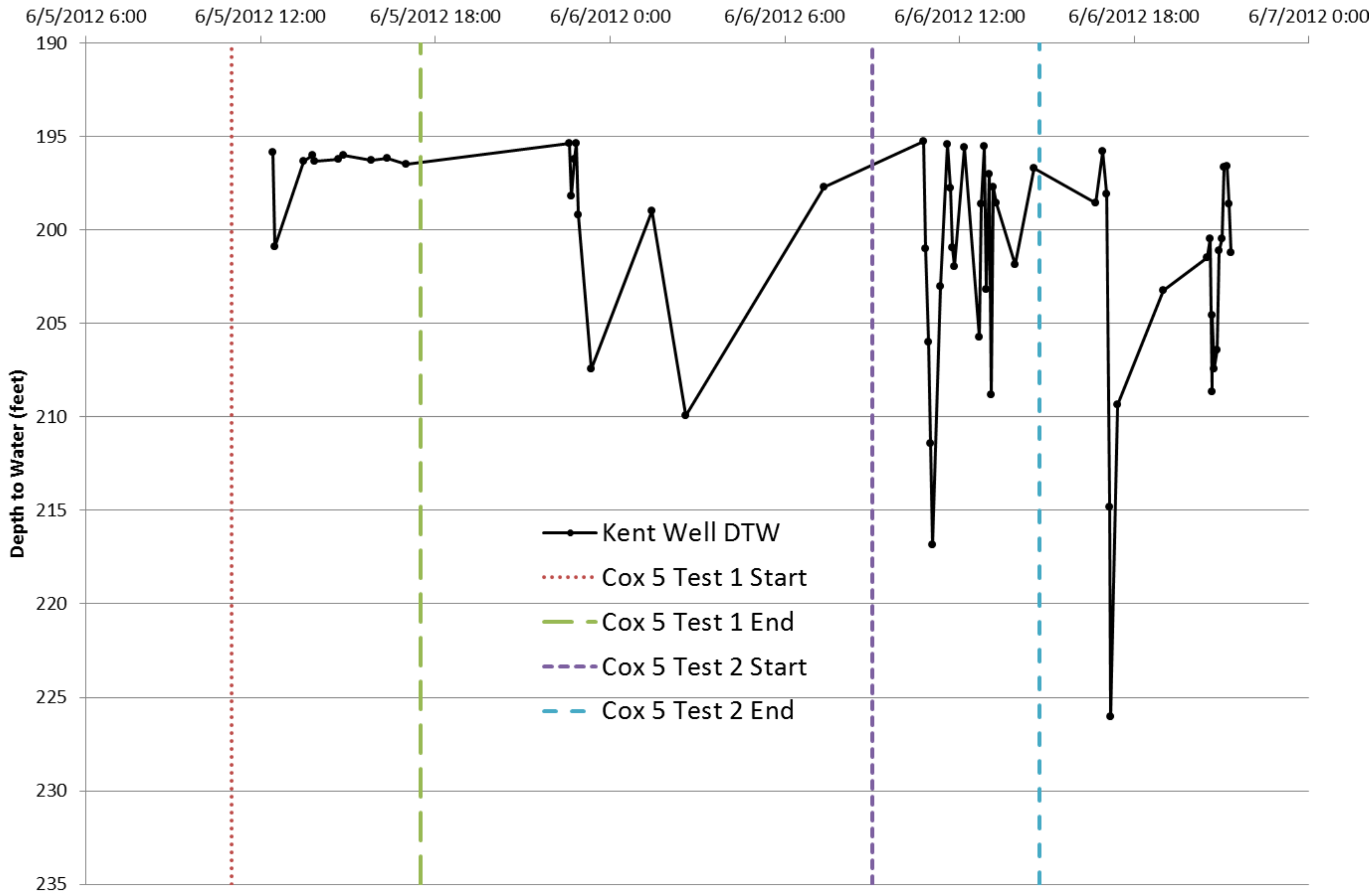


Figure 2-8. Groundwater Levels at Possumwood Ridge Road Well During Cox #5 Profile

2.8.3 COX ROAD PRIVATE WELL

A Ravensgate sonic sounder was used to measure four groundwater levels in the Cox Road private well on June 5-6. Table 2-8 shows the measurement times and results relative to pumping during the Cox #5 well profile. The data observed on June 6, measurements taken before, during, and after pumping, do not show any definite pattern of drawdown or recovery.

Table 2-8. Cox Road Private Well Measurements During Cox #5 Well Profile

Date and Time	Private Well Depth to Water (feet)	Cox #5 Pump On/Off
6/5/2012 11:30		Pump started
6/5/2012 12:18	72.2	
6/5/2012 17:00		Pump stopped
6/6/2012 09:37	71.4	
6/6/2012 11:00		Pump started
6/6/2012 14:27	70.1	
6/6/2012 14:30		Pump stopped
6/6/2012 14:56	70.2	

2.9 OTHER DATA COLLECTED

The data discussed above will be provided in Appendix A on a compact disk. Other data that were collected, but not analyzed, will also be included on the Appendix A compact disk. These data include:

- Groundwater level data from Cox #5, Cox #2, and Cox #3 collected during the Cox #5 profile that were not analyzed to evaluate well performance or aquifer properties because the pump was stopped and re-started to accommodate installation of BESST, Inc. profiling tools;
- Laboratory reports for all water quality analyses performed for Rob Roy #12 and Cox #5 well profiles; and
- Recovery data collected during Cox #3 aquifer test.

SECTION 3
RECOMMENDATION FOR COX ROAD WELLS
(TASK 4.3)

As part of grant Task 4.3, recommendations were provided to CWD for improving the Cox well field to meet its customers' beneficial uses. Specifically, the results of the Cox well test data and other available information were evaluated to recommend whether the Cox wells should be rehabilitated or replaced. Other available information includes historical data, a set of well schematics dated April 1983, anecdotal recollections of knowledgeable personnel, and recent field work. This report section documents the recommendations for CWD’s Cox road wells. The section was distributed to the TAC for review as part of a draft technical memorandum on January 9, 2013.

There are three wells in the CWD’s Cox Road Well field –#2, #3, and #5. Cox #2 is inactive, while the remaining two are active wells. All three are located on Cox Road proximate to the CWD’s office.

3.1 COX #2 ASSESSMENT

The Cox #2 well was drilled in 1953 by the cable tool method. The well was originally 12 inches in diameter. Although the reason is undocumented, an 8-inch diameter steel liner with a stainless steel screen intake section was installed in 1975. The 12-inch diameter casing was perforated between 105 and 245 feet, and the 8-inch liner had perforations from 220 to 260 feet. The well’s original performance is undocumented; however, a 1970 Department of Public Health (DPH) form reports a discharge rate of 300 gpm with 34 feet of drawdown, for a specific capacity of 8.8 gpm/foot. This specific capacity value predates the installation of the liner. The well drawing provided as Appendix C documents a post-liner specific capacity of 4.7 gpm/foot at a discharge rate of 437 gpm in 1983.

Cox #2 has been inactive since 1987 due to high iron concentrations. It is assumed that the well is unusable and will either be destroyed or designated as a dedicated monitoring well.

3.2 COX #3 ASSESSMENT

The Cox #3 well is located approximately 150 feet from the CWD's office. Available records provided in Appendix C document this well to have been drilled in 1960 by the rotary method. The well drawing shows the well to be 300 feet in depth and constructed of 12-inch diameter steel casing with horizontal saw-cut perforations between the depths of 172 and 292 feet. The well is reportedly gravel packed and has a surface seal to a depth of 50 feet.

Performance data for Cox #3 are sparse. The DPH form dated 1970 documents a discharge rate of 300 gpm with a static water level of 113 feet depth and pumping water level of 215 feet depth, for a specific capacity of about 3 gpm/foot. A hand notation on the well diagram lists a specific capacity of 4.31 gpm/foot in 1983 at 230 gpm. The September 12, 2012 aquifer test showed a 24-hour specific capacity of 1.34 gpm/foot (Section 2.4.2).

As discussed above, videos of the well from 1996 and 2012 were reviewed. The review of the videos reveals that the perforations start at a depth of 139 feet (as opposed to 172 feet as documented in the drawing). The video review reveals the perforations to be vertical machine cut slots rather than the horizontal slots documented in the drawing. The perforations are documented at 3/32 inch; based on the well video this may be accurate, although the perforations may be as large as 1/8 inch. The drawing states that the well was constructed by the rotary method; however, the use of 6 foot lengths of pipe indicates that the well may be of cable tool construction.

In the 1996 video, there appear areas with holes, enlarged perforations, or both, areas obscured by encrustation, and gravel pack (or coarse material) visible through the perforations. At the time of the 1996 video the well was in fair to poor condition. The 2012 video showed an accumulation of fill of about 10 feet, but beyond that, little additional degradation of the well's condition.

3.3 COX #5 ASSESSMENT

According to available records, Cox # 5 was drilled by the rotary method in 1967. It was drilled to a depth of 266 feet and is constructed of 12-inch diameter casing to a depth of 218 feet. Below this depth it is 8-inch diameter casing. The well is perforated between 195 and 210 feet depth in the 12-inch diameter section and between 228 and 262 depth feet in the 8-inch diameter section. The casing and screen materials are undocumented in the drawing.

Like the other Cox wells, limited performance data are available. The 1970 DPH form documents a discharge rate of 300 gpm with 53 feet of drawdown, resulting in a specific capacity of 5.7 gpm/foot. The well schematic, dated March 1983, documents a discharge rate of 166 gpm with 56.5 feet of drawdown, resulting in a specific capacity of 2.94 gpm/foot. The maximum difference between static and pumping water levels measured in 2011 was 74 feet. The discharge rate during the Cox #5 profile was 100 gpm. This provides a rough estimate of specific capacity of 1.4 gpm/foot.

In the May 29, 2012 video, no apparent structural casing problems were noted. The video revealed that in the upper screened interval (195 to 210 feet depth) the screen was mostly plugged on one side of the casing and slightly plugged on the other. This likely results from the pump being set against the side of the casing. The lower screen was plugged a couple feet from the top and a couple feet from the bottom of the screened interval; otherwise the screen was relatively clean. The plugging on the top and bottom is likely galvanically-driven encrustation due to dissimilar metals – the suspected carbon steel blank and cellar juxtaposed with the stainless steel screen. The video survey suggested 3.5 feet of fill.

3.4 COX WELL FIELD CONCLUSIONS

The Cox wells are over 45 years old. Cox #2 has a liner installed, has high iron concentrations, and is assumed to be unusable. Given that the typical service life for steel casing is around 30 to 40 years, all of the wells are at, or near, the end of their service life.

Additionally, Cox #5 is constructed of both stainless steel and carbon steel. Galvanic action between dissimilar metals can result in accelerated degradation of casing and plugging of screen shortening service life. Typically, the juxtaposition of dissimilar metals in a well is addressed in the well design by calling for extra-thick welding collars where the dissimilar metals touch to compensate for accelerated degradation of the area. It is unknown whether this step was taken in the construction of Cox #5.

Although data are sparse, the available performance data reveal significant declines in well performance.

The performance of Cox #3 and Cox #5 might be improved through rehabilitation work. However, rehabilitation work (exclusive of hydrogeologic oversight) is estimated at $25,000 per well and could result in well failure.

3.5 COX WELL FIELD RECOMMENDATIONS

Given the age and documented degradation of well performance it is recommended that all the existing wells be destroyed or converted to dedicated monitoring wells and replaced by a single modern-designed production well. A well-designed and constructed well would likely have a discharge capacity of 300 to 400 gpm.

Based on the analysis of dry season well capacity and the flow and water quality profile, it is recommended that the new well have a top of screen at least 200 feet below ground surface. In addition, drilling the well test hole up to 650 feet deep would give CWD the opportunity to screen more of the Purisima aquifer unit.

Cox #3 or Cox #5 could be maintained as a backup well, but the performance of these wells is likely to continue to deteriorate. In their current conditions, these wells would not be able to replace the discharge capacity of a new well should it be taken out of service.

SECTION 4
EVALUATE ROB ROY #12 WELL TO IMPROVE WATER QUALITY (TASK 4.2)

A primary motivation of the potential strategy to shift pumping from the Rob Roy well field to the Cox well field is the chromium VI detected at the Rob Roy well field. Estimates of the potential dry season capacity for a new well at the Cox well field show that CWD may not be able to shift enough pumping the Cox well field to reduce system concentrations of chromium VI below the future drinking water standard. Therefore, conducting a flow and water quality profile of CWD's largest current producer, Rob Roy #12, was added to grant task 4.2 to help evaluate the possibility of modifying the well to reduce chromium VI concentrations.

This report section documents the field activities, data collected, and data analyses of the Rob Roy #12 flow and water quality profile. The section was distributed to the TAC for review as part of a draft technical memorandum on January 9, 2013.

4.1 CHAIN ACCESS SURVEY

On May 11, 2012, a chain access survey on Rob Roy #12 was conducted and found that the well had adequate access for the profiling tools.

4.2 PROFILE DETAILS

BESST, Inc. carried out flow and water quality profiling of Rob Roy #12 on May 16 and 17, 2012.

On May 16, the static water level depth at Rob Roy #12 was 163 feet before the pump was turned on at 0930 hours. The pumping rate was adjusted to be approximately 590 gpm during the profiling, and was checked and adjusted multiple times to maintain a steady pumping rate of about 590 gpm. Once the water level in this well had stabilized, BESST, Inc. began to conduct injections of Rhodamine dye and to record the return times from various depths using a fluorometer. Injections were performed between 1200 and 1430 hours. The injection depths were 225, 229, 233, 240, 250, 260, 270, 284, 296, 310, 320, 330, 340, 350, 380, 420, 465, 490, 500, 510, and 520 feet. Following the dye tracer study, depth-discrete water quality samples were collected from the well head and at depths of 245 and 265 feet. The Rob Roy #12 pump was turned off at 1700 hours.

On May 17, the Rob Roy #12 pump was turned on at 1030 hours. The pumping rate at this time was also approximately 590 gpm. This rate was checked and adjusted throughout the day to maintain a flow of about 590 gpm. After the pumping water level in the well had stabilized, BESST, Inc. collected depth-discrete water quality samples at depths of 290, 305, 315, 335, 420, and 500 feet. The Rob Roy #12 pump was turned off at 1400 hours.

The samples were sent to Monterey Bay Analytical Services to be analyzed for chromium VI, General Water Quality parameters (including Iron and Manganese) and Title 22 inorganics as shown in Table 4-1.

Table 4-1. Rob Roy #12 Depth-Discrete Sample Analyses

Sample	chromium VI	General Water Quality	Title 22 Inorganics
Well Head	X		X
245	X	X	
265	X	X	
290	X		X
305	X	X	
315	X	X	
335	X	X	
420	X		X
500	X	X	

4.3 Flow Profile Results

The difference in return times of the Rhodamine dye between two depths is used to calculate the average velocity between the two depths, as described for Cox #5. Average flow in each of these intervals is calculated using the cross-sectional area for flow based on the casing diameter of 12 inches. The pump intake is above the uppermost screen, so flow is upwards for all screens, although the flow profile does show noise. Because flow was calculated throughout the screened interval, these flow results can be scaled up to the corrected average flow so that the total flow is equal to the well head flow of 590 gpm.

Table 4-2 shows the flow profile results with the blank sections highlighted. The corrected average flow is plotted against depth in Figure 4-1.

Table 4-2. Rob Roy #12 Flow Profile Results

Depth (feet)		Return Time (seconds)		Avg. Velocity (feet per second)	Uncorrected Avg.Flow (gpm)	Corrected Avg. Flow (gpm)
Top	Bottom	From Top	From Bottom			
229	240	106	116	1.1	388	519
240	250	116	124	1.3	441	590
250	260	124	134	1.0	353	472
260	270	134	146	0.8	294	393
270	284	146	162	0.9	308	413
284	296	162	175	0.9	325	436
296	310	175	194	0.7	260	348
310	320	194	212	0.6	196	262
320	330	212	229	0.6	207	278
330	340	229	246	0.6	207	278
340	350	246	273	0.4	131	175
350	380	273	331	0.5	182	244
380	420	331	408	0.5	183	245
420	465	408	503	0.5	167	224
465	490	503	559	0.4	157	211
490	500	559	592	0.3	107	143
500	510	592	656	0.2	55	74
510	520	656	No Return			

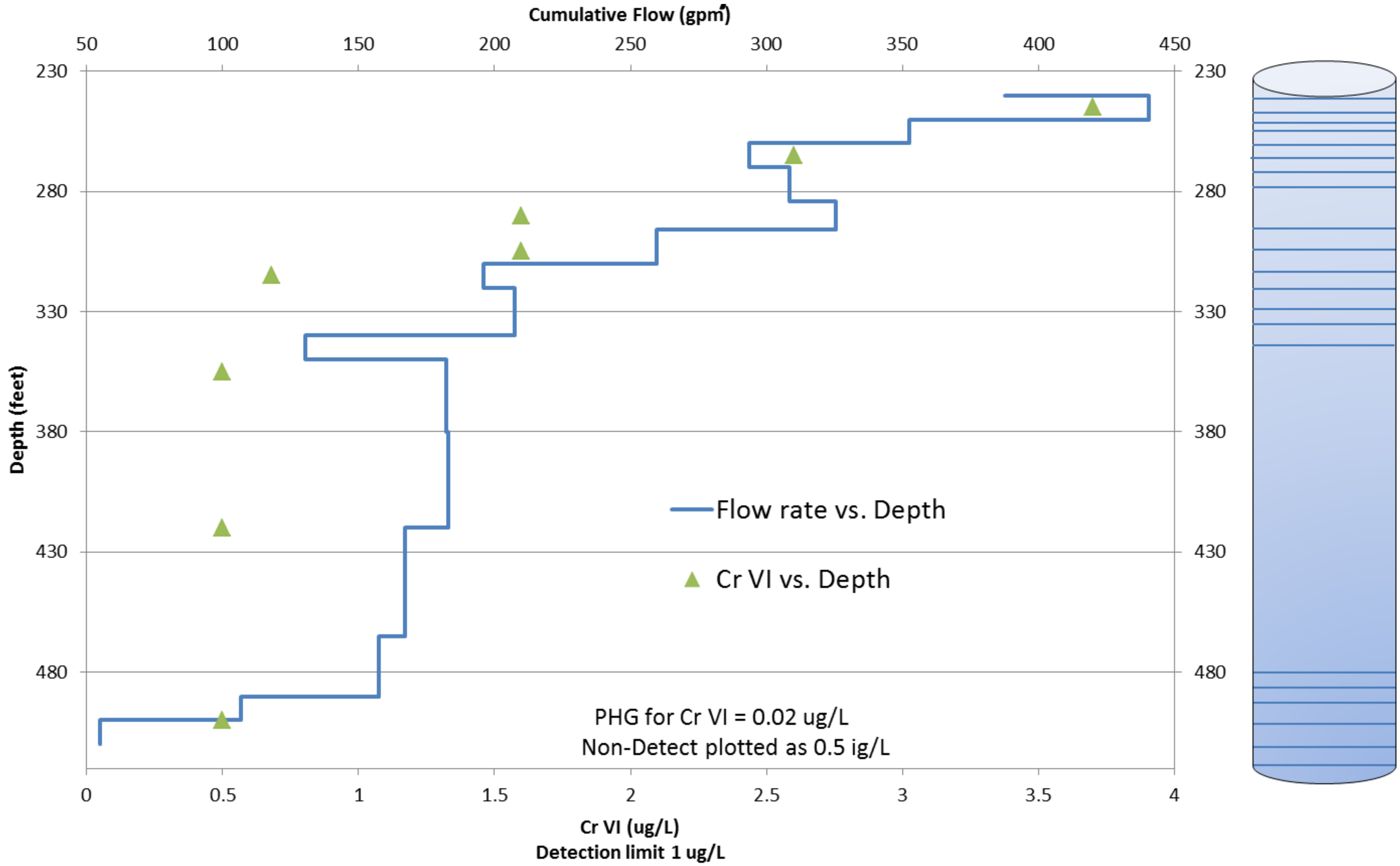


Figure 4-1. Rob Roy #12 Flow and chromium VI Concentration Profile

Precise flows at injection points or any other points within the screened interval should not be calculated with a simple flow balance so flow contributions by the aquifer within each injection interval are not calculated. However, the average well flow for an interval in the blank is an estimate for the constant flow through the blank section. The interval between the depths of 284 and 296 feet is between the upper and middle screens. The dye injection intervals 380 and 420 feet and 420 and 465 feet are between the middle and lower screens. Calculated flows in these intervals provide an estimate that 40% of the flow comes from the lowest screen, 34% of the flow comes from the middle screen, and 26% of the flow comes from the upper screen.

Further analysis of aquifer flow contributions within screen intervals requires a more advanced tool such as an axi-symmetric model or the US Geological Survey program AnalyzeHole. Using an advanced tool for further analysis will be necessary to predict flow for a modified well.

4.4 WATER QUALITY PROFILE RESULTS

Chromium VI is the water quality constituent of primary concern at Rob Roy #12, as the well head concentration of 3.7 μg/L may be above a future drinking water standard. Chromium VI is generally associated with the shallower Aromas Red Sands sediments. If the well is modified to draw a higher percentage from the underlying Purisima Formation, iron and manganese concentrations could become an issue. The iron concentration at the well head is 86 μg/L, below the drinking water standard of 300 μg/L. Manganese is non-detect at a detection limit of 10 μg/L. The depth-discrete samples provide concentrations for chromium VI, iron, and manganese in the well while Rob Roy #12 is pumping, as shown in Table 4-3. Figure 4-1 shows the depth-discrete chromium VI concentrations compared to the flow profile.

Table 4-3. Chromium VI, Iron, and Manganese Concentrations with Depth in Rob Roy #12

Sample	chromium VI (µg/L)	Iron (µg/L)	Manganese (µg/L)
Well Head	3.7	86	ND<10
245	3.7	26	ND<10
265	2.6	14	ND<10
290	1.6	27	ND<10
305	1.6	11	ND<10
315	0.68	22	ND<10
335	ND<1.0	16	ND<10
420	ND<1.0	422	ND<10
500	ND<1.0	71	ND<10

Since it is not recommended to use a simple flow balance to calculate flow contribution by the aquifer in each interval, it is not recommend to do a simple mass balance to calculate chromium VI and iron contribution by the aquifer in each interval. However, an approximate estimate of the contribution of each of the screens can be calculated by using the corrected flow in the blanks between the screened intervals. This results in approximately 5,600 of 8,300 µg/min (or 70%) of chromium VI produced by the upper screen. Assuming that groundwater at 420 feet depth contains chromium VI at a concentration of about half the detection limit or 0.5 µg/L, about 30% of chromium VI is produced by the middle screen and 10% by the lower screen (note that the assumed concentration at 420 feet depth may be an overestimate, considering that groundwater at 315 feet depth contained 0.68 µg/L). Mass balance calculations for iron show inconsistency between iron concentrations measured from below the top two screens, concentrations in the top two screens, and 86 µg/L measured at the well head, as that would require mass flux to decrease as flow goes up the well.

These results indicate that modifying the well so that water is not produced from the upper screen could reduce chromium VI concentrations while maintaining the majority of flow. Eliminating flow from the middle screen as well would reduce chromium VI concentrations further, but flow may be reduced in half or more. Producing a higher percentage from lower depths could increase iron concentrations based on the inconsistent data collected.

Manganese was non-detect at a detection limit of 10 μg/L for the well head sample and all eight depth-discrete samples in the well.

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SECTION 5
EVALUATE LOCAL PURISIMA FORMATION SUSTAINABLE YIELD: GROUNDWATER MODEL UPDATE SETUP (TASK 4.1)

This section documents the updates to the setup of CWD's groundwater model as a tool to evaluate the sustainable yield of the Purisima Formation for Task 4.1. CWD's groundwater model was updated to evaluate the strategy of shifting pumping from the Rob Roy well field to the Cox well field. The updates include conversion of model from steady-state to a transient simulation of Water Years 1984-2009 conditions, estimates of water use, records and estimates of pumping, areal recharge estimates from a watershed model, and changes to boundary conditions. The updated model was then calibrated to groundwater level data from Water Years 1984-2009 (Section 6, Task 4.1). The calibrated model was used simulate groundwater management alternatives to evaluate whether the strategy can sustainably meet CWD's water supply goals and broadly assess the potential environmental impacts of the strategy (Section 7, Task 4.5).

The section was distributed to the Technical Advisory Committee (TAC) for review as a draft technical memorandum on August 6, 2013.

5.1 MODEL BACKGROUND

CWD's groundwater model was developed for Drinking Water Source Assessments (DWSAP) to submit to the Department of Public Health (Johnson, 2009). It is a steady state model using the original MODFLOW code (McDonald and Harbaugh, 1988) and represents existing conditions for estimating capture zones of CWD's five production wells. Johnson (2009) described several areas for further model improvement:

1. Analysis of the simulated water budget
2. More thorough calibration
3. Transient simulations
4. Incorporation of the SEAWAT package so that the model accurately represents density gradients associated with subsurface saltwater and seawater.
5. Simulation of perched zones

To facilitate evaluation of the groundwater management strategy of shifting pumping from the Rob Roy well field to the Cox well field, the model updates address the first three areas of improvement. As documented in this section, the model is converted from steady-state to transient to evaluate the groundwater management strategy under different conditions that occur over time. The model is calibrated to historical groundwater levels so that it can be used as a predictive tool (Section 6). The simulated water budget is evaluated to document the relative importance of boundary conditions implemented in the model (Section 6). These last two improvement areas are discussed in other sections.

SEAWAT is not incorporated into the model. Solving salt transport and density dependence would be numerically intensive. SqCWD has established protective elevations at its coastal monitoring wells in the Aromas to protect the basin from seawater intrusion based on SEAWAT models (HydroMetrics WRI, 2012). These protective elevations will be included as numeric targets for the basin management objective to prevent seawater intrusion in planned updates to the GMP (HydroMetrics WRI, 2013a). Therefore, the protective elevations are used as numeric targets to evaluate seawater intrusion risk using CWD model simulations as has been done in the Seaside Basin (HydroMetrics LLC, 2009d).

Simulation of perched zones is also not incorporated into the model. The groundwater management strategy of shifting pumping being evaluated should not be affected by flow in perched zones. Using estimates of net recharge that reach the regional aquifer is the appropriate level of detail for the purpose of the model update. Furthermore, there are limited available data for simulating flow in perched zones.

The primary part of the model used for the DWSAP (Johnson, 2009) that is not changed for the update is the model layering representing the hydrostratigraphic structure. The model layering was based on the most recent conceptual model of hydrostratigraphy (Johnson, 2006 and Johnson et al., 2004) and there has not been enough new data collected to necessitate a reevaluation of the conceptual model. There are ten layers in the model with four hydrostratigraphic units represented (Table 5-1). For the evaluation of the strategy to shift pumping, it may have been justified to consolidate layers to speed up run times, but the ten layers were maintained in case there is a need to do transport runs that require the greater vertical resolution in the future.

The horizontal and vertical locations of boundary conditions were typically unchanged from the DWSAP model, but implementation of boundary conditions were changed based on available data and revised conceptualization of some boundaries.

Table 5-1. Model Layers for hydrostratigraphic Units

Hydrostratigraphic Unit	Model Layers
Upper Aromas	1-3
Lower Aromas	4-6
Purisima F	7-9
Purisima DEF	10

5.2 CONVERSION OF MODEL TO TRANSIENT

5.2.1 UPDATED MODEL CODE

In order to facilitate the conversion of the CWD model to simulate transient conditions, the model code was updated from original MODFLOW (McDonald and Harbaugh, 1988) to MODFLOW 2000-SSPA, a public domain code developed by SS Papadopulos & Associates, Inc. (SSPA, 2012) for the model calibration runs. The model can also be converted to the USGS code MODFLOW NWT (Niswonger et al., 2011).

One of the main limitations of the original MODFLOW is the difficulty it has in solving heads in cells that dry out and rewet. This occurs when groundwater levels fall below layer bottom elevations then rise back above bottom elevations. When simulating transient conditions with the ten layer CWD model, this occurs frequently. Both MODFLOW 2000-SSPA and MODFLOW NWT implements solutions to this issue with the help of a Newton-Raphson method.

Another feature provided by upgrading the MODFLOW version is to define stress periods as steady state or transient. Since the updated CWD model needs to simulate initial conditions, the first stress period is steady state and all subsequent stress periods are transient.

New MODFLOW packages have been developed that are not compatible with the original version. One such package is the multi-node well package, MNW2 (Konikow et al., 2009). This package internally apportions flow from a well to the multiple layers the well is screened across. MNW2 is incorporated into the updated CWD model for CWD and SqCWD municipal wells where pumping and screen intervals are well defined.

Updating the MODFLOW code also facilitates use of the stream routing package, SFR2 (Niswonger and Prudic, 2005). Implementation of this package is not strictly necessary for this model update due to the assumptions behind areal rainfall-recharge estimates used in the model. However, SFR2 is incorporated to facilitate possible future conversion to a GSFLOW model (Markstrom et al., 2008), which would numerically couple the precipitation-runoff modeling system (PRMS) watershed model developed for the area (HydroMetrics WRI, 2011) with the MODFLOW groundwater flow model.

5.2.2 Calibration Period

The calibration period for the updated model is November 1983-September 2009. This period starts with the first month of available SqCWD well pumping data and overlaps with the SqCWD Water Year 1984-2009 calibration period of the PRMS model used to provide estimates of rainfall recharge for input to the updated model. In addition, most of CWD and SqCWD's monitoring wells in the model domain came online during this period. Monthly stress periods are used because SqCWD and CWD record monthly pumping. Only eleven months in the water year 1984 are included in the calibration period, while full water years 1985-2009 are included.

Initial Conditions

Model initial conditions are based on average conditions prior to November 1984, as estimated based on available data for each boundary condition or model input. As stated above, the first stress period of the model is run to steady state using these average conditions to produce a stable set of initial conditions for the transient simulation.

5.3 Water Use Estimates

A number of model inputs result from calculations of water use for each parcel in the model. These model inputs include pumping for some small water systems, private well pumping, return flow recharge within CWD and SqCWD, and return flow recharge outside CWD and SqCWD.

5.3.1 Average Annual Water Use Based on Land Use Analysis

The calculations for average annual water use are based on an analysis of land use using geographic information system (GIS) software. Figure 5-1 shows the land use by parcel, which is primarily based on the Santa Cruz County land use dataset (Santa Cruz County, 2012c) overlain on County parcels (Santa Cruz County, 2012a). A current aerial photo (ESRI, 2012) was also used to identify parcels that were marked as agricultural by the County land use maps, but had not been farmed; most of these parcels had a residence on the parcel and were therefore designated as rural residential.

Agricultural areas on the County's land use maps were not designated by crop type or farming activity so California Department of Water Resources land use data (DWR, 1997) was used to add this information to agricultural parcels designated by the County land use dataset. Additionally, the current aerial photo (ESRI, 2012) and information from CWD (Bracamonte, 2013) was used to manually identify crop types where crop types may have changed since 1997. Figure 5-2 shows the agricultural crop type map.

Water use factors are annual water demand by parcel or by area based on land use or crop. Table 5-2 shows the water use factors in acre-feet per year applied for parcels with non-agricultural land use.

Table 5-2. Water Use Factors for Non-Agricultural Land Use

Land Use	AFY/parcel	Source
Residential/Accommodations		
Low-Medium Urban/Suburban	0.39	Faler (1992)
High Urban	0.43	Faler (1992) – average of SFR and duplex
Mountain/Rural	1.00	Faler (1992)
House on Agricultural Parcel	0.39	Same as suburban land use
Mobile Park	0.12	Faler (1992)
Commercial		
Public Facility/ Neighborhood/Community	1.00	Faler (1992)
Service	0.50	Estimate
Office	0.30	Estimate
Recreation/Open Space	**AFY/ acre**	Source
Parks and Recreation	1.00	Faler (1992), adjusted for 60% of parcel irrigated
Urban Open Space		
Golf Course	1.93	Faler (1992)

Agricultural water use is based on crop demand divided by an estimated crop efficiency of 80% (Johnson et al., 2004).

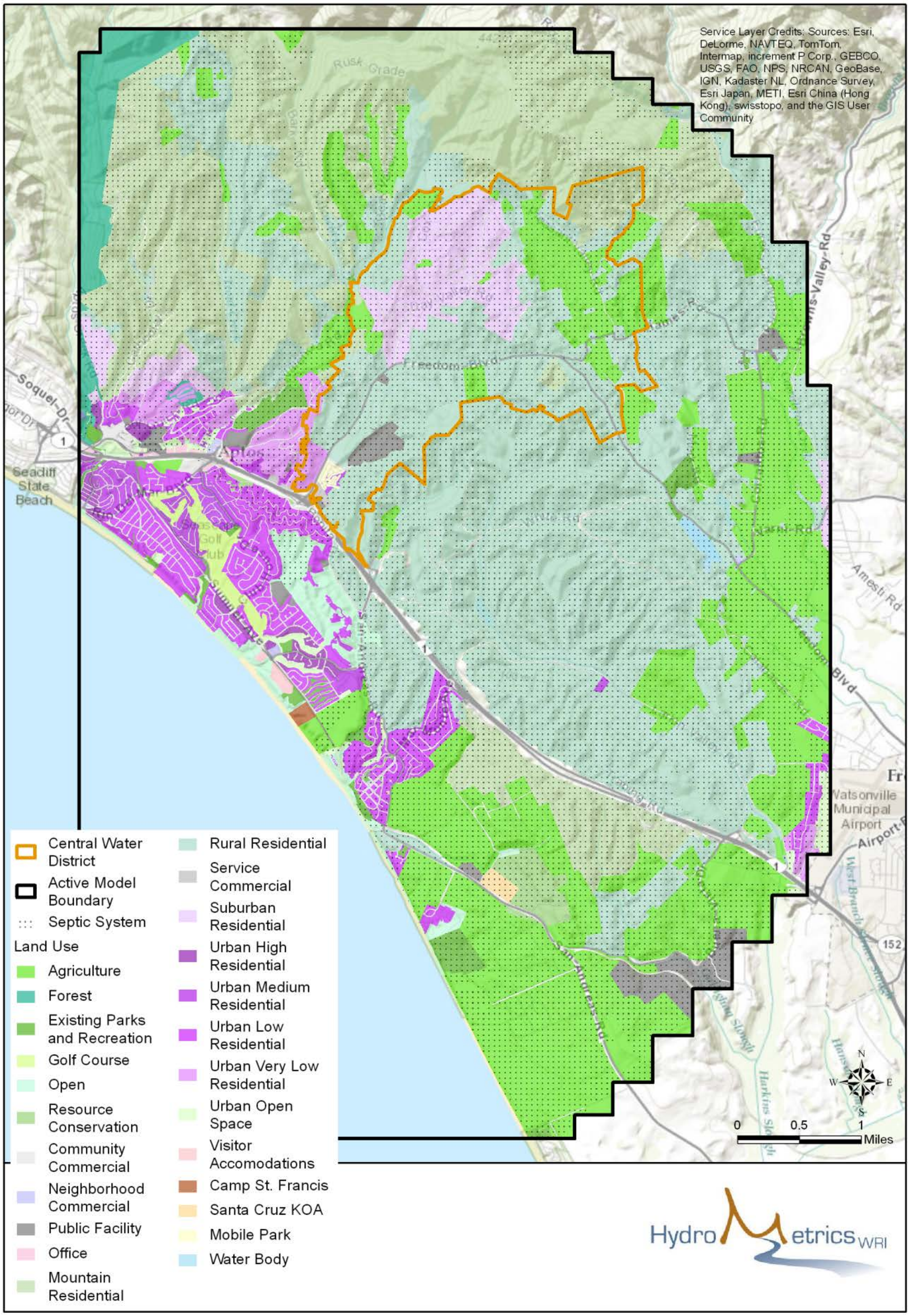

Figure 5-1. Land Use by Parcel

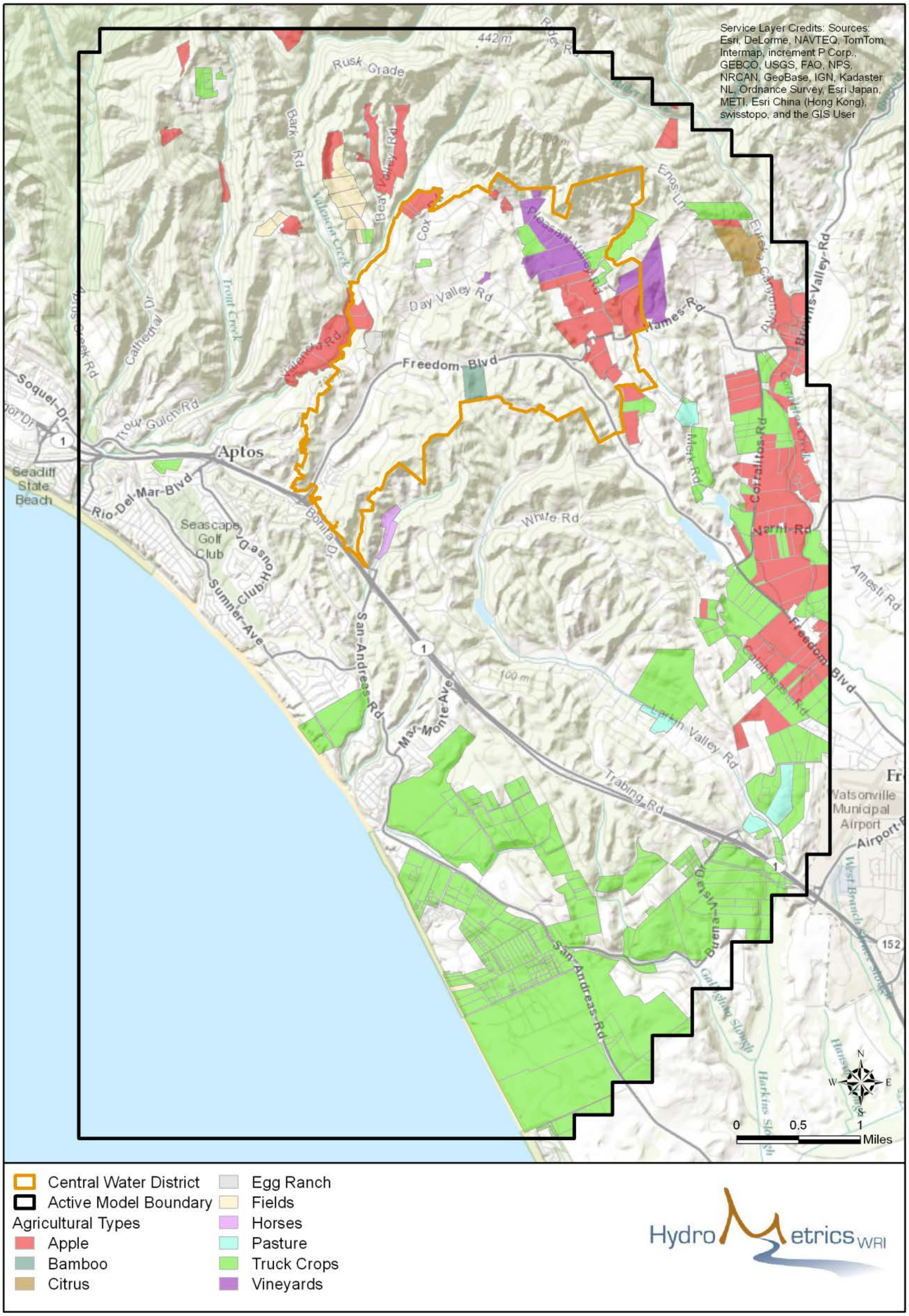

Figure 5-2. Agricultural Crop Types by Parcel

Table 5-3 shows the agricultural water use factors in acre-feet per year and the source of crop demand or water use.

Table 5-3. Water Use Factors for Agricultural Crop Type

Crop	AFY/ acre	Source
Truck	2.00	Faler (1992), San Andreas Mutual
Apple	0.23	CWD Usage 2010-11 (little water applied to established trees)
Vineyards	0.40	CWD Usage 2010-11 (little water applied to established vines)
Pasture	2.0	Faler (1992), adjusted for warm season only
Fields	1.71	Faler (1992)
Bamboo	0.43	CWD Usage 2010-11
Citrus	0.23	Same as apple
Other Ag	**AFY/parcel**	**Source**
Egg Ranch	2.70	CWD Usage 2010-11
Horses	3.00	Estimate

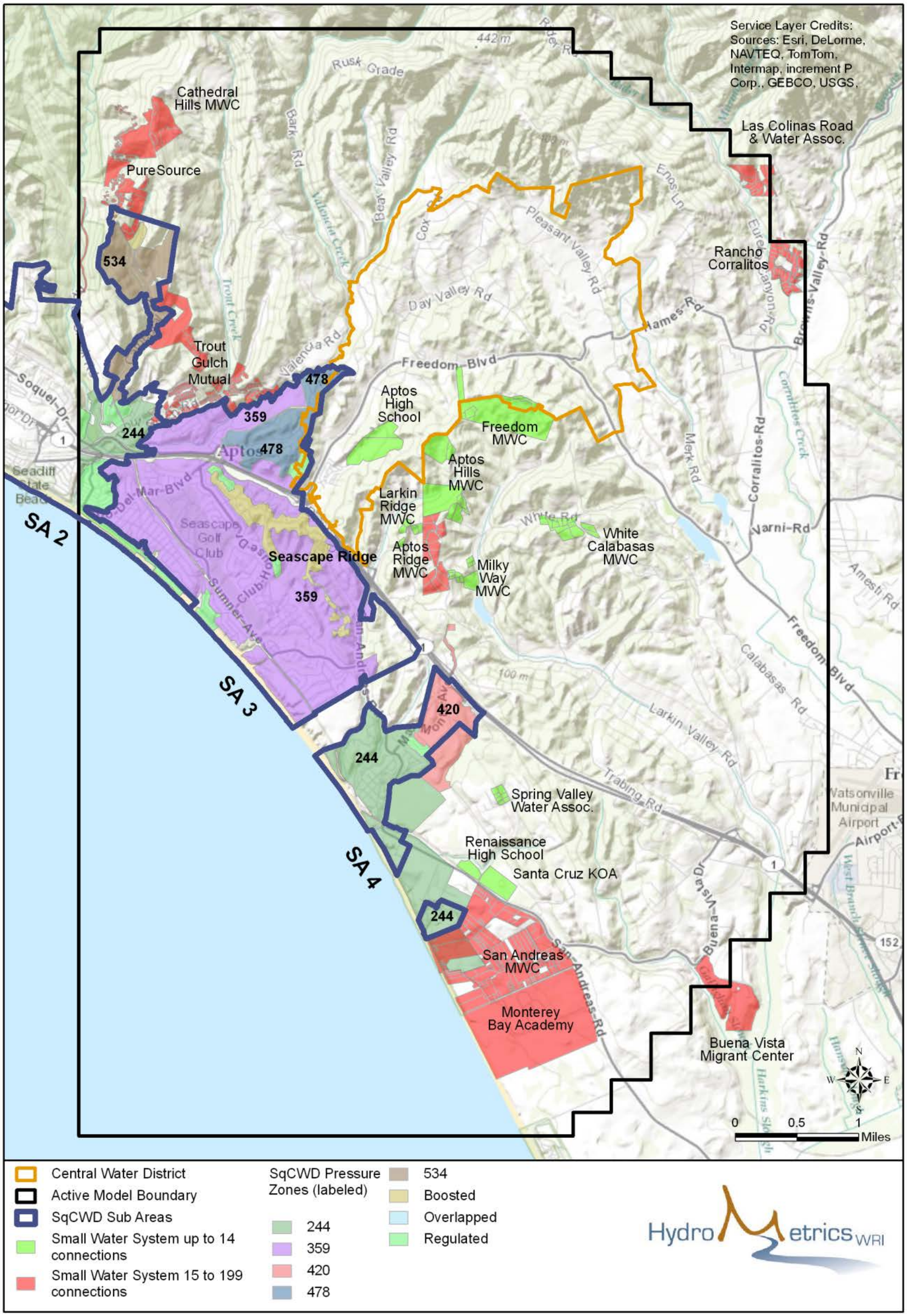

Figure 5-3. CWD, SqCWD Sub Areas and Pressure Zones, and Small Water Systems

5.3.2 WATER USE FOR CWD, SQCWD, AND SMALL WATER SYSTEM PARCELS

For parcels in CWD, SqCWD, and small water systems (Figure 5-3) with water production information, the parcel water use is adjusted to be consistent with total estimated deliveries (pumping minus estimated system losses) for CWD, SqCWD, and small water systems (also see Section 5.4 and Section 5.5.3). The estimated parcel water use is calculated based on land use and water use factors as discussed above. The estimated parcel water uses of all parcels receiving CWD water, all parcels within each SqCWD pressure zone, and all parcels receiving deliveries from each small water system are summed for an estimated total for CWD, each SqCWD pressure zone, and each small water system. The estimated water use for each individual parcel are divided by the estimated system (or pressure zone) total to estimate the percentage of total system (or pressure zone) water use allocated to each parcel. The parcel's average annual water use is the total average annual delivery for the parcel's system or pressure zone multiplied by the parcel's percentage of use.

For some agricultural parcels within the boundaries of CWD and the San Andreas Mutual Water Company, calculated water use is greater than recorded delivery to the parcel, so private pumping is assumed to make up the difference. The water use from this assumed private pumping is excluded from the water use adjustment discussed in the previous paragraph.

Average annual CWD delivery of 460 acre-feet per year is estimated based on average annual CWD pumping minus estimated average system loss percentage for Water Years 1984-2009 of 9.8%. This water year average was estimated based on unaccounted loss percentages recorded for each fiscal year (Central Water District, 2013) applied by month (Section 5.5.3).

Average annual SqCWD deliveries for each pressure zone (Table 5-4) are estimated based on average annual SqCWD pumping minus estimated system loss of 7% (Dufour, 2012). SqCWD provided data for 2012 for estimating the percentage of sub area deliveries used in each SqCWD pressure zone as shown in Table 5-4. Water delivered to Sub Area II is water pumped outside the CWD model domain.

Average annual deliveries for some small water systems are based on County estimates for small water system consumption (Ricker, 2012).

Table 5-4. Estimated Deliveries to SqCWD Pressure Zones

Sub Area	Pressure Zone	Based on Pumping	Percentage of Delivery	Estimated Annual Delivery (AFY)
II	244 ft Within Model	Sub Area II + Tannery II - Ledyard	12%	124
II	534 ft		20%	197
III	359 ft and Seascape Ridge	Sub Areas III + IV	87%	1,474
III	478 ft		2%	39
IV	244 ft and 420 ft		11%	181

5.3.3 MONTHLY WATER USE

The average annual water use calculated above was distributed to the monthly stress periods based on reported SqCWD monthly pumping, CWD deliveries based reported monthly pumping and fiscal year unaccounted water percentages (CWD, 2013, Section 5.5.3) and monthly CWD deliveries to different agricultural crop types. CWD and SqCWD deliveries and pumping over time reflect how water use within CWD and SqCWD changes over time. CWD and SqCWD deliveries and pumping are not directly associated with water use for parcels outside CWD and SqCWD. However, the monthly distribution of CWD and SqCWD each year does indicate how seasonal demand likely changed throughout the area each year. Monthly distribution of agricultural water use is assumed to depend on crop type. The available information on monthly distribution of crop water use is CWD monthly deliveries to different crop types for each month between June 2010 and May 2011 (Table 5-5).

For parcels within CWD and SqCWD, water use is distributed to monthly stress periods based on CWD deliveries and SqCWD pumping so that annual water use changes from year to year while maintaining the annual average calculated above for the whole calibration period. For parcels outside CWD and SqCWD, water use is distributed to monthly stress periods such that annual water use in each year is equivalent to the average annual water use calculated above. Average annual water use is multiplied by the factors summarized in Table 5-6 to calculate monthly water use.

Table 5-5. Monthly Distribution of CWD Deliveries to Crop Types June 2010-May 2011

Transaction Date	Month	Apples	Bamboo	Truck[1]	Vines
7/1/2010	June	2.0%	13.9%	23.8%	8.3%
8/2/2010	July	54.3%	12.9%	12.2%	12.9%
9/1/2010	August	31.7%	15.1%	17.8%	19.4%
10/4/2010	September	0.0%	12.1%	13.0%	16.4%
11/1/2010	October	9.0%	9.7%	7.2%	18.6%
12/1/2010	November	0.0%	6.0%	2.4%	9.7%
1/3/2011	December	0.0%	1.7%	0.4%	0.9%
2/1/2011	January	0.5%	3.0%	2.0%	1.5%
3/2/2011	February	0.0%	2.9%	4.0%	2.3%
4/4/2011	March	1.7%	2.8%	0.8%	0.8%
5/2/2011	April	0.5%	6.7%	5.8%	3.4%
6/1/2011	May	0.1%	13.1%	10.4%	5.8%

[1] Monthly distribution for truck crops applied to pasture and fields (Sudan)

Figure 5-4 shows estimated water use by land use, Figure 5-5 shows estimated water use by agricultural crop, and Figure 5-6 shows estimated water use by municipal water agency area.

5.3.4 Initial Conditions

For parcels in the CWD and SqCWD sub areas, water use for steady state initial conditions is estimated as the parcel's average annual water use multiplied by a factor based on annual deliveries calculated for pre-1984 conditions for each district. CWD annual pumping before 1984 is estimated as 304 acre-feet (Section 5.4.1) and the system loss is estimated as 14.3% based on data for 1984 and 1985. As a result, pre-1984 CWD deliveries are estimated as 261 acre-feet (Figure 5-6), 57% of the annual average for the calibration period of 460 acre-feet. SqCWD annual pumping pre-1984 in the Aromas (Sub Areas III and IV) are estimated as 1,011 acre-feet and deliveries estimated as 940 acre-feet based on system loss of 7%. Pre-1984 SqCWD deliveries are estimated as 54% of annual average deliveries based on the annual average for Sub Areas III and IV of 1,745 acre-feet. Including Sub Area II, pre-1984 SqCWD deliveries are estimated as 1,125 acre-feet (Figure 5-6).

For parcels outside the CWD and SqCWD service areas, model inputs for steady state initial conditions are based on the annual average water use for each parcel described above.

Table 5-6. Monthly Water Use Factors Based on Parcel Type

Land Use	Location	Factor Multiplied by Average Annual Water Use	Modeled Annual Water Use
Non-agricultural	CWD	Monthly CWD Deliveries/Average Annual CWD Deliveries	Variable
	SqCWD Sub Area II	Monthly Pumping/Average Annual Pumping for Sub Area II + Tannery II - Ledyard	Variable
	SqCWD Sub Areas III & IV	Monthly Pumping/Average Annual Pumping for Sub Areas III & IV	Variable
	Near CWD	Monthly CWD Deliveries /Yearly CWD Deliveries	Constant
	Near SqCWD Sub Area II	Monthly Pumping/Yearly Pumping for Sub Area II + Tannery II - Ledyard	Constant
	Near SqCWD Sub Areas III & IV	Monthly Pumping/Yearly Pumping for Sub Areas III & IV	Constant
Agricultural	CWD	Table 5-5 Monthly Percentage x Yearly CWD Deliveries /Average Annual CWD Deliveries	Variable
	SqCWD (Sub Areas III & IV only)	Table 5-5 Monthly Percentage x Yearly Sub Area Pumping/Average Annual Sub Area Pumping	Variable
	Near CWD	Table 5-5 Monthly Percentage	Constant
	Near SqCWD	Table 5-5 Monthly Percentage	Constant

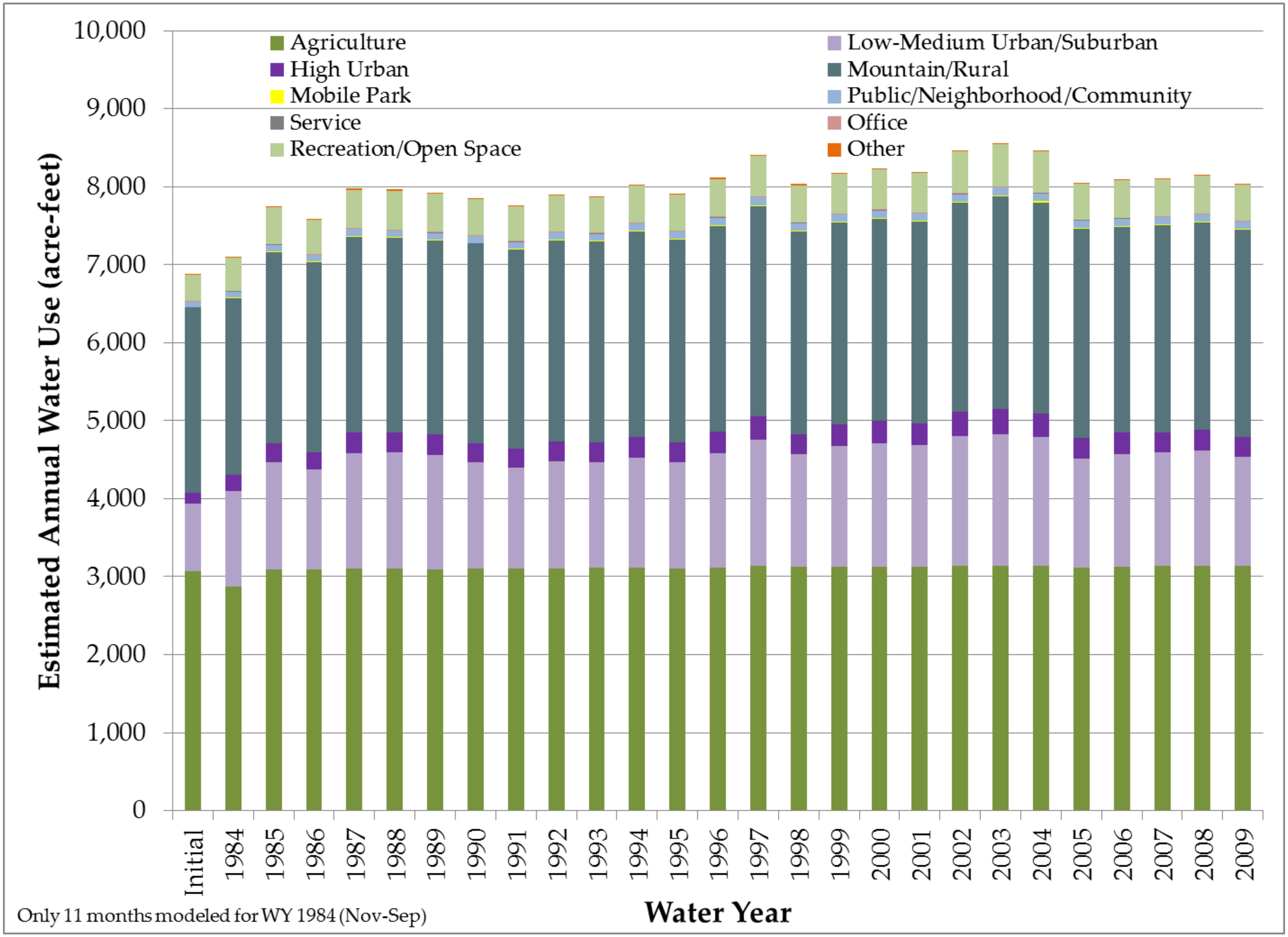

Figure 5-4. Annual Water Use Estimated by Land Use

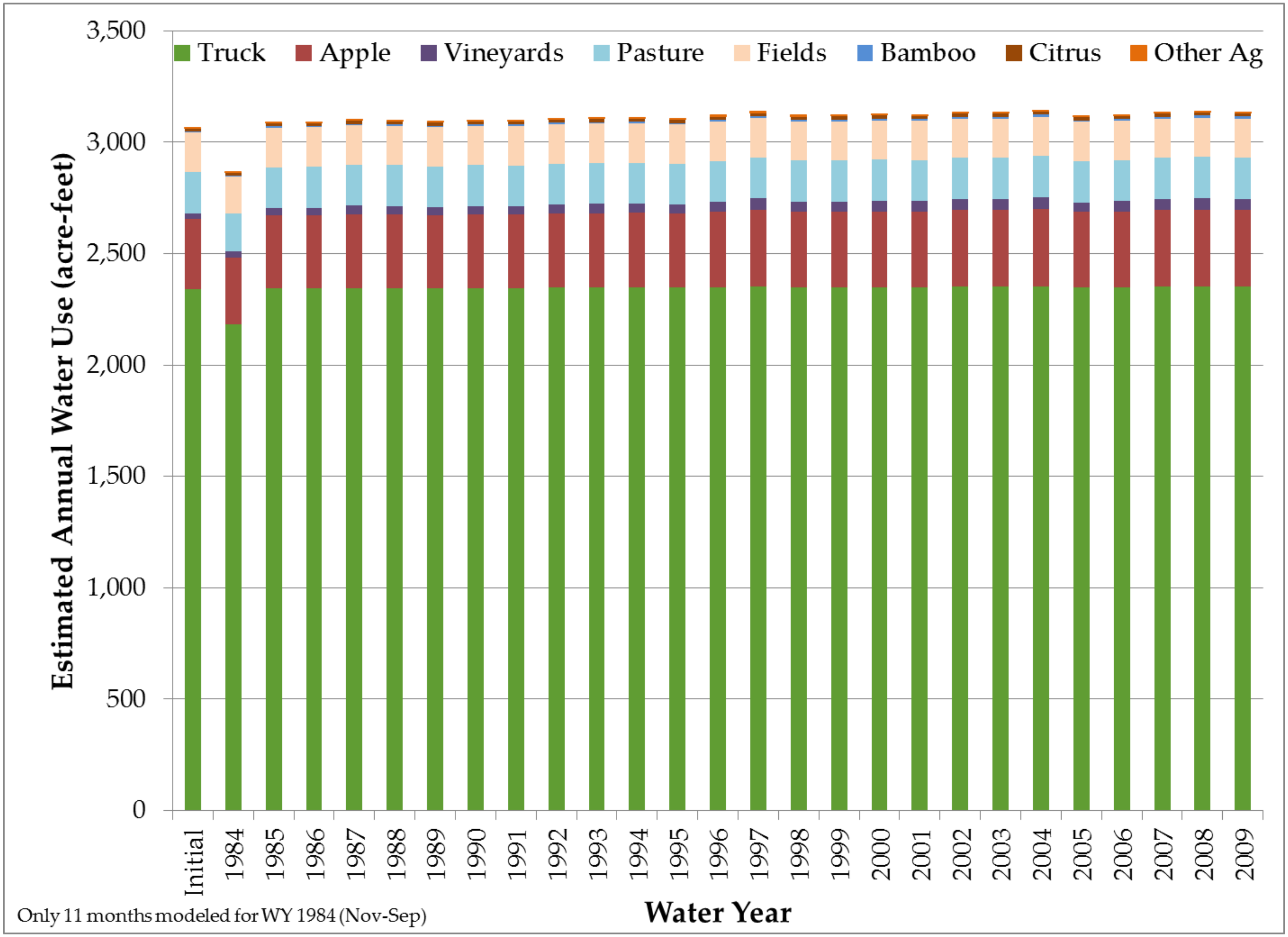

Figure 5-5. Annual Water Use Estimated by Agricultural Crop

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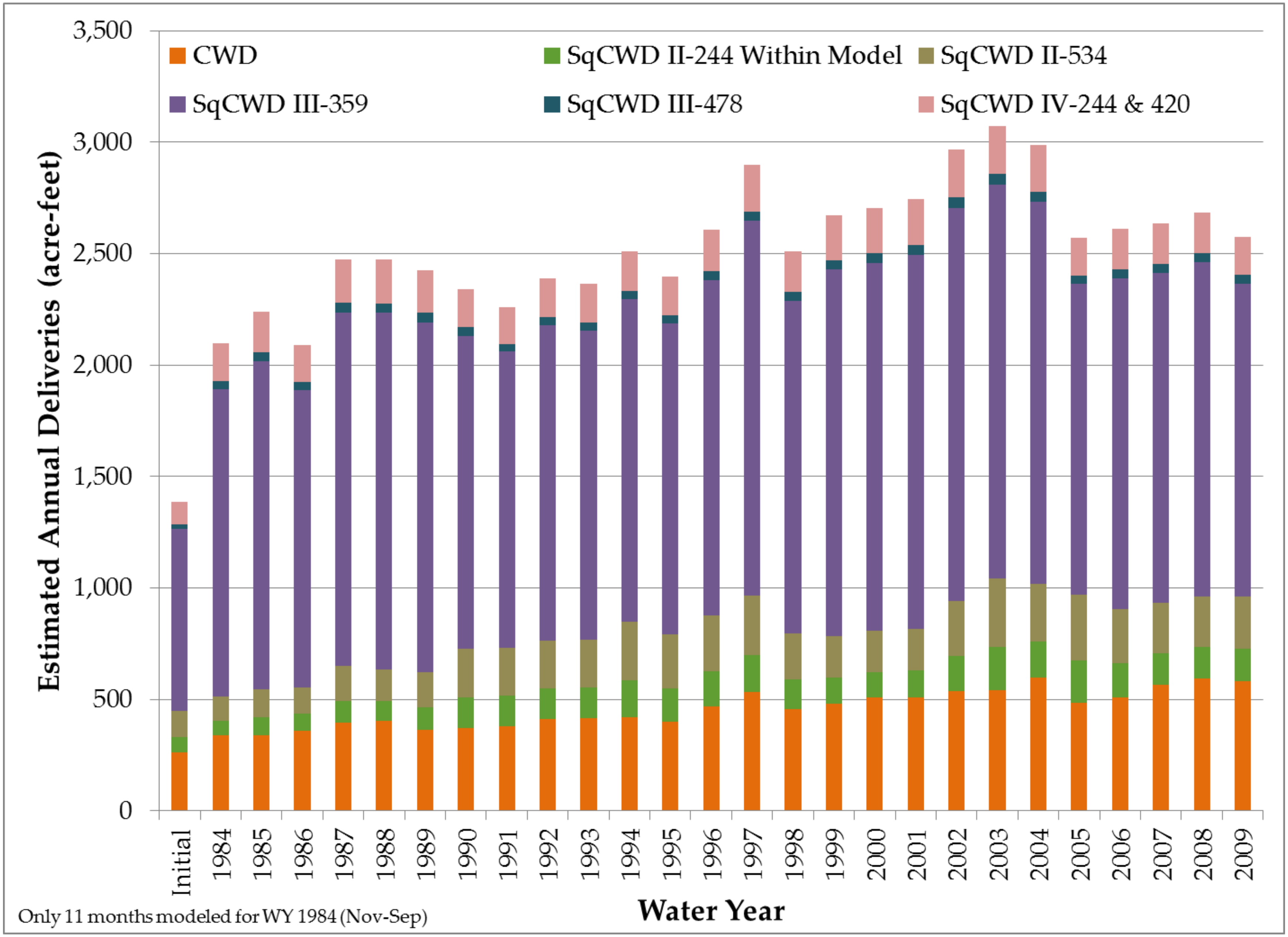

Figure 5-6. Annual Water Use within CWD and SqCWD Pressure Zones

5.4 PUMPING

Pumping is simulated in the model at the two municipal water agencies CWD and SqCWD, small water systems, and private wells. The locations of the municipal wells are known, while the locations of private wells and small water system wells and are estimated at the centers of the private parcels and small systems (Figure 5-7). Pumping quantities at municipal wells have been recorded, private pumping was estimated by an analysis of land use, and some of the small water system pumping was estimated by Santa Cruz County.

5.4.1 CWD PUMPING

During the model calibration period, CWD has pumped from six production wells, wells #2, #3, and #5 at the Cox well field, and wells #4, #10, and #12 at the Rob Roy well field. Monthly pumping quantities for each of the six wells have been recorded by CWD and the quantities for November 1983-September 2009 used as input to the model. Figure 5-8 shows the CWD pumping used for each water year of the calibration period. Cox #2 went offline in 1987 and Rob Roy #12 came online in 1999 at a separate site from Rob Roy #4 and #10 (Figure 5-7).

Pumping rates representing initial conditions are based on the annual average pumping for water years 1974-1983, the full water years with data available since CWD began recording monthly pumping at its wells starting in July 1973. Figure 5-8 shows the pre-1984 pumping used as initial conditions.

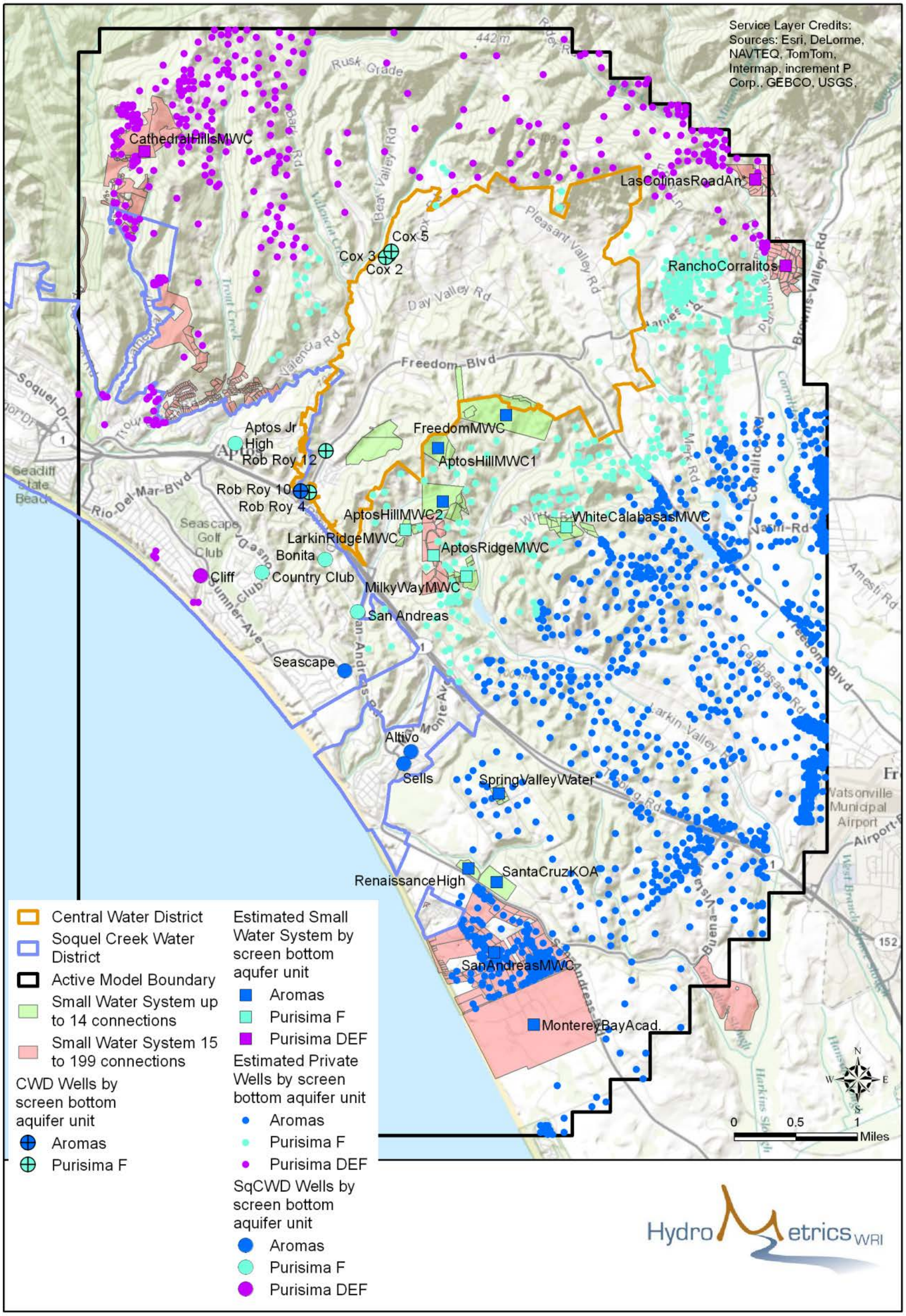

Figure 5-7. Modeled Locations of Municipal, Small Water System, and Private Wells

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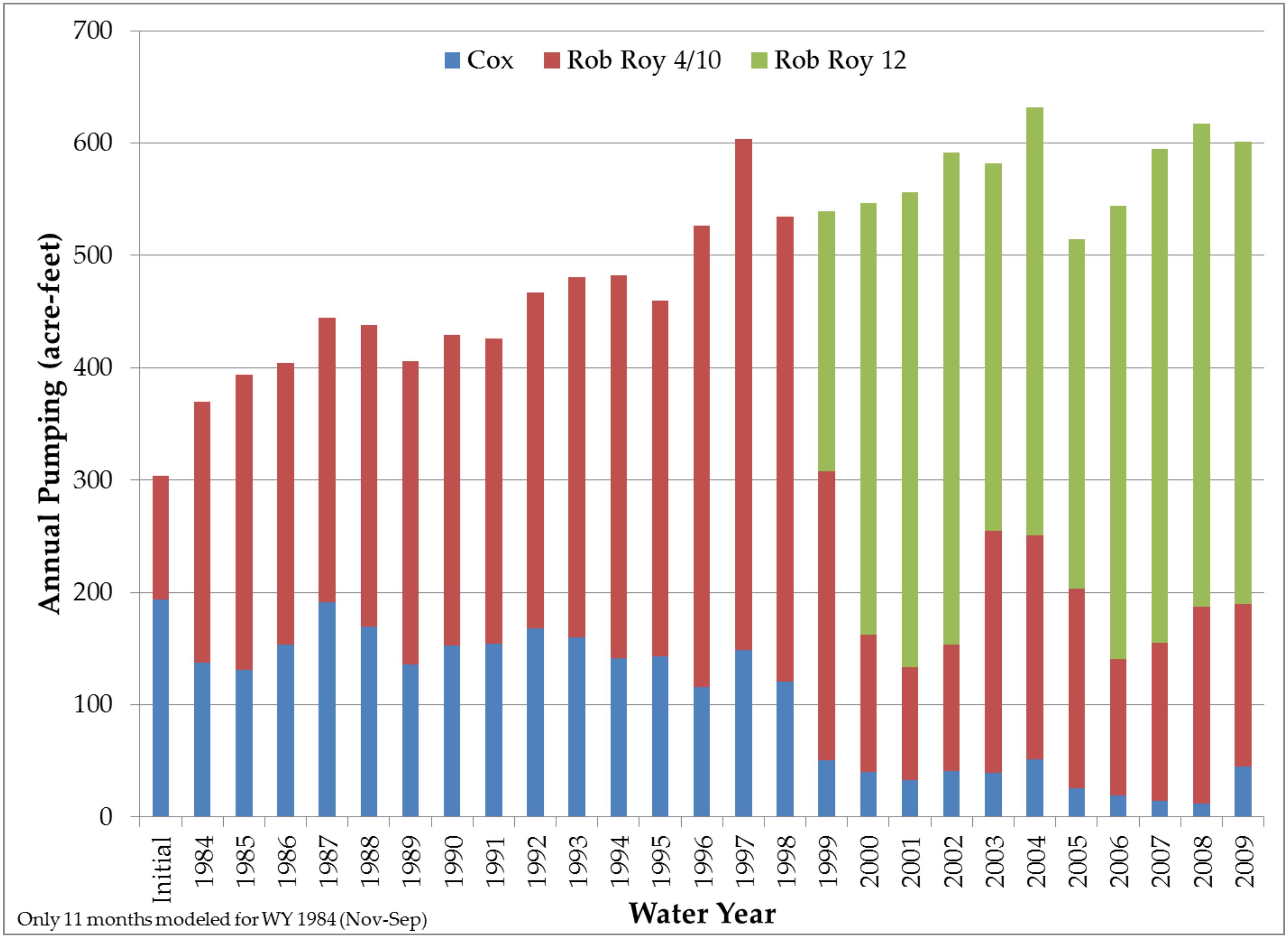

Figure 5-8. Initial Conditions and Annual Pumping Modeled at CWD Wells

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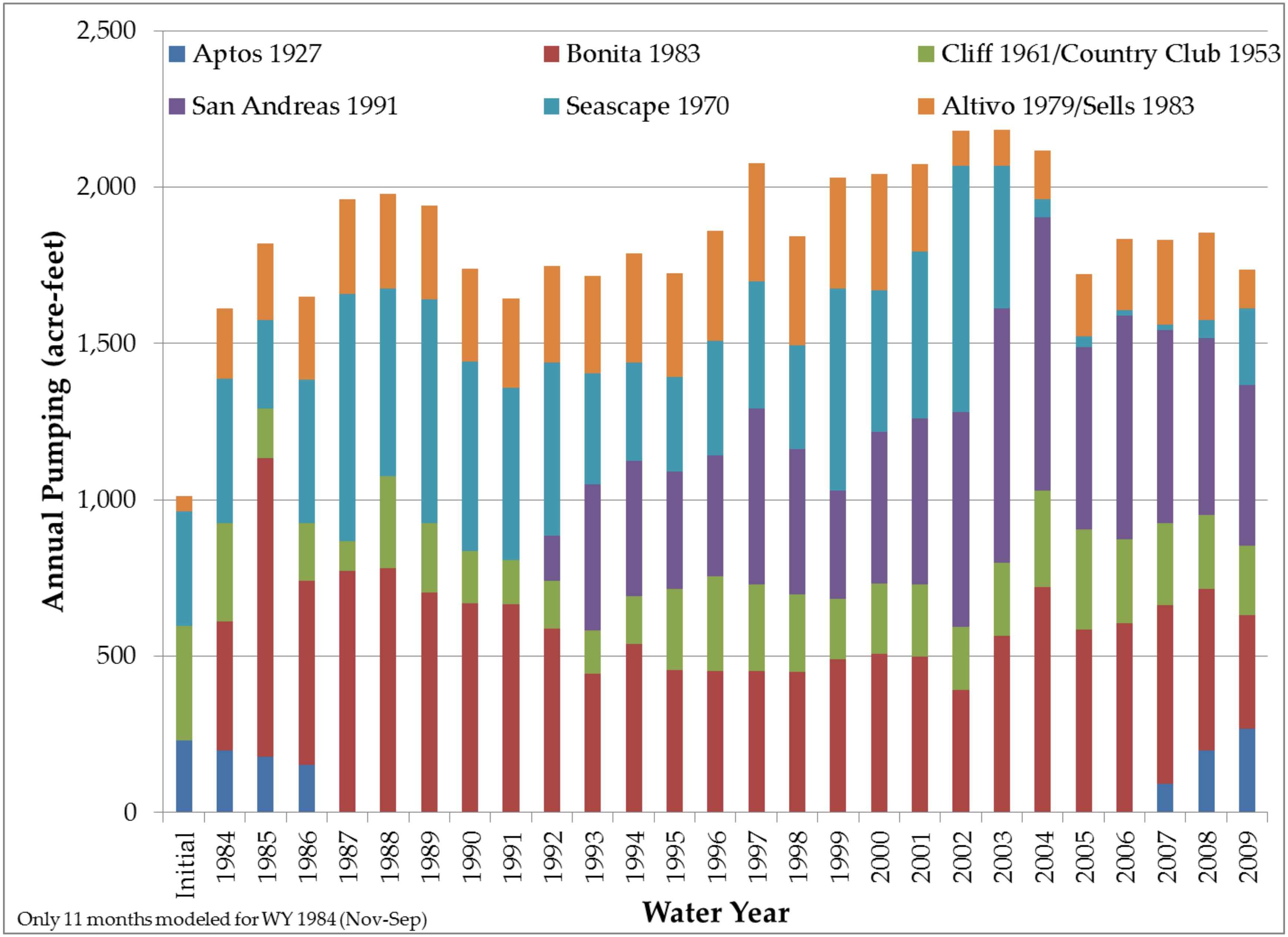

Figure 5-9. Initial Conditions and Annual Pumping Modeled at SqCWD Wells

5.4.2 SQCWD PUMPING

SqCWD has pumped from eight wells in the model domain. Monthly pumping data for individual wells have been recorded by SqCWD from November 1983 and the quantities for November 1983-September 2009 used as input to the model. Figure 5-9 shows the CWD pumping used for each water year of the calibration period. The Cliff well went offline in 1985 and the Sells well went offline in 2009. The Aptos Jr. High well was offline from 1987 into 2007.

With individual well pumping data unavailable prior to November 1983, pumping for the initial conditions is based on the annual average for the Aromas from 1966-1983 of 1,011 acre-feet per year. The initial conditions pumping for each well was based on the proportion of Aromas pumping recorded for the well in Water Year 1984 and adjusted for the number of years the well was online between 1966 and 1983.

5.4.3 MULTI-LAYER MUNICIPAL PUMPING

CWD and SqCWD pumping is implemented in the multi-node well MNW2 package (Konikow et al., 2009). This package apportions flows to the layers across which the well is screened. For the CWD model, the apportionment is based on the Thiem assumption and is primarily dependent on aquifer transmissivity, model cell size, and well radius, and ignores well skin effects and turbulent flow near the well. In addition, full penetration of each layer is assumed for numerical stability. Table 5-7 shows the layers and well radii simulated for each municipal well screen.

Table 5-7. Municipal Well Parameters for Multi-Node Well Package

Well	Top Elevation (feet msl)	Bottom Elevation (feet msl)	Model Layers	Well Diameter (inches)
Cox #2	195	55	7	8
Cox #3	155	7.6	7-8	12
Cox #5	121.6	106.6	7	12
	94	59.5	7	8
Rob Roy #4	6	-204	6-8	18
Rob Roy #10	-4	-24	6	12
	-64	-84	6	12
	-99	-109	6	12
Rob Roy #12	-58	-108	6	12
	-128	-188	7-8	12
	-308	-358	9	12
Aptos Jr. High	-60	-197	8-9	12
Cliff	-130.75	-330.75	8-10	12
Country Club	-56.87	-104.87	6	12
	-128.87	-152.87	7	12
	-176.87	-200.87	7-8	12
	-224.87	-297.87	8	12
Bonita	-103.83	-137.83	6	16
	-153.83	-183.83	6-7	16
	-216.83	-259.83	7	12.75
	-292.83	-316.83	7-8	12.75
	-334.83	-354.83	8	12.75
	-374.83	-408.83	8-9	12.75
	-440.83	-496.83	9	12.75
San Andreas	-105.5	-185.5	6	16
	-231.5	-291.5	6-7	16
	-349.5	-449.5	7-8	16
Seascape	-141.46	-166.46	6	16
	-197.46	-217.46	6	16
	-241.46	-272.46	6	16
Sells	-89.48	-149.48	5	16
	-190.48	-230.48	5-6	12.75
	-300.48	-320.48	6	12.75
Altivo	-154.44	-184.44	5	8
	-219.44	-259.44	5-6	8
	-274.44	-304.44	6	8

5.4.4 SMALL WATER SYSTEM PUMPING

There are a number of small water systems in the CWD model area (Figure 5-10, Santa Cruz County, 2012e). In 2009-2010, Santa Cruz County compiled estimates for consumption (Ricker, 2012). Pumping by the small water systems input to the model is based on the consumption estimates by the County where available. For systems for which the County did not estimate pumping, pumping is based on the parcel land use analysis and water use factors discussed above. Unlike municipal pumping, the small water system pumping estimates assume no system loss.

Although annual pumping is assumed to be constant for all small water systems, the monthly distribution changes from year to year based on pumping in nearby CWD or SqCWD sub area, as shown in Table 5-8 and discussed above for water use.

Small water system pumping is simulated using the MODFLOW WEL package at the center of the system and applied to the layer corresponding to the low end of elevation ranges for well completions mapped by Johnson (2006). The lowest layer is used because the MODFLOW WEL package automatically shuts off wells in layers that dry out; this is more likely to occur in shallower layers.

5.4.5 PRIVATE PUMPING

Private pumping is based on the water use for parcels not served by CWD, SqCWD, or small water systems, as estimated based on the parcel land use analysis and water use factors discussed above. Private pumping is simulated using the MODFLOW WEL package at the center of the system (Figure 5-7) and applied to the layer corresponding to the low end of elevation ranges for well completions mapped by Johnson (2006) or top active layer if the low end is no-flow in the model. The lowest layer is used to limit wells shutting off in layers that dry out.

Table 5-8. Model Input for Small Water System Pumping

System Name	Model Input Annual Pumping (AFY)	Source	Monthly Distribution Based on	Modeled Layer (Aquifer)
Aptos High School	0	Parcel Land Use	CWD	Negligible pumping estimated
Aptos Hills	6	County Estimate	SqCWD III	6 (Aromas)
Aptos Ridge	10	County Estimate	SqCWD IV	7 (Purisima F)
Buena Vista Migrant Center	21	County Estimate	SqCWD II	Assumed well location outside of model
Cathedral Hills	6	County Estimate	CWD	10 (Purisima DEF)
Freedom	4	County Estimate	SqCWD III	5 (Aromas)
Larkin Ridge	4	Parcel Land Use	CWD	8 (Purisima F)
Las Colinas Road and Water	11	Parcel Land Use	SqCWD III	10 (North of Zayante)
Milky Way	2	County Estimate	SqCWD IV	7 (Purisima F)
Monterey Bay Academy	444	County Estimate	SqCWD II	6 (Aromas)
PureSource Water Inc.	40	County Estimate	CWD	10 (Purisima DEF)
Rancho Corralitos	28	Parcel Land Use	SqCWD IV	10 (North of Zayante)
Renaissance High School	1	Parcel Land Use	SqCWD IV	6 (Aromas)
San Andreas	58	County Estimate	SqCWD IV	6 (Aromas)
Santa Cruz KOA	10	Parcel Land Use	SqCWD IV	6 (Aromas)
Spring Valley	9	County Estimate	CWD	5 (Aromas)
White Calabasas	3	County Estimate	CWD	7 (Purisima DEF)

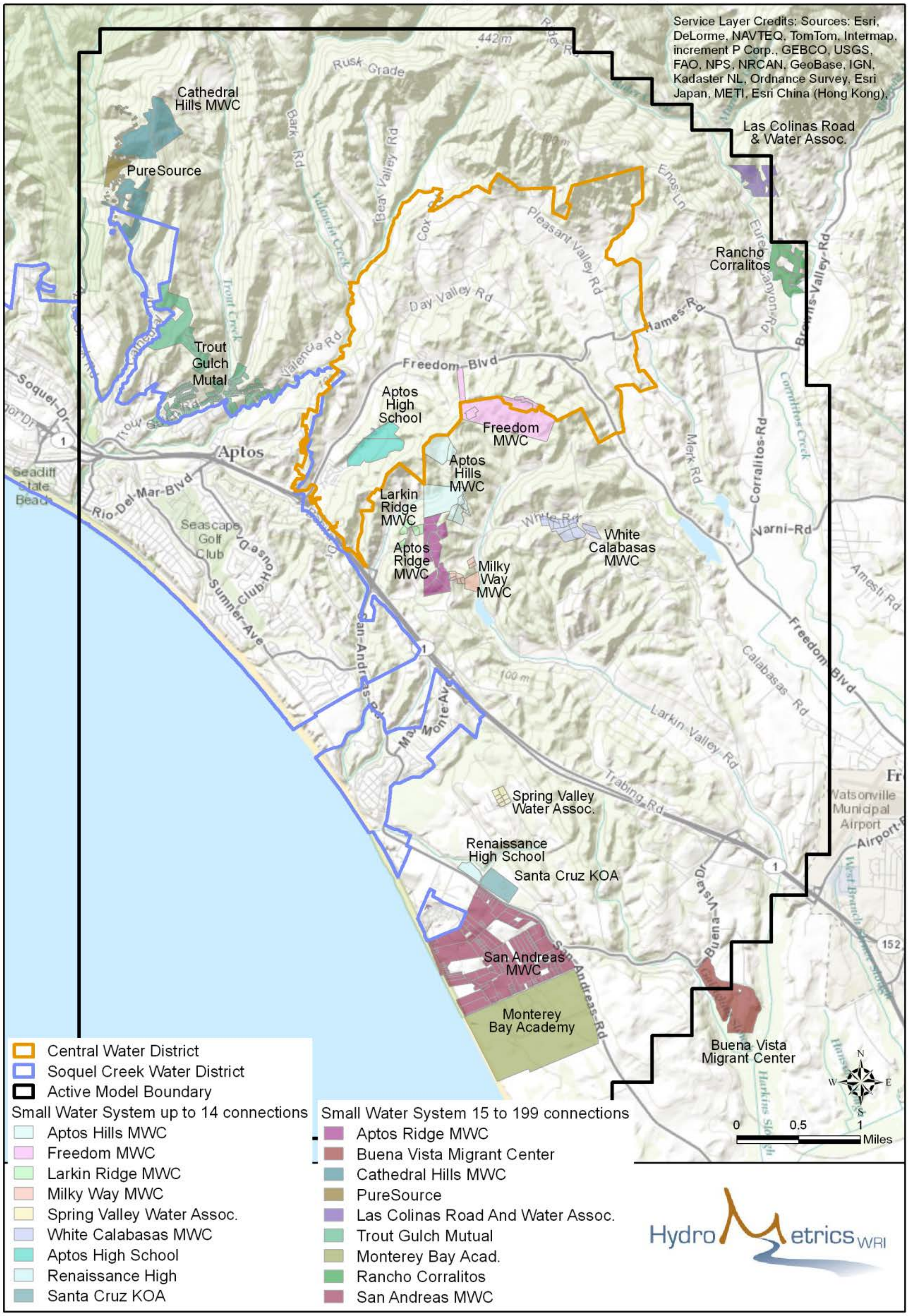

Figure 5-10. Small Water Systems in CWD Model Area

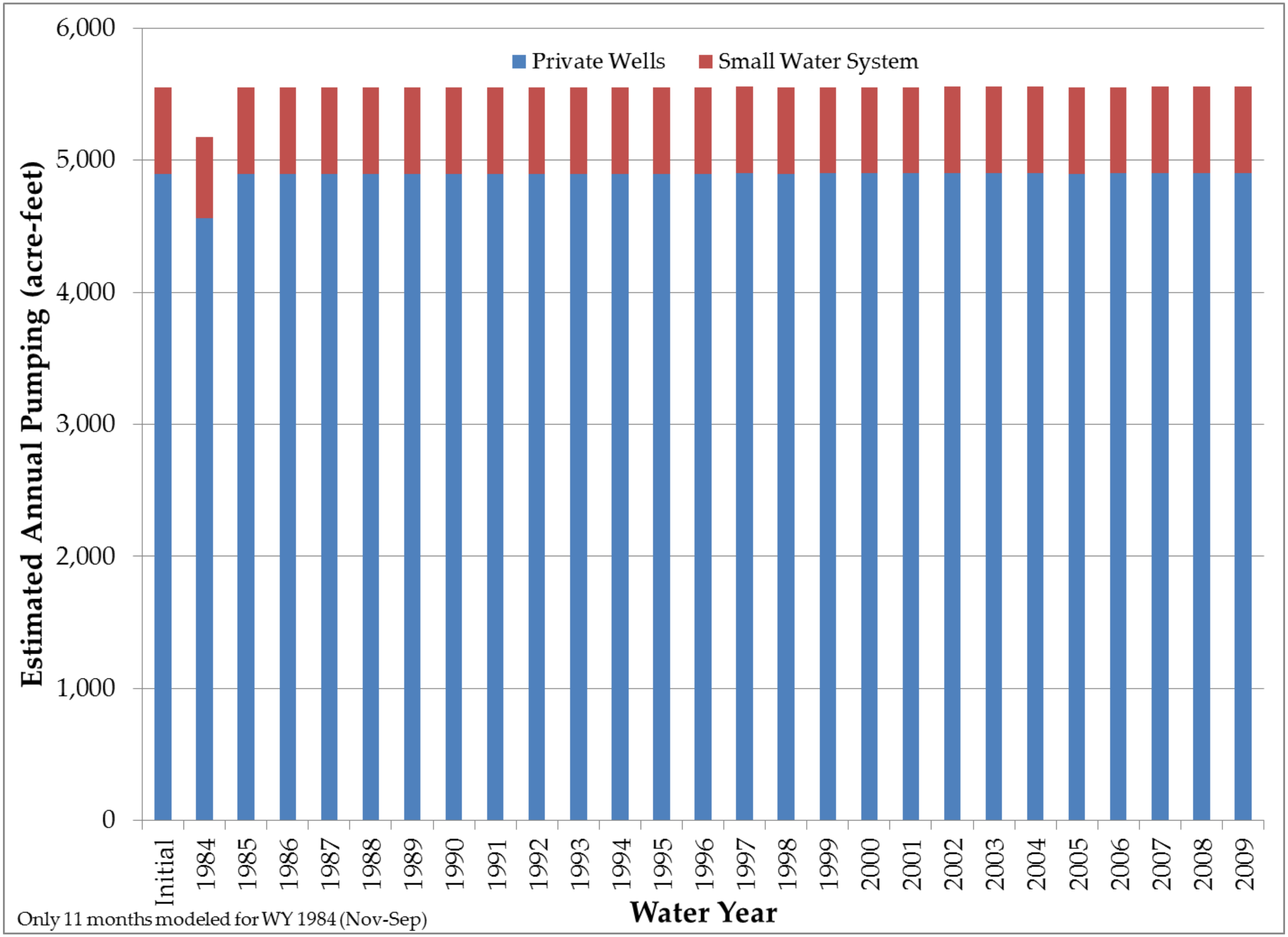

Figure 5-11. Initial Conditions and Annual Pumping Modeled at Small Water System and Private Well

5.5 AREAL RECHARGE

There are three main components to areal recharge included in the model: recharge from rainfall, return flow from water use, and system losses. The components are estimated for each monthly stress period from November 1983 to September 2009 throughout the model domain and combined in the MODFLOW recharge package, RCH. In the CWD model, the recharge package is set up to add the flow to the uppermost active layer.

5.5.1 RAINFALL-RECHARGE FROM PRMS MODEL

CWD, SqCWD, and the City of Santa Cruz funded a watershed model (HydroMetrics WRI, 2011) using the Precipitation-Runoff Modeling System (PRMS, USGS, 2011) to estimate deep groundwater recharge from rainfall for the Soquel-Aptos area. The model is a distributed-parameter, physically based hydrologic model that uses precipitation and temperature data to calculate runoff, evapotranspiration, and deep groundwater recharge.

The model area contains the Aptos Creek and Valencia Creek watersheds, as well as portions of the Corralitos Creek and Branciforte Creek watersheds in CWD's area, and overlaps most of the domain of the CWD groundwater model. Figure 5-12 shows the Hydrologic Response Units (HRUs), which are subwatersheds that make up the spatial discretization of the PRMS model. The PRMS model domain does not overlap the southeastern portion of the CWD groundwater model domain so southeastern HRUs are extended to cover that southeastern portion of the CWD model domain. Recharge from the extended HRUs are used in that southeastern portion. Recharge calculated for HRUs are translated to the CWD model grid based on areal overlap.

This PRMS model was calibrated for Water Years 1984-2009. Monthly recharge was calculated from the daily PRMS model results for input into the updated CWD groundwater model. Figure 5-13 shows the conceptual water balance for a HRU in PRMS. The recharge input into the CWD model is based on the groundwater recharge (PRMS variables gw_in_soil + gw_in_ssr) that percolates through the soil-zone reservoir to the groundwater reservoir (blue box on bottom) minus groundwater flow to streams (PRMS variable gwres_flow) out of the groundwater reservoir.

Figure 5-13 also shows upslope flow to the groundwater reservoir and groundwater flow to the downgradient groundwater reservoir. PRMS estimates these flows between HRUs as the groundwater flows between HRUs that interact with streams. These

PRMS groundwater flow results are omitted from the recharge input to the updated CWD model for two reasons:

1. Previous evaluation of stream-aquifer interaction showed that there is a vertical separation between Valencia Creek and regional groundwater levels near the Cox well field (HydroMetrics LLC, 2009e). Based on surface elevations and groundwater level data, this separation is expected to exist across much of the CWD model domain. Therefore, there should not be groundwater flows between HRUs that interact with streams. The PRMS model is consistent with this finding as it calculates these groundwater flows in the CWD model domain as negligible.

2. PRMS does not calculate groundwater flow by solving groundwater flow equations like MODFLOW. Omitting PRMS calculations of groundwater flow between HRUs prevents overlap of calculations.

Figure 5-12 also shows how HRUs are grouped into subbasins. HydroMetrics WRI (2011) defined subbasins for the PRMS model with stream gauges on their downstream end. The PRMS model was calibrated to streamflow data from these gauges. Based on the calculation for recharge input discussed above, rainfall percolation through the soil zone in these gauged subbasins that does not result in streamflow at the downstream gauge is used in the updated CWD model as areal recharge for the subbasin. To organize water balance in this report, we defined ungauged subbasins as well. Areal recharge estimated by PRMS in these ungauged subbasins cannot be calibrated to streamflow.

Since there are no calibrated PRMS results prior to Water Year 1984, the annual average of recharge for the PRMS calibration period from Water Year 1984-2009 is used in the steady state stress period to determine the initial conditions for the transient simulation

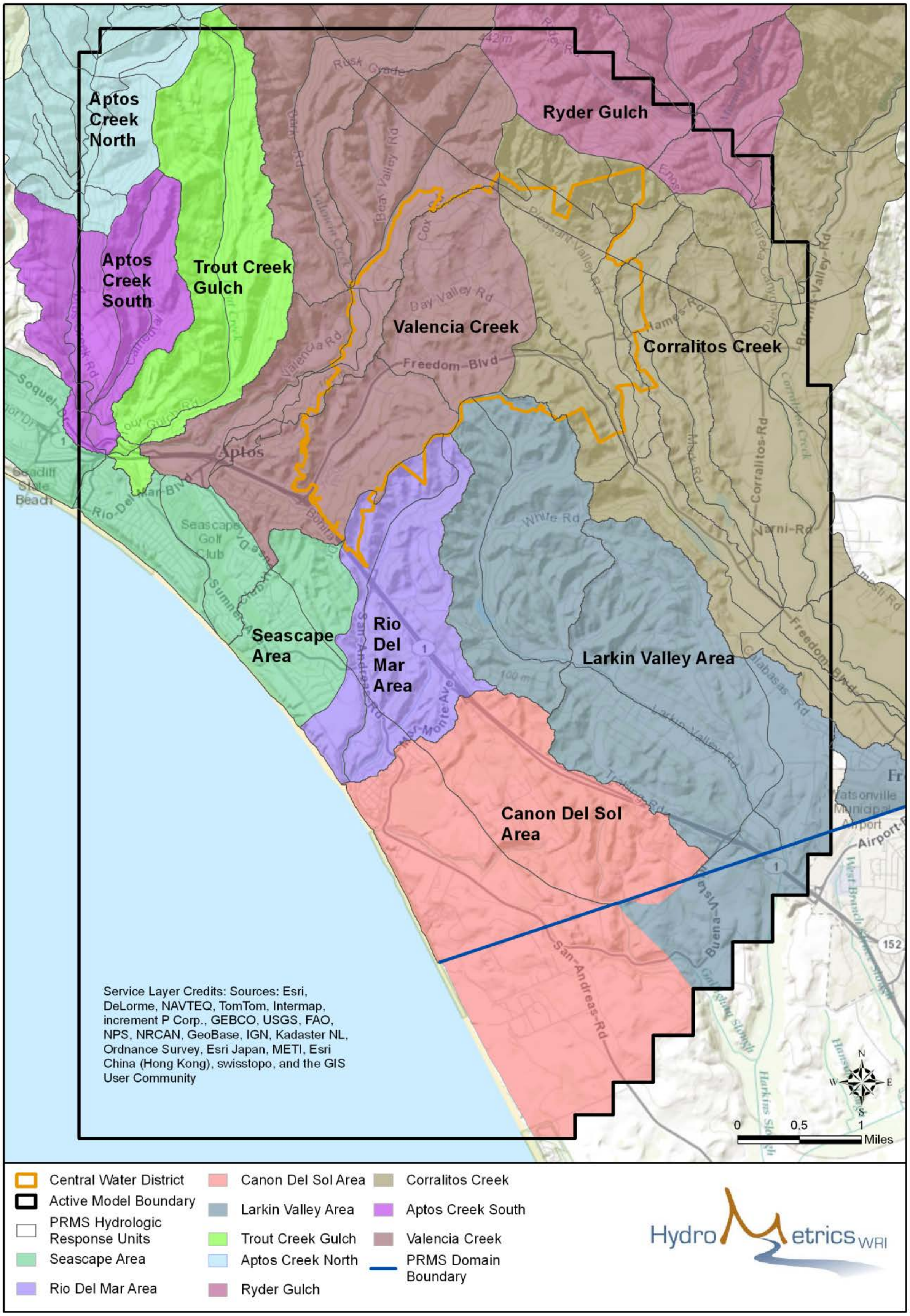

Figure 5-12. PRMS Hydrologic Response Units and Subbasins

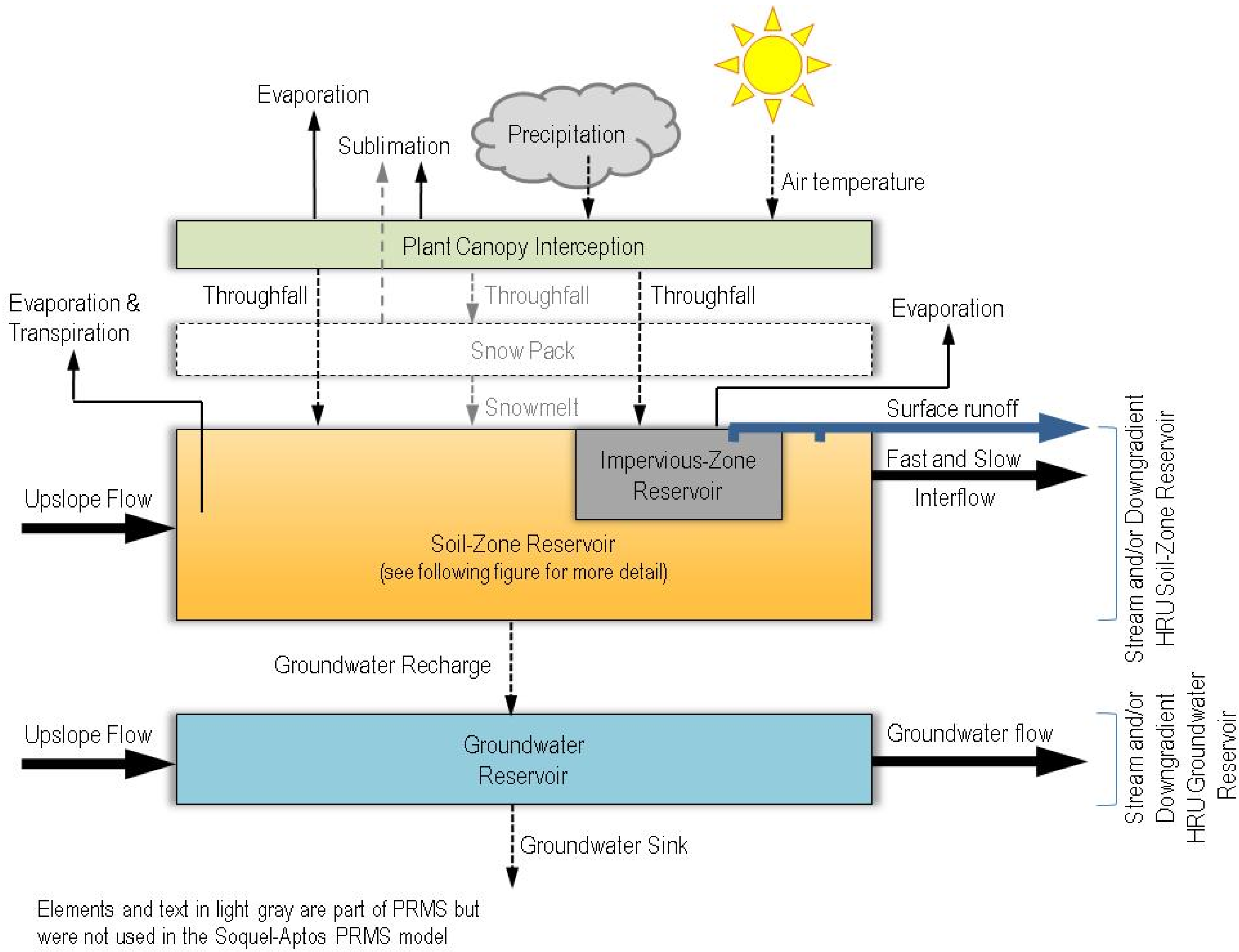

Figure 5-13. Overview of PRMS Conceptualization of HRU Components and Fluxes (HydroMetrics WRI, 2011)

5.5.2 RETURN FLOW

Return flow recharge is the portion of applied water such as irrigation that passes below the root zone and reaches the regional groundwater system. Since return flow is not calculated by the PRMS model, return flow is calculated as a proportion of the parcel water use calculated for each month based on land use, crop type, water use factors, and CWD, SqCWD, and small water system pumping as discussed above. Return flow is calculated based on the estimates of Johnson et al. (2004) which apportion total water use into indoor and outdoor usage by land use (Table 5-9) and the percentage of water use that becomes return flow for indoor, outdoor, and agricultural use. Indoor use return flow depends on whether the parcel is on sewer or septic (Santa Cruz County, 2012d). Agricultural return flow is 20%, which is consistent with the 80% efficiency assumed in calculating water use from crop demand.

To identify parcels with septic systems, the County septic system datasets (Santa Cruz County, 2012d) were used together with visual identification of residences on recent aerial photographs that are not served by the County's sewer system.

Table 5-9. Indoor and Outdoor Use Percentages by Land Use

Land Use	Indoor/ Outdoor Use
Agricultural	0%/100%
Residential/Accommodations	
Low-Medium Urban/Suburban	70%/30%
High Urban	70%/30%
Mountain/Rural	50%/50%
House on Agricultural Parcel	70%/30%
Mobile Park	50%/50%
Visitor Accommodations	70%/30%
Commercial	
Public Facility	70%/30%
Service	70%/30%
Neighborhood/Community	70%/30%

Table 5-10. Return Flow Percentages

Water Use	Sewer/Septic	Return Flow Percentage
Indoor	Septic	75%
Indoor	Sewer	0%
Outdoor Non-Agricultural		20%
Agricultural		20%

5.5.3 SYSTEM LOSSES

System loss recharge is based on CWD deliveries and estimated SqCWD deliveries to pressure zones (Table 5-4). Modeled CWD system loss varies over time based on unaccounted water losses by fiscal year which runs from July to June (CWD, 2013). Monthly CWD system loss is estimated by multiplying monthly CWD pumping by the current fiscal year's percentage of unaccounted water loss.

SqCWD system loss is estimated as 7% of estimated SqCWD pumping for each pressure zone (Dufour, 2012). Sewer system loss is estimated based on the SqCWD system loss of 7%.

CWD system loss is distributed to the model grid based on areal intersection with CWD. SqCWD system loss is distributed to model grid based on the linear intersection of the distribution system for each pressure zone (SqCWD, 2012) with the model grid (Figure 5-15). Sewer system loss is also distributed to the model grid based on the linear intersection of the system (Santa Cruz County, 2012b) with the model grid (Figure 5-16).

Figure 5-17 shows modeled recharge for initial conditions and each water year in the calibration period.

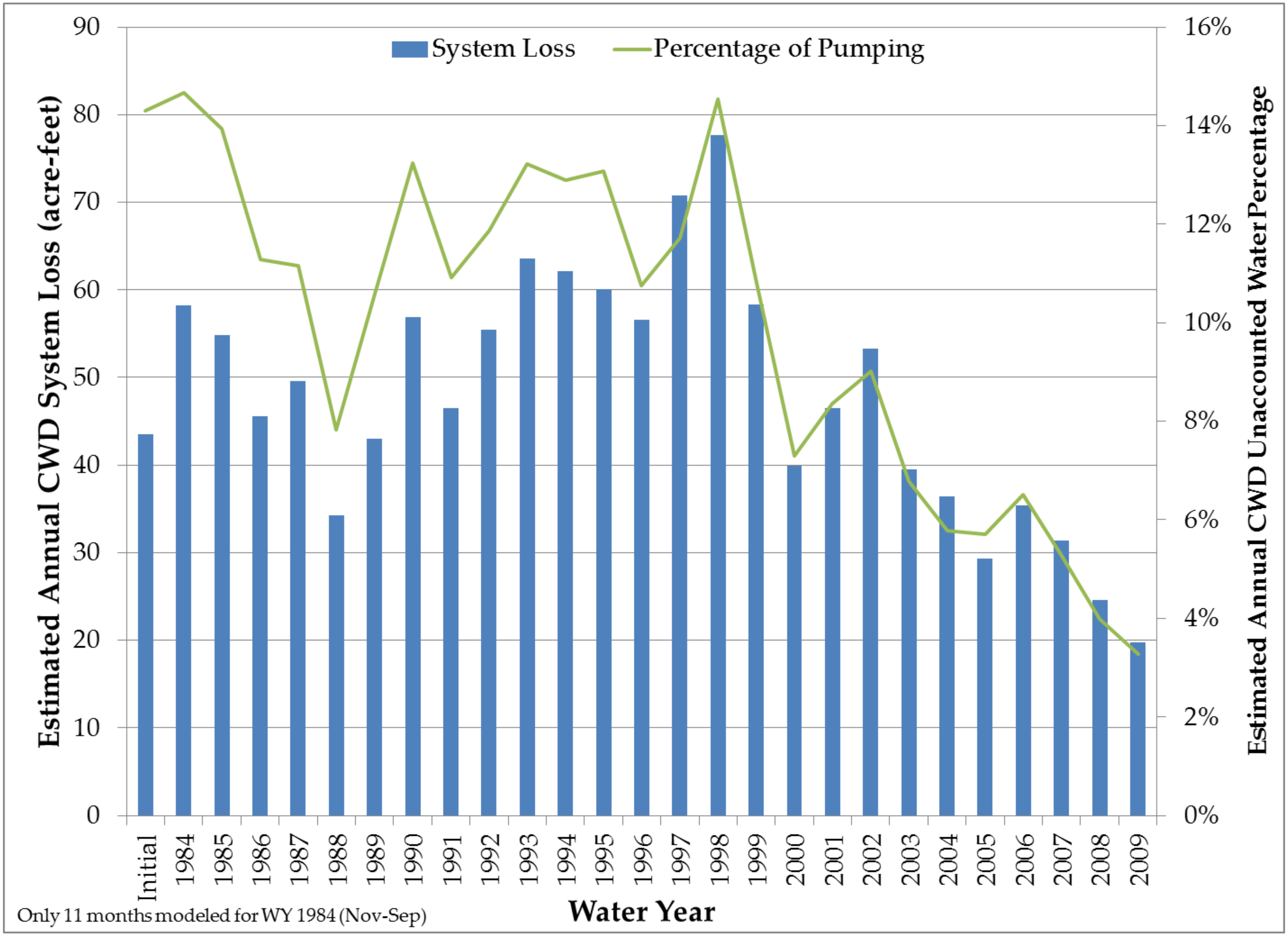

Figure 5-14. CWD Estimated Annual System Loss Based on Unaccounted Water Percentage

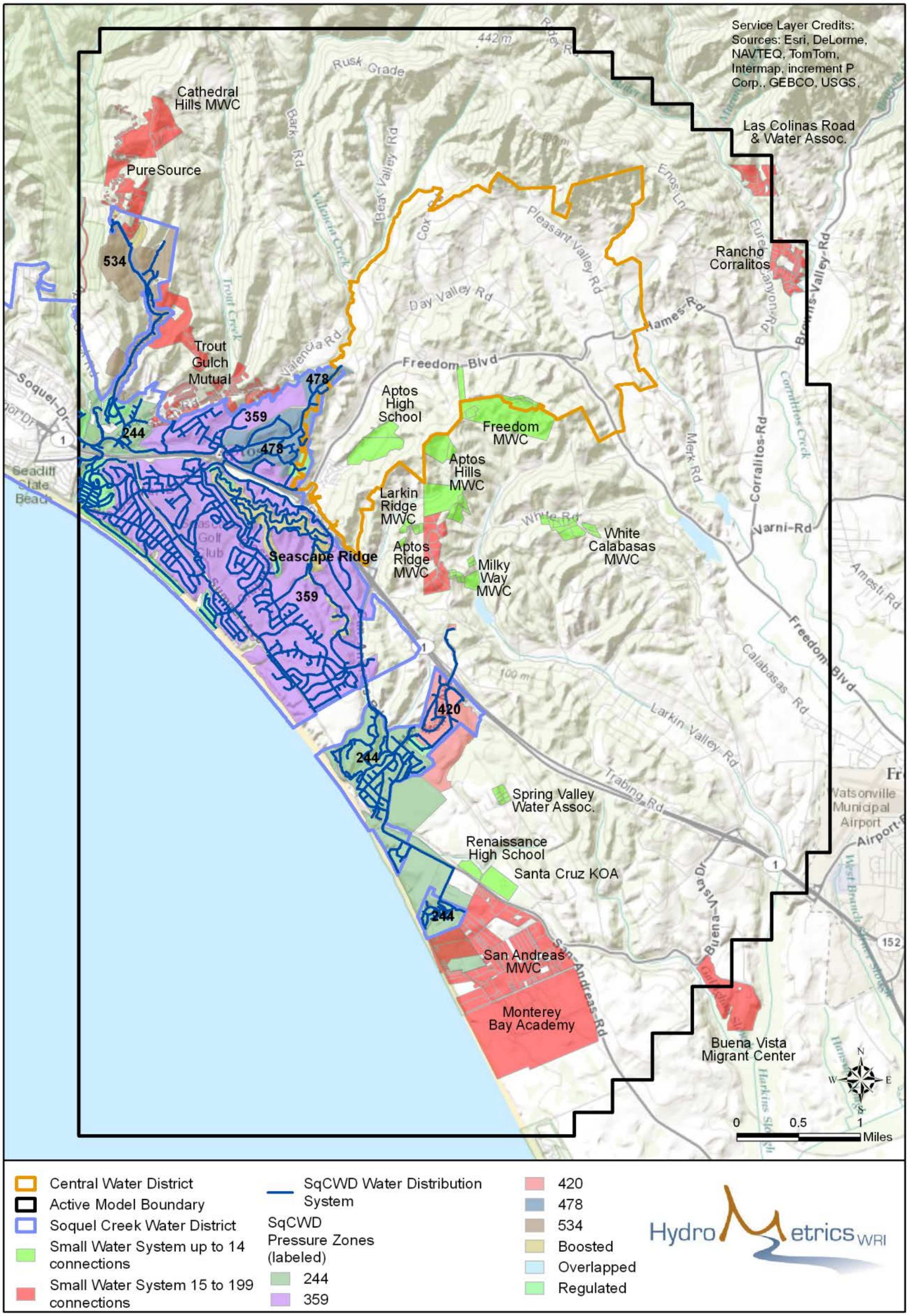

Figure 5-15. SqCWD Pressure Zones and Distribution System

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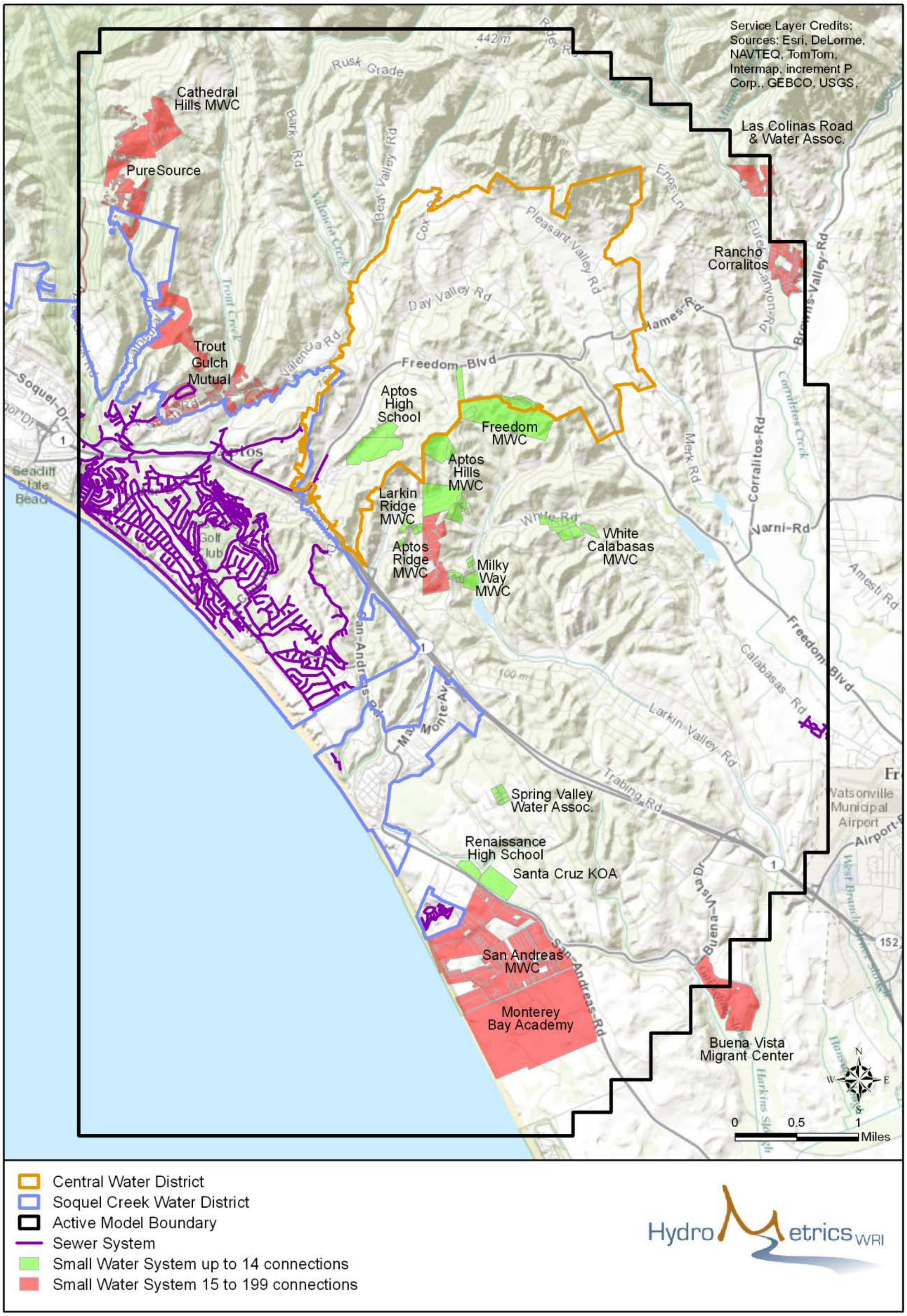

Figure 5-16. Sewer System in CWD Model Area

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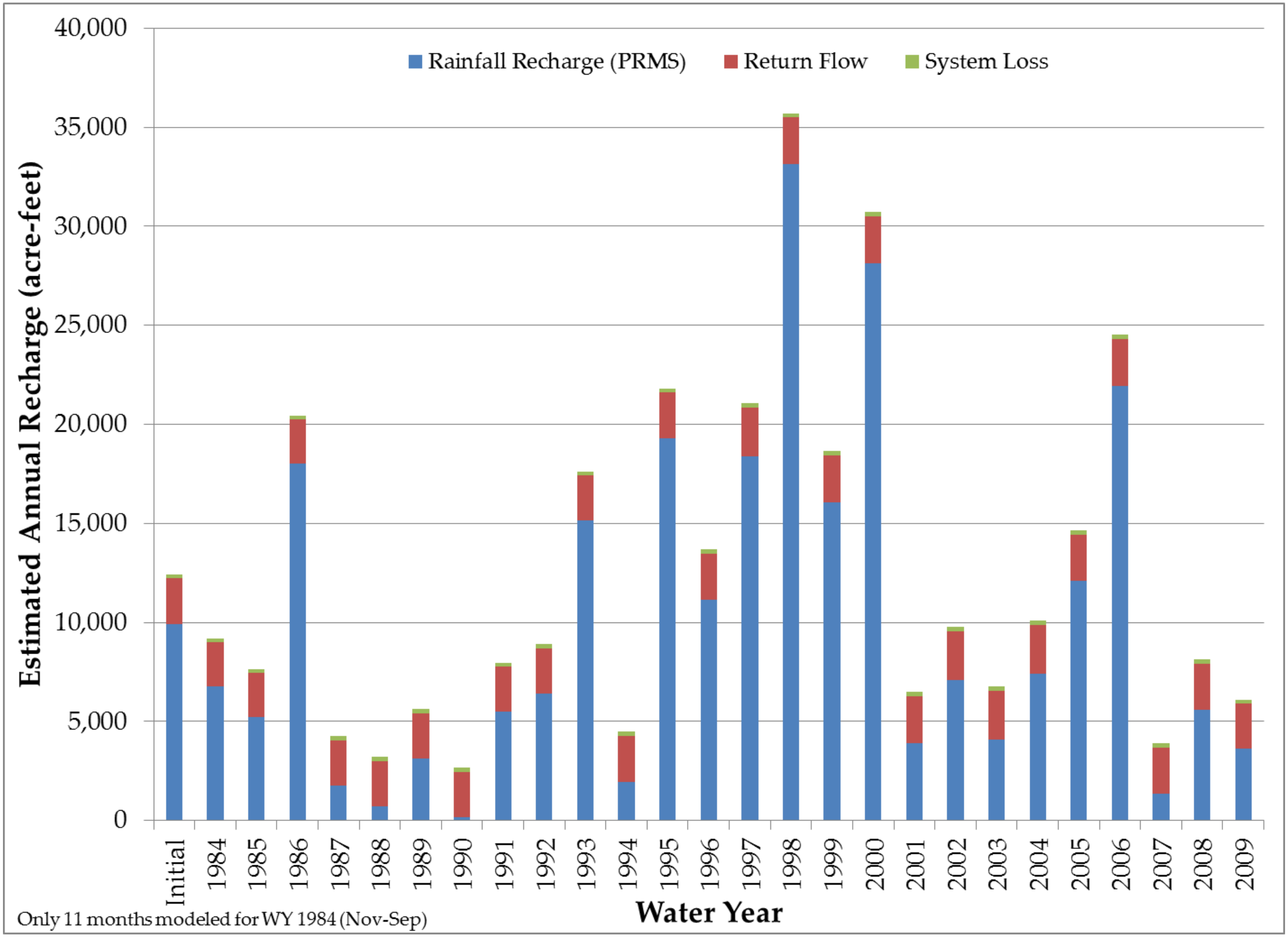

Figure 5-17. Modeled Initial Conditions and Annual Recharge

5.6 Boundary Conditions

The updated CWD model uses the following boundary conditions (Figure 5-18):

- Transient specified heads for the western boundary in the Purisima DEF unit
- General head boundary for the eastern boundary with Pajaro Valley
- General head boundary for sea level
- Specified flux for flux from upgradient watersheds west of the Zayante Fault
- Constant specified heads for upgradient boundaries east of the Zayante Fault
- Streams

5.6.1 Purisima DEF Unit West Boundary

The DWSAP model used a general head boundary on the western edge of the model domain in layers 9 and 10 for the deep Purisima F unit and the Purisima DEF unit. In the updated CWD model, transient specified heads are used to define conditions along the west boundary (Figure 5-19) from the boundary condition for inflow from the upgradient Aptos Creek watershed and the boundary condition for Monterey Bay. Groundwater level data from the T. Hopkins, Aptos Creek, SC-8D, and Seacliff wells are used to define heads along the boundary over the calibration period of Water Year 1984-2009. Since the groundwater levels used to define the boundary reflect pumping conditions at the T. Hopkins, Aptos Creek, and Seacliff wells, pumping from those wells is not included in the model.

The specified head boundary must have heads defined for every monthly stress period of the simulation. The heads are based on monthly average groundwater levels calculated for the four wells. However, none of the four wells had groundwater level measurements every month of the calibration period (Table 5-11, Figure 5-20). The Aptos Creek well was monitored throughout the calibration period, but the other three wells had major data gaps. These major data gaps are addressed differently for each well. For minor data gaps of up to three months, monthly heads are estimated by interpolating between groundwater levels on both side of the gap.

The T. Hopkins well was monitored regularly beginning early 1993. T. Hopkins groundwater levels have been consistent with nearby Aptos Creek groundwater levels so Aptos Creek groundwater levels prior to Water Year 1993 are considered representative of the T. Hopkins and Aptos Creek area.

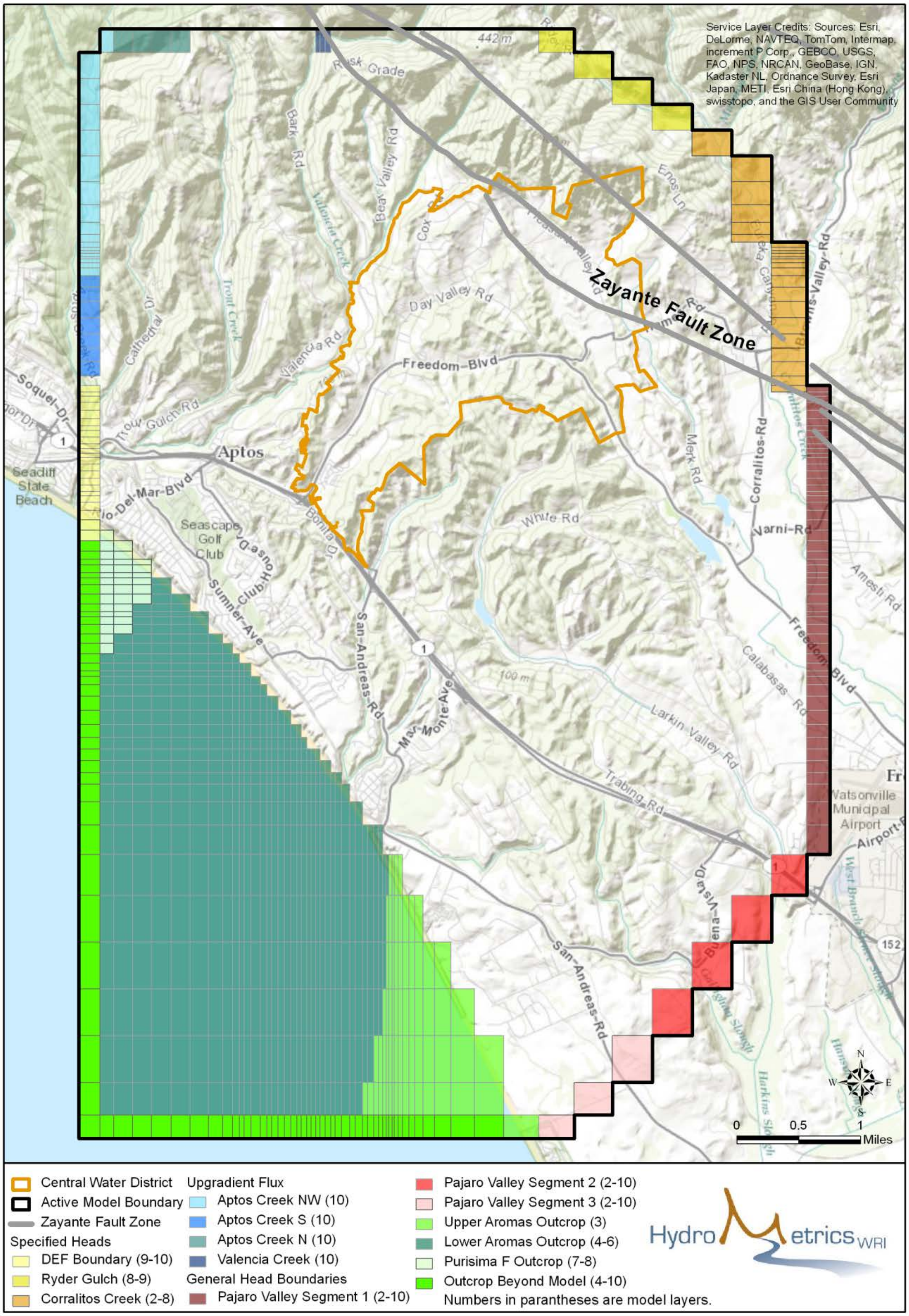

Figure 5-18. Model Boundary Conditions

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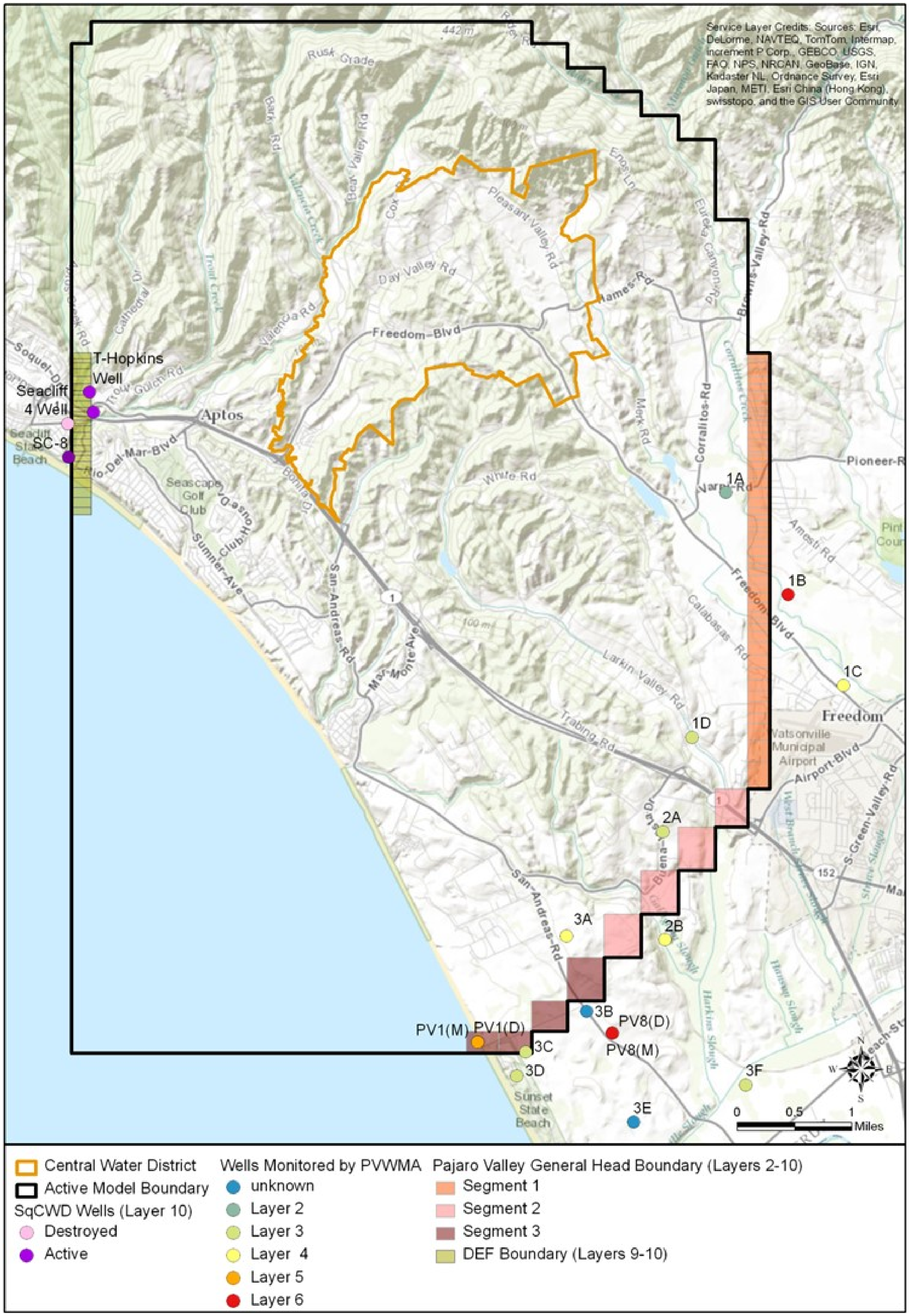

Figure 5-19. Boundary Conditions Based on Groundwater Level Data

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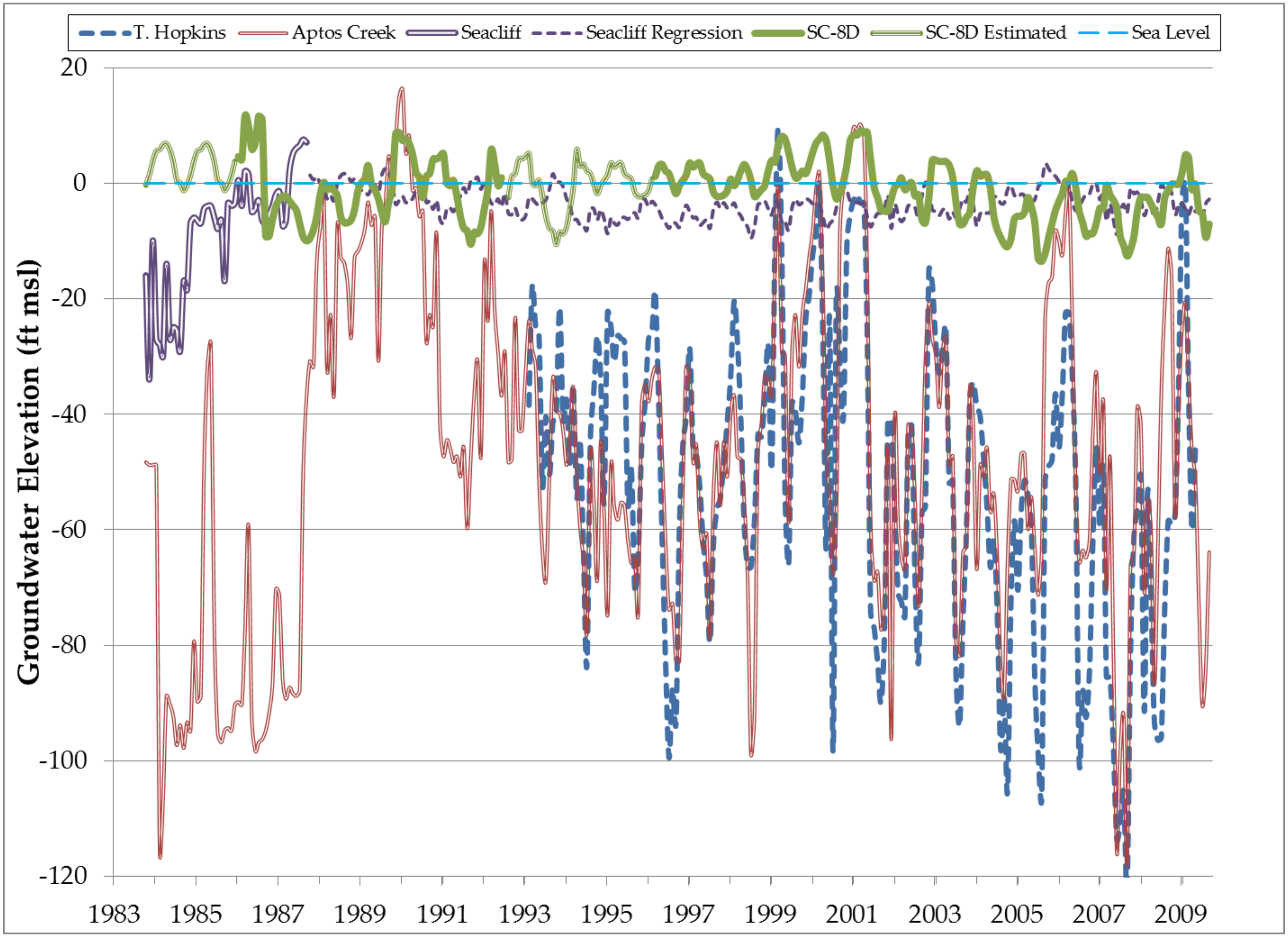

Figure 5-20. Measured and Estimated Groundwater Levels Used for DEF Unit West Boundary

Measurements did not begin at SC-8D until 1986 and were not taken from 1992-1996 when the well needed replacement. In order to have a control point closer to the coast that has seasonal variation, the pre-1986 and 1992-1996 data gaps at SC-8D are filled in. The pre-1986 data gap is filled in with monthly averages of measured groundwater levels. The June 1992-March 1994 data gap is filled in by repeating June 1990-March 1992 groundwater levels and the April 1994-February 1996 is filled in by duplicating April 1996-February 1998 groundwater levels.

Measurements at the Seacliff well stopped in January 1988. Although the Seacliff well was closer to the Aptos Creek well than the SC-8D well, groundwater levels were more similar to those of the SC-8D well so groundwater levels at the Seacliff well after Water Year 1987 are estimated based on a linear regression applied to Seacliff data against Aptos Creek and SC-8D data for Water Years October 1983-January 1988. Groundwater levels at the Seacliff wells for Water February 1988-September 2009 are based on the regression with measured and estimated Aptos Creek and SC-8D well groundwater levels using the following formula where h is estimated groundwater level in the feet. The mean squared error for the regression is 101 square feet.

$$h_{Seacliff} = 0.09h_{Aptos\ Creek} - 0.47h_{8D} - 0.25$$

Table 5-11. West Boundary Groundwater Level Data Availability during Calibration Period

Well	Period of Regular Monitoring	Maximum Data Gap During Period of Regular Monitoring
T. Hopkins	2/1993 - 9/2009	93 days
Aptos Creek	10/1983 - 9/2009	91 days
Seacliff	10/1983 – 1/1988	21 days
SC-8D	1/1986 - 5/1992 3/1996 – 9/2009	92 days

Groundwater levels were interpolated and extrapolated for each cell in the boundary condition based on available and estimated data at the four wells and sea level at the southern end using Shepard's inverse-distance weighting method. Initial conditions were based on October 1983 measurements. Figure 5-21 shows an example cross-sections of specified heads along the boundary at several times.

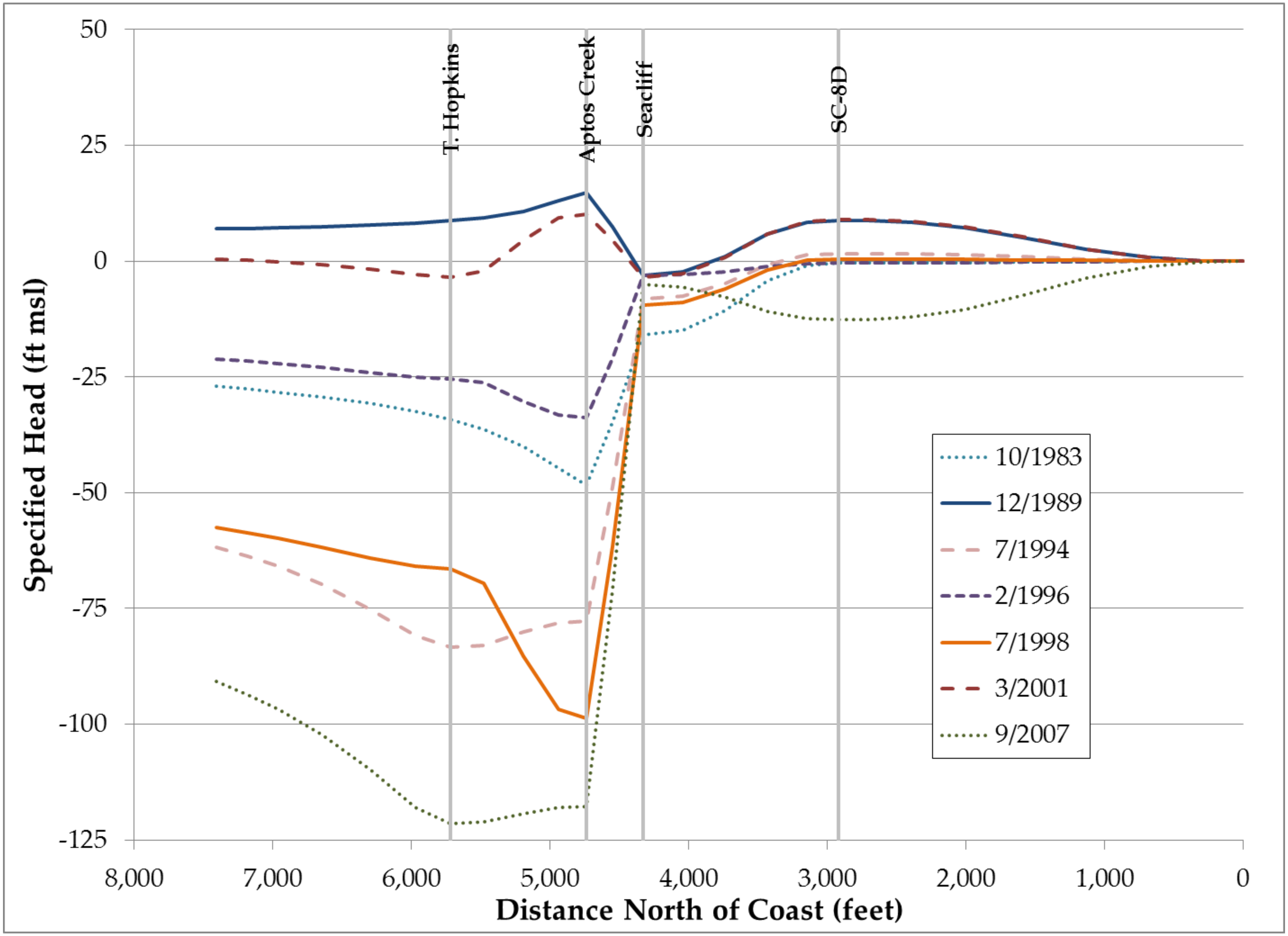

Figure 5-21. Example Cross-Sections of Specified Heads along Western DEF Unit Boundary Condition

5.6.2 SOUTHEAST BOUNDARY CONDITION

A general head boundary is used for the southeast model boundary, with transient heads estimated for the entire length of the boundary based on groundwater level data provided by PVWMA. However, groundwater level data in the proximity of the boundary are fairly limited, with only a few of the wells having sufficient measurements to characterize groundwater levels over the full calibration period of Water Years 1984-2009. Therefore, the general head boundary is used to reflect the greater uncertainty of groundwater levels at the boundary than at the west DEF boundary where specified heads are used.

The steps were used to estimate the transient head along the GHB with the same seasonal variation repeated each year were:

1) Evaluating the general pattern of groundwater levels;
2) Evaluate trends in groundwater levels over the simulation period
3) Quantifying how average annual groundwater levels vary spatially along the boundary;
4) Quantifying how the seasonal pattern of groundwater levels varies spatially; and
5) Calculate boundary heads for each monthly stress period that incorporates the appropriate spatial and temporal variability.

1) GENERAL GROUNDWATER LEVEL PATTERN

The groundwater level pattern in the area of the southeastern model boundary was evaluated by consulting fall groundwater contour maps from PVWMA's annual reports for 1992, 2006, and 2008 to 2010 in the fall (PVWMA, 1993, 2007, 2009, 2010, 2011). The pattern shown on these maps is not consistent from year to year, presumably because a different set of wells was measured for each map. The 2006, 2009, and 2010 maps (Figure 5-22) showed head patterns along the boundary that were reasonably consistent, so these maps were used to develop the head pattern along the boundary. The 1992 map was not used for the analysis.

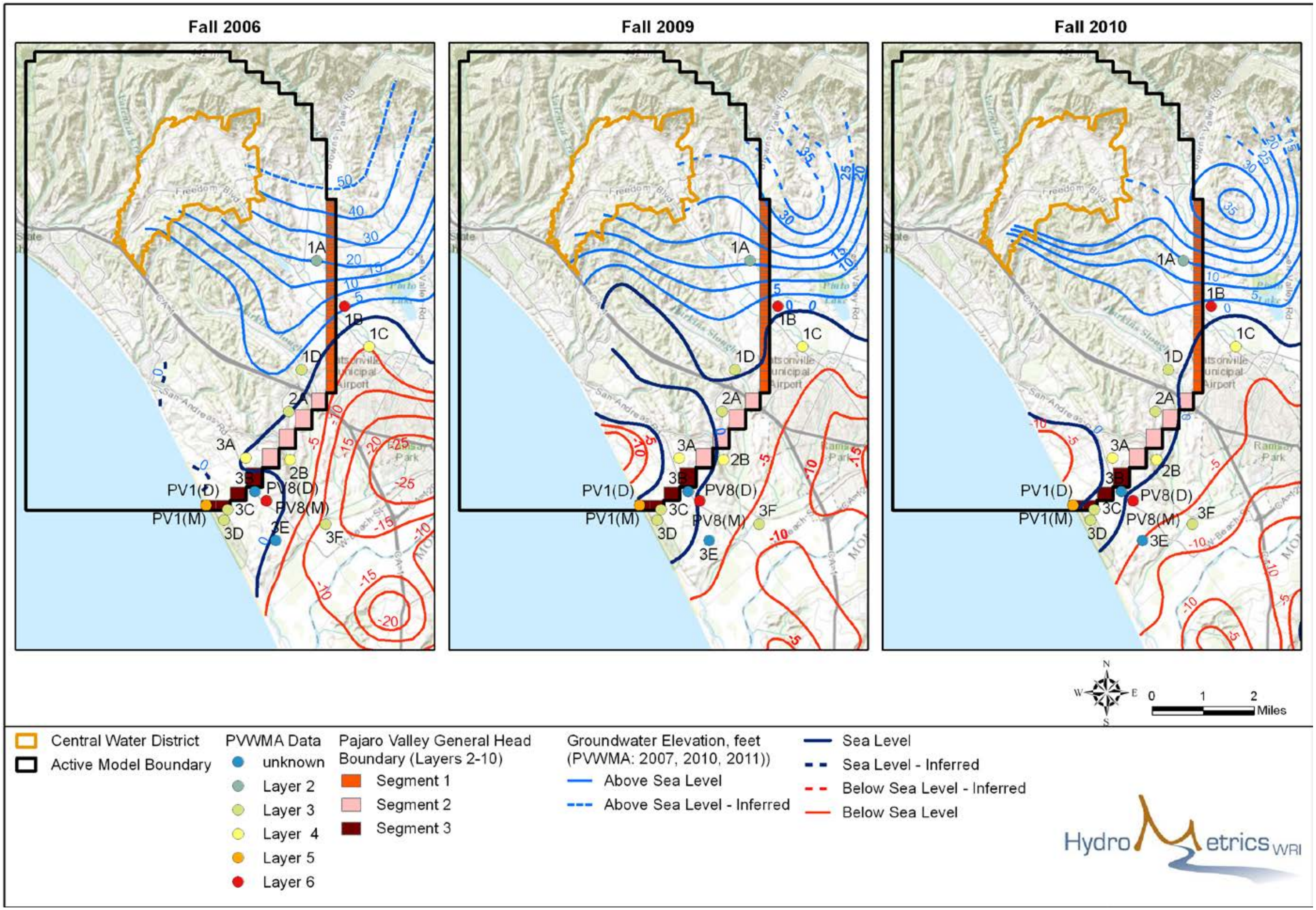

Figure 5-22. PVWMA Annual Report Fall Contour Maps for 2006, 2009, and 2010

Based on the groundwater level pattern, the boundary is divided into three segments (Figure 5-19) with each segment showing a different groundwater level pattern. Groundwater levels decrease from north to south along Segment 1. Groundwater levels are approximately uniform along Segment 2, with contours approximately parallel to the boundary. Groundwater levels decrease from northeast to the coast along Segment 3.

2) Examining Groundwater Level Trends

The data do indicate that groundwater levels did not seem to change much over the calibration period except for seasonal variation observed each year. Therefore, each cell in the boundary has a constant average annual head with a seasonal variation.

3) Quantifying Spatial Variation of Annual Average Groundwater Levels

Groundwater levels at wells near the boundary (Figure 5-19) were not taken at regular time intervals. In order to be representative of all seasons, averages for each of the twelve months were first calculated and the average of these monthly averages was used as the annual average. The spatial variation of annual average groundwater levels along the boundary was based on data from five wells (Table 5-12).

Table 5-12. Wells Used for Spatial Variation of Groundwater Levels at Southeast Boundary

Wells	Segment	Model Layers	Annual Average (ft msl)	Seasonal Fluctuation (ft)	Measurements	Years
1A	1	1-10	12.0	7.3	128	1994-2012
2A	2	1-10	2.7	5.0	193	1970-2012
2B	2	1-10			106	1970-2012
PV1(M)	3	1-4	2.8	1.8	197	1989-2012
PV1(D)	3	5-10	1.5	2.3	195	1989-2012

Groundwater levels at wells 2A and 2B were averaged to set the annual average for head all cells in Segment 2 as the groundwater level pattern for Segment 2 is approximately uniform.

The north-south spatial gradient for the annual average for head in Segment 1 is 0.00072 feet/feet based on the linear interpolation between the average annual groundwater level at Well 1A and the average annual groundwater level at Segment 2 applied to the southern end of Segment 1. The gradient is extrapolated to the northern end of the boundary beyond Well 1A. Wells 1B-D were not used as each had only 16-33 measurements.

The northeast-coast spatial gradients for Segment 3 of -0.00017 feet/feet in layers 1-4 and 0.00014 feet/feet in layers 5-10 were based on the linear interpolation between the average annual groundwater level at Segment 2 applied to the northeast end of Segment 3 and wells PV1M (layers 1-4) and PV1D (layers 5-10) near the coast. Data from wells 3A-C, are not used as each had only have 5-24 total measurements. Data from PV-8M and PV-8D and wells 3D-E are not used as they are offset from the boundary (Figure 5-19).

4) Quantifying Spatial Variation of Groundwater Level Seasonal Fluctuation

While the average annual groundwater level has been relatively constant over time at most of the wells in the area, there has been a consistent seasonal pattern to the groundwater level measurements. Groundwater levels are higher in the winter to early spring and lower in the late summer to autumn.

The seasonal fluctuation was quantified by determining the approximate change from high to low season in each year for the five wells used to quantify the spatial variation of annual average groundwater levels (Table 5-12). The spatial variation of the seasonal fluctuation is calculated in the same manner as was used for average annual groundwater levels, the seasonal fluctuation assumed to be uniform along Segment 2 and to change linearly along Segment 1 and Segment 3. Gradients in seasonal fluctuation were calculated along Segments 1 and 3 using the ends of Segment 2 as end points and well 19 for Segment 1, and wells PV1M and PV1D for Segment 3 as the other points in the gradient calculation. Calculated gradients in seasonal fluctuation are 0.00018 feet/feet decreasing to the south along Segment 1, 0.00040 feet/feet along Segment 3 in layers 1 to 4 increasing to the southwest, and 0.00033 ft/ft along Segment 3 in layers 5 to 10 decreasing to the southwest.

5) CALCULATING BOUNDARY CELL HEADS FOR EACH MONTH

Based on the spatially interpolated or extrapolated annual average groundwater level and seasonal fluctuation at each general head boundary cell, boundary cell heads were calculated for each month based on the following equation:

$$h_{i,t} = \frac{A_{ann,i}}{2}\sin\left(\frac{2\pi}{12}m_t\right) + \left(h_{avg,i} - \frac{A_{ann,i}}{2}\right)$$

where $h_{i,t}$ is the calculated head at grid cell i for time t, $A_{ann,i}$ is the amplitude of the seasonal groundwater level fluctuation at grid cell i (ft), m_t is the month of the year (January = 1, February = 2, etc.) of time t, and $h_{avg,i}$ is the annual average groundwater level at grid cell i (ft).

Figure 5-23 shows the head profiles for March and September. Figure 5-24 Figure 5-25 show comparisons of heads used in the model with measured groundwater levels.

Heads for the initial conditions are set equal to the average annual groundwater level calculated along the boundary.

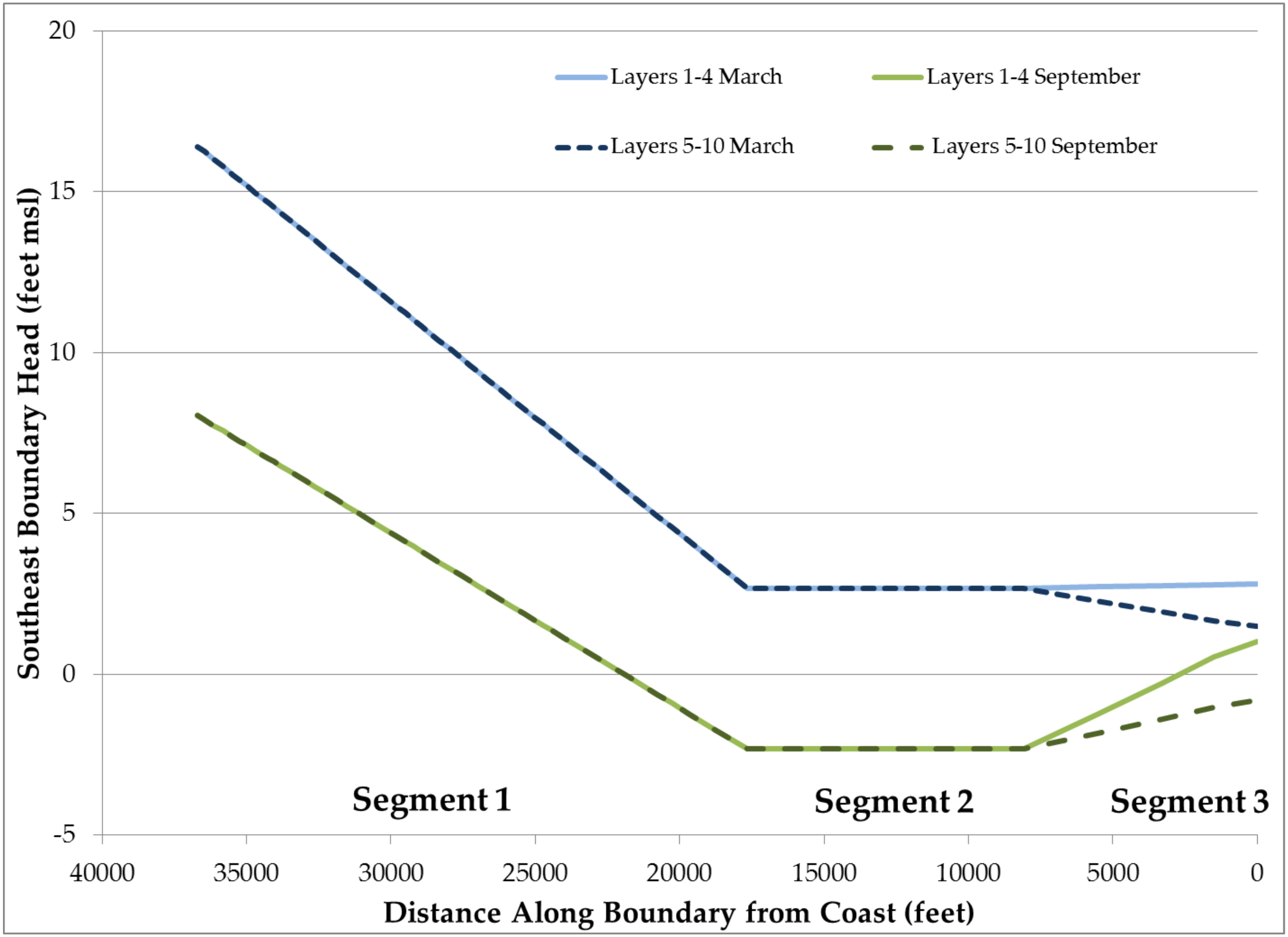

Figure 5-23. Seasonal Variation of Southeast Boundary Head

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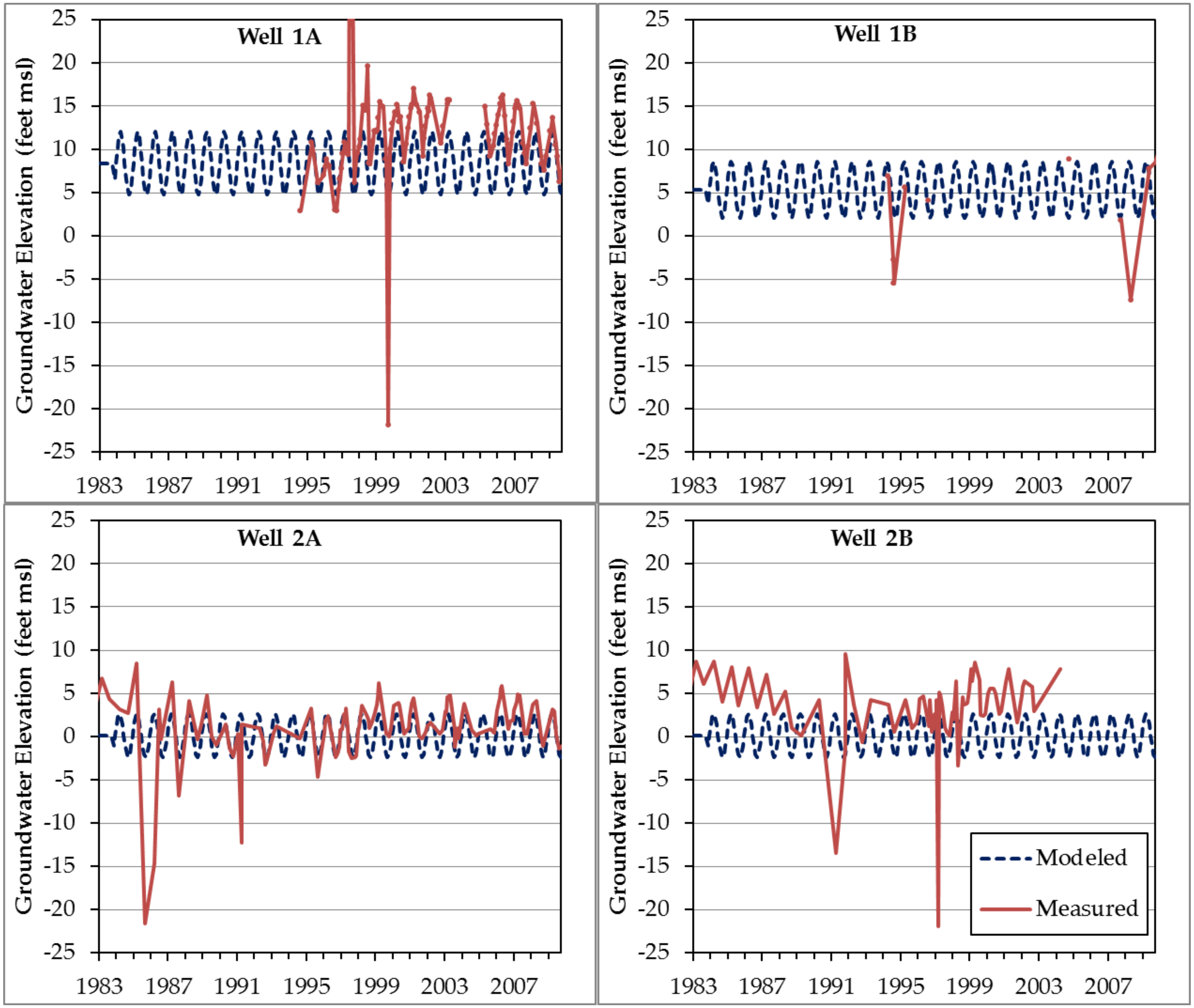

Figure 5-24. Comparison of Southeast Boundary Heads (Segments 1 and 2) with Nearby Measured Groundwater Levels

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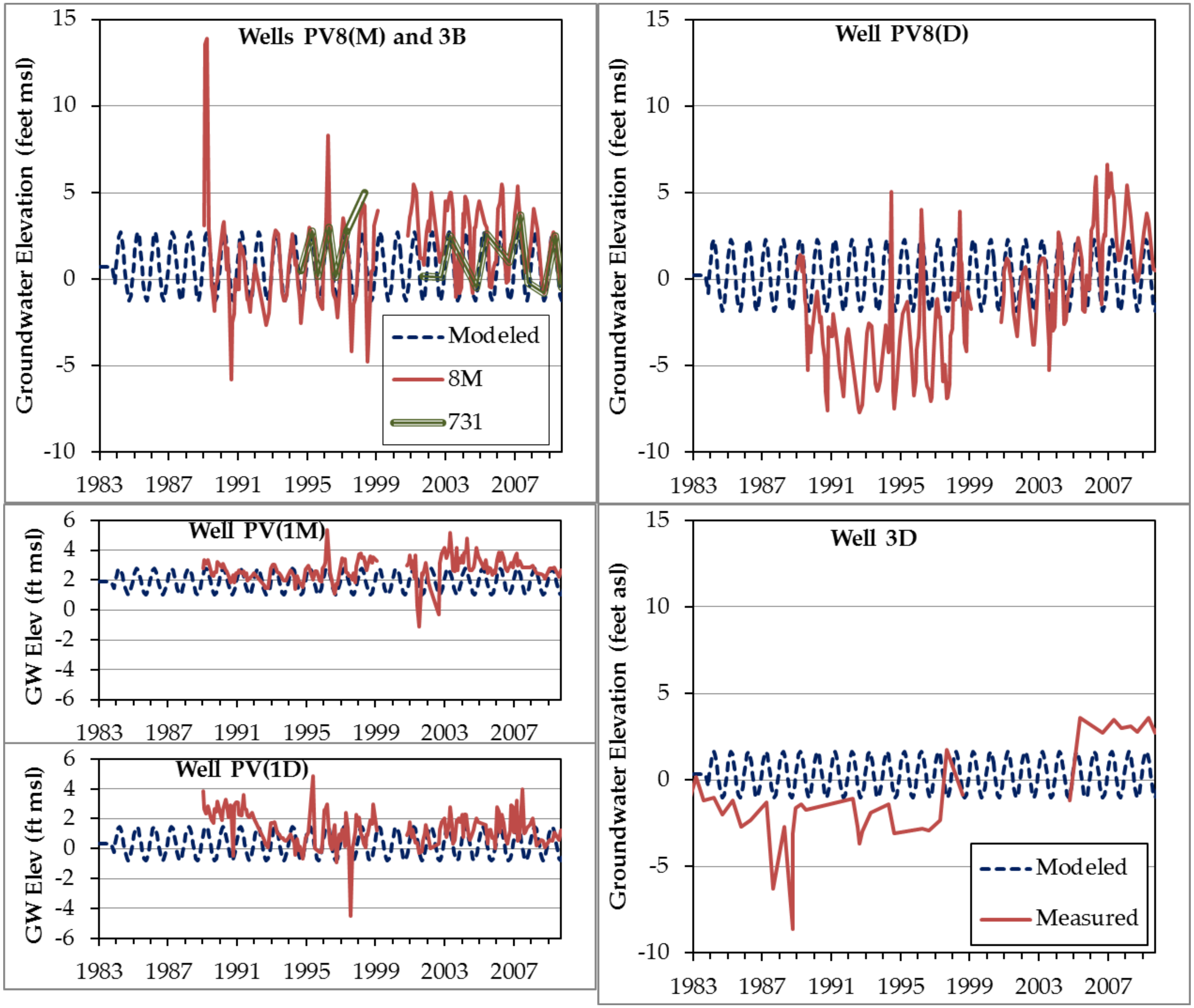

Figure 5-25 Comparison of Southeast Boundary Heads (Segment 3) with Nearby Measured Groundwater Levels

5.6.3 Monterey Bay Sea Level

The updated CWD model simulates the Monterey Bay sea level boundary condition as a general head boundary with an elevation of 0 feet (Figure 5-18). There are two types of sea level boundary condition cells: 1) cells along the edge of the model grid boundary where the layer has not outcropped to the seabed within the model domain, and 2) model cells that outcrop to the seabed.

Using a general head boundary for cells along the edge of the model grid where the layer has not outcropped to the seabed is consistent with the CWD DWSAP model. This general head boundary occurs in layers 4-10 (Figure 5-18). The general head boundary has a conductance representing distance from the grid to the outcrop and hydraulic conductivity for that distance. The conductance of this general head boundary in each layer is estimated during the calibration process.

Model layers 3-8 outcrop to the seabed in some areas of the model grid domain (Figure 5-18). The CWD DWSAP model used constant heads for outcropping cells, which applies 0 feet as the head for the model cells. The updated CWD model uses general head boundaries for the outcropping cells. The conductance of the general head boundaries used in the updated model represent seabed sediments which allow non-zero head to be simulated in the aquifer below the seabed. The inclusion of a conductance for seabed sediments is also consistent with the cross-sectional modeling that estimated protective water elevations at SqCWD's coastal monitoring wells (HydroMetrics LLC, 2009a). The conductances of the general head boundaries for each layer of outcropping cells were estimated during the calibration process.

5.6.4 Upgradient Flux West of Zayante Fault

CWD's DWSAP model (Johnson, 2009) used constant heads to simulate upgradient boundary conditions. These constant heads were located along Aptos Creek, Valencia Creek, and Ryder Gulch/Corralitos Creek, but the flow into the model at these boundaries represented more than just leakage from the creeks. The upgradient flow is actually predominantly groundwater flow from upgradient aquifers that flows in the saturated zone well below the creeks.

The upgradient inflow for the updated CWD model is based on recharge estimated by the PRMS model for subbasins with HRUs upgradient of the CWD model boundary west of the Zayante fault (Figure 5-26). As with areal recharge within the model domain, recharge upgradient of the model boundary is calculated as groundwater recharge percolating through the soil zone minus flow to streams (PRMS variables gw_in_soil + gw_in_ssr - gwres_flow). Calculated monthly recharge for full and partial HRUs upgradient of the model boundary are added up for the Aptos Creek West, Aptos Creek Northwest, Aptos Creek North, and Valencia Creek subbasins. Upslope and cascade flows are omitted as they will cancel each other out when adding up flows for HRUs that lead to the CWD model boundary.

There is a delay between upgradient recharge and its inflow into the CWD model gradient so the time series of groundwater flow into the model should be smoother (i.e less variable) than the time series calculated monthly recharge from the PRMS model. In addition, monthly recharge calculated from PRMS is sometimes negative when flow to streams exceeds percolation through the soil zone for the month. Upgradient inflow to the model should always be positive, as it is unlikely that regional gradients would ever be reversed at these boundaries. The updated CWD model approximates upgradient inflow based on a 7 year running average of upgradient recharge from PRMS to ensure there are no negative values for the boundary flow (Figure 5-27). The running average for a monthly stress period is the mean of the PRMS upgradient recharge for the current month and each of the preceding 83 months.

Initial conditions for the upgradient inflow is based on the annual average of upgradient recharge calculated from the PRMS model for its entire Water Year 1984-2009 calibration period. Upgradient inflow for the first seven years of the CWD model calibration period is based on a 7 year running average assuming that upgradient inflow prior to November 1983 was equivalent to the annual average applied for the initial conditions. Figure 5-28 shows the upgradient flux for intitial conditions and each water year of the calibration period.

The boundary condition is implemented as injection wells (positive inflow). The boundary conditions are placed in the lowest layer (layer 10) to ensure that all specified inflow even as shallower layers dry out.

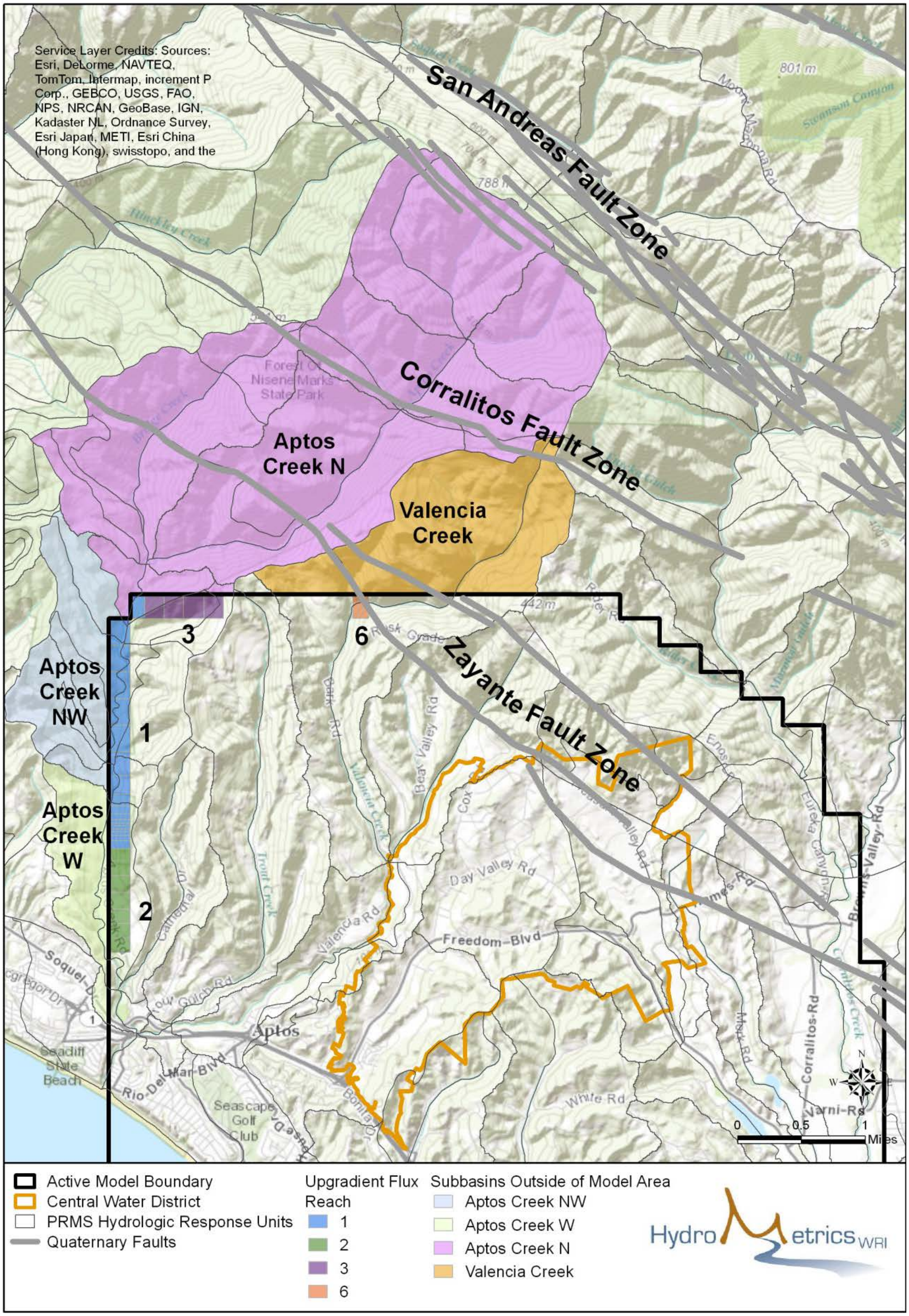

Figure 5-26. PRMS HRU Subbasin Areas Upgradient of Model West of Zayante Fault

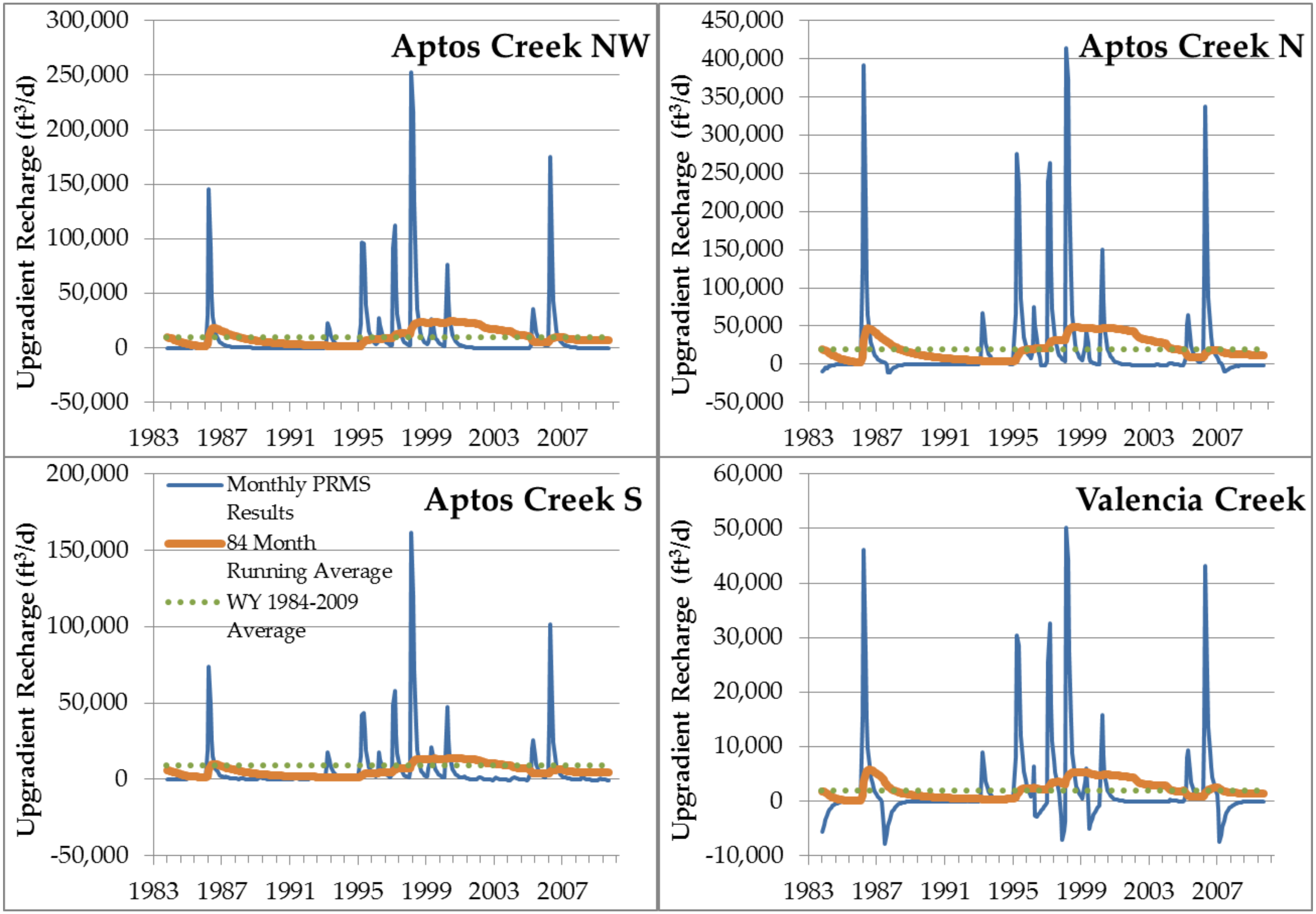

Figure 5-27. PRMS Monthly Upgradient Recharge and Upgradient Inflow

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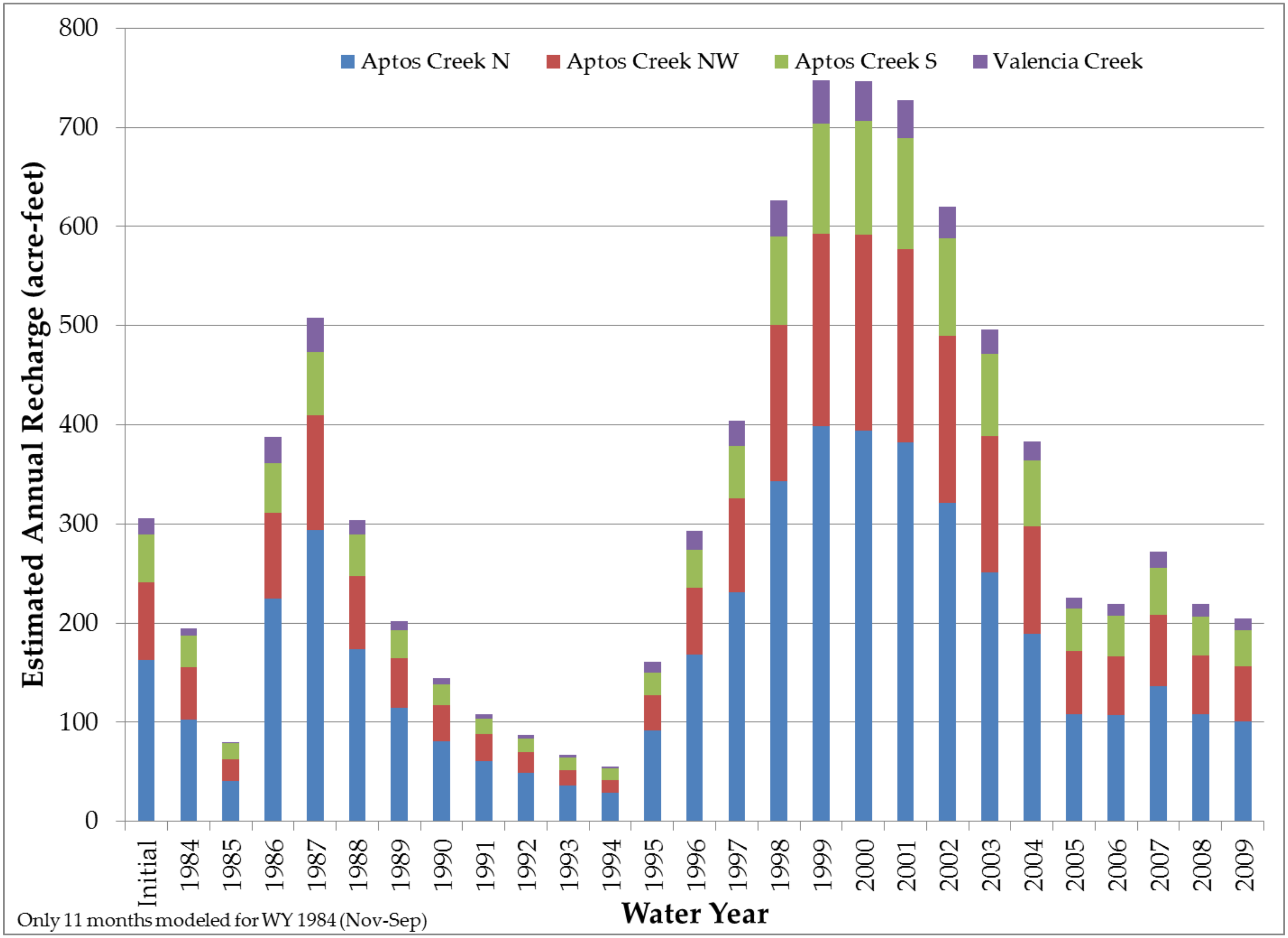

Figure 5-28. Initial Conditions and Annual Upgradient Inflow West of Zayante Fault by Subbasin

5.6.5 UPGRADIENT HEADS EAST OF ZAYANTE FAULT

The upgradient boundary east of the Zayante Fault at Ryder Gulch and Corralitos Creek was unchanged from the steady-state constant heads used in the CWD DWSAP model. Modeling transient conditions in the area east of the Zayante Fault is unimportant for evaluating the strategy of shifting pumping between the Rob Roy and Cox well fields west of the Zayante Fault (Figure 5-18). .

5.6.6 STREAMS

The SFR2 stream package is included in the updated CWD model. The SFR2 package calculates leakage between streams and groundwater, but also routes flow in a stream network (Niswonger and Prudic, 2005). However, the updated CWD model implements the SFR2 package such that there is no modeled stream flow or leakage between streams and groundwater for much of the model domain. The package is implemented to indicate where there may be stream-aquifer interaction and to facilitate future implementation of an integrated stream-aquifer model with GSFLOW (Markstrom et al., 2008)

Previous evaluation of stream-aquifer interaction showed that there is a vertical separation between Valencia Creek and regional groundwater levels near the Cox well field (HydroMetrics LLC, 2009b); this separation is known as a disconnected stream. Based on surface elevations and groundwater level data, this separation is expected to exist across much of the model domain. Leakage from the disconnected streams is independent of groundwater levels where this separation exists.

This leakage is calculated by PRMS, and incorporated in the areal recharge package RCH. The PRMS model was calibrated to streamflow and ignored leakage from streams (
Figure 5-29). Therefore, any leakage from the streams is included in the totals for areal recharge spatially distributed to the PRMS HRUs. To make MODFLOW consistent with the input from PRMS, leakage in the MODFLOW stream SFR2 package should be zero (Figure 5-30). As a result, the updated CWD model does not show the resolution of groundwater levels below streams that would result from a portion of recharge being concentrated under the stream network. For the regional assessment of the groundwater management strategy of shifting pumping from the Rob Roy well field to the Cox well field, this resolution is not necessary. However, this resolution may be necessary for evaluating effects at specific locations near streams.

To ensure zero leakage in the model where simulated groundwater levels are below the stream bed, streamflow modeled by the SFR2 package is set to zero wherever stream enters the model. As a result, where groundwater levels are below the stream bed from the upstream end, streamflow in the model is zero and leakage is zero. The SFR2 package in the updated CWD model does not represent actual streamflow. Instead, any streamflow that is simulated by the model represents baseflow contributed by groundwater within the model domain.

Locations where the model simulates streamflow would indicate a connection between groundwater and streamflow, where pumping could affect streamflow. However, these areas would not be consistent with the zero leakage assumption discussed above. If model results show that pumping effects on streamflow could be an issue in these locations, more detailed modeling of the stream-aquifer interaction that addresses this inconsistency may be necessary. With the implementation of the SFR2 package, the updated CWD model could be converted to a GSFLOW model (Markstrom et al., 2008) that fully integrates PRMS and MODFLOW to simulate stream-aquifer interaction in more detail.

Figure 5-31 shows the SFR2 stream segments implemented in the CWD model. Only streams interior to the model grid are included; parts of the streams above the model domain boundary are simulated using boundary conditions, as described in the previous two sections.

The creeks that are most likely to be affected by the groundwater management strategy being evaluated are Valencia Creek (stream segments 6-7) and its tributary (segments 4-5). Modeled results for these segments will be evaluated to see if and when the model shows stream-aquifer interaction for these creeks.

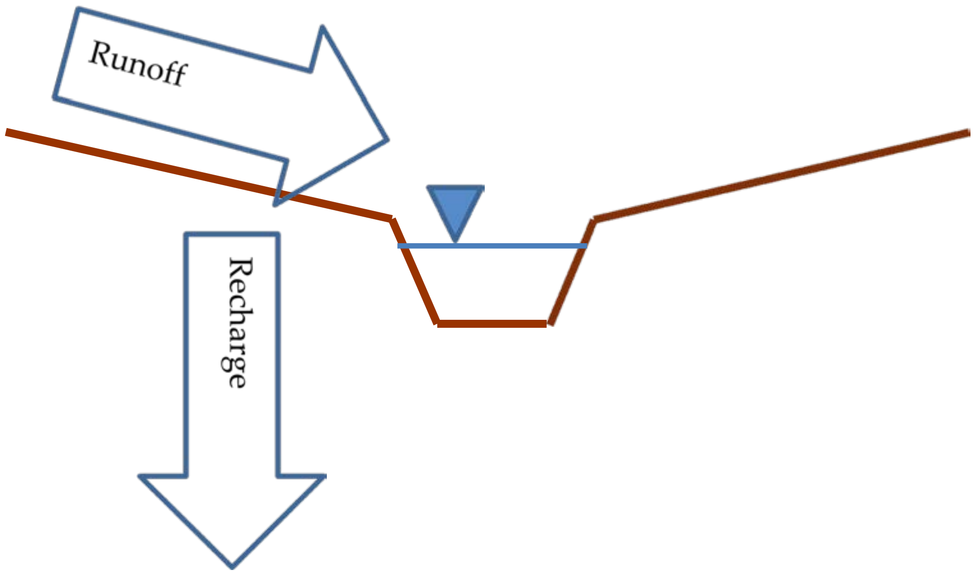

Figure 5-29. Schematic of PRMS Recharge Calculation Ignoring Stream Leakage

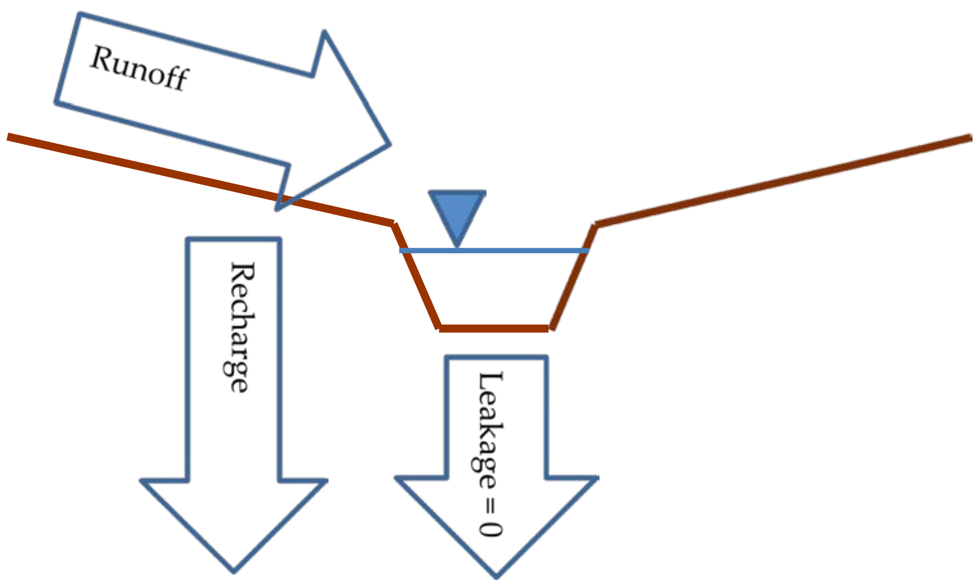

Figure 5-30. Schematic of Stream Leakage in MODFLOW Stream (SFR2) Package Consistent with PRMS Recharge Calculation

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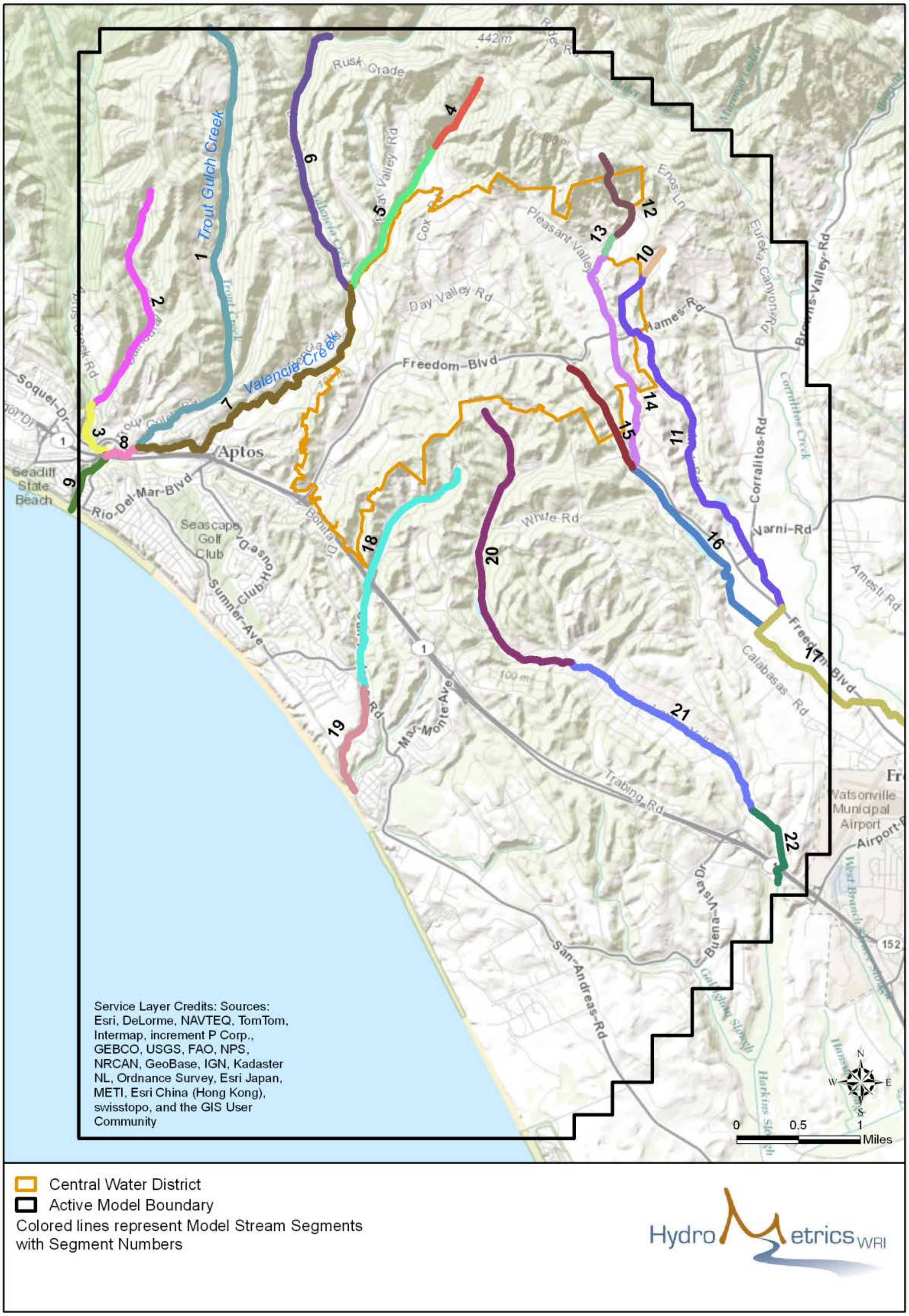

Figure 5-31. PRMS Stream Segments Implemented in Model SFR2 Package

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SECTION 6
EVALUATE LOCAL PURISIMA FORMATION SUSTAINABLE YIELD: GROUNDWATER MODEL CALIBRATION (TASK 4.1)

CWD's updated groundwater model (Section 5, Task 4.1) was calibrated so that the model can be defensibly used to evaluate the strategy of shifting pumping from the Rob Roy well field to the Cox well field. Therefore, calibration focused on accurately simulating groundwater level observations near the two well fields. This section documents model simulations used to evaluate whether the strategy can sustainably CWD's water supply goals and broadly assess the potential environmental impacts of the strategy (Section 7, Task 4.5).

The section was distributed to the Technical Advisory Committee (TAC) for review as a draft technical memorandum on January 14, 2014.

6.1 Calibration Approach

Calibrating CWD's updated groundwater model involved successive attempts to match model output to measured data from the calibration period. Simulated hydraulic heads were compared against available observed groundwater elevations. The model was considered calibrated when simulated results matched the measured data within an acceptable measure of accuracy, 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.

6.1.1 Pilot Point Method for Calibration of Aquifer Properties

A pilot point approach, rather than a zoned aquifer property approach, was used to distribute aquifer properties during calibration. The pilot point approach results in smoothly varying aquifer property fields. Doherty (2003) describes the methodology for the use of pilot points in groundwater model calibration. Using this method, the values of aquifer properties are estimated at the locations of a number of points, called pilot points, spread throughout the model domain. Aquifer properties are then assigned to the model grid by kriging data from the pilot points to the finite difference grid (Doherty, 2007).

The pilot point methodology avoids undesirable estimation of extreme or unrealistic aquifer properties at pilot points by using regularization. Regularization imposes a preference for homogeneity on the parameter estimation process. Heterogeneity is only included when necessary to match calibration data (Doherty, 2003 and Doherty and Hunt, 2010). As a result, we did not need to guess where unmapped heterogeneity might exist within a model domain ahead of the calibration process. Instead, the calibration process imposes heterogeneity only where necessary.

Prior to estimating any aquifer properties, the pilot points were selected manually based on following criteria (Doherty, 2002):

1) More pilot points were placed where there are more data;
2) Pilot points were placed between well locations in order to calibrate to head difference between wells;
3) Pilot points were placed in between wells and outflow boundaries.
4) Pilot points were placed to eliminate big gaps between adjacent pilot points;

For CWD's updated model, 20 to 50 pilot points were assigned to layers 1 through 6 which represent the Aromas Red Sands (Figure 6-1**Error! Reference source not found.**); and 45 to 70 pilot points were assigned to layers 7 through 10 which represent the Purisima Formation (Figure 6-2). Layers 1 through 3 use a consistent set of pilot points; and the aquifer property fields are the same for Layers 1 through 3. The pilot points are assigned to the three original hydraulic conductivity zones used in the DWSAP model (Johnson, 2009). Aquifer properties within each zone and within each layer are based on spatial interpolation of pilot point values from that zone only. The preferred condition of homogeneity is applied to each of these zones by layer. Only three to five pilot points per layer are used for the Zayante Fault zone, and one pilot point per layer is used for the zone north of the Zayante Fault as there is no calibration information available in these zones. Three to five pilot points are used for the Zayante Fault zone to allow for heterogeneity of fault properties that could affect downgradient groundwater levels. Using one pilot point north of the Zayante Fault results in the zone having one value for each property in each layer.

The hydrogeologic properties estimated at each of the pilot points included horizontal hydraulic conductivity, vertical anisotropy (the ratio of horizontal conductivity to vertical conductivity), specific storage, and specific yield. The initial values for the pilot points were based on the zone values in the DWSAP model.

The use of pilot points methodology results in over 1,300 property values that can be varied in the calibration. PEST software and its Singular Value Decomposition (SVD)-assist functionality (Watermark Numerical Computing, 2004) was used to help update the full set of parameter values and improve the calibration.

Property fields resulting from the use of the pilot points methodology were visually reviewed to assess whether the fields were geologically realistic. Several iterations of the methodology were conducted to improve geologic realism while maintaining adequate calibration to the observation data.

6.1.2 Calibrated Boundary Conditions

Conductances of the general head boundaries representing the model's seabed outcrop were calibrated for each layer (Figure 6-3 and Figure 6-4). Where the layer outcrops within the model domain, the conductance of the general head boundaries used in the updated model represent seabed sediments, which provide additional resistance to flow between the ocean and the shallow aquifer. The inclusion of a conductance for seabed sediments is also consistent with the cross-sectional modeling that estimated protective water elevations at SqCWD's coastal monitoring wells (HydroMetrics LLC, 2009a). For the general head boundaries at the edge of the model representing outcrop beyond the model domain, the conductance represents the distance to the outcrop and the hydraulic conductivity of the layer between the edge of the model and the outcrop.

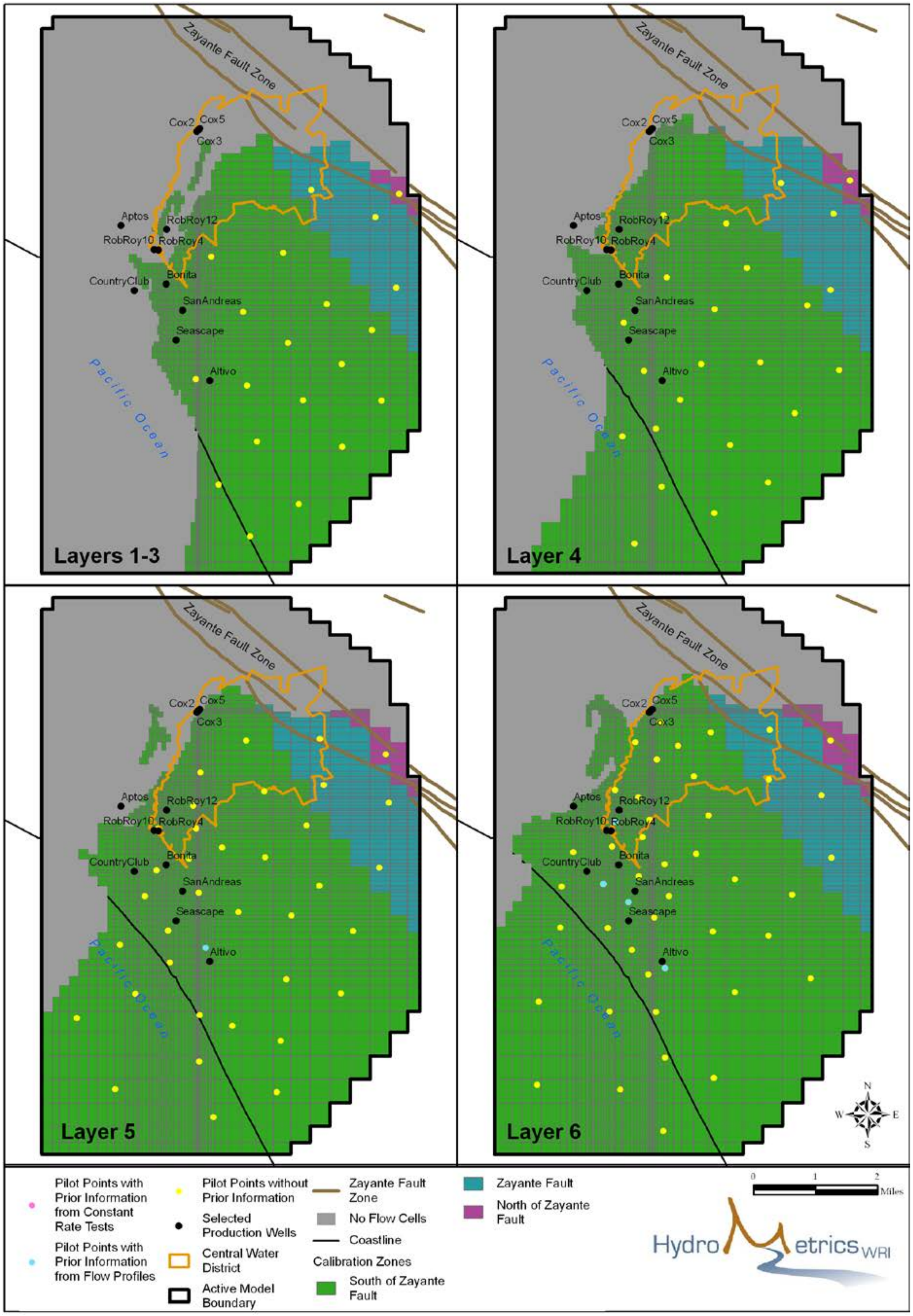

Figure 6-1. Pilot Points for Aromas Red Sands Layers 1-6

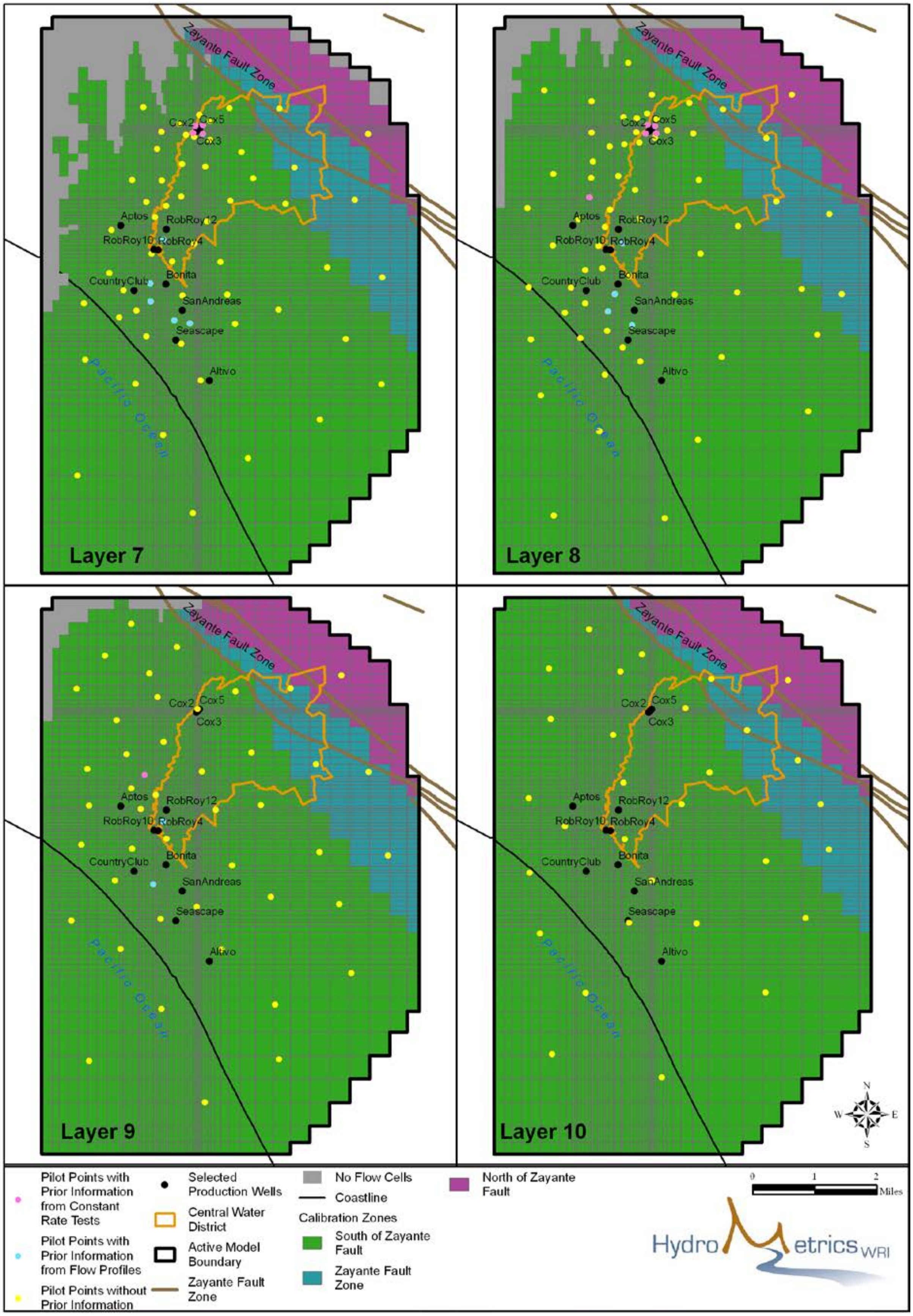

Figure 6-2. Pilot Points for Purisima Formation Layers 7-10

6.2 CALIBRATION DATA

6.2.1 GROUNDWATER ELEVATION MEASUREMENTS

Groundwater elevations have been measured at a number of production and monitoring wells in the model domain. CWD, SqCWD, PVWMA, and Santa Cruz County (County) record the data and provided it for this study (Table 6-1).

Figure 6-3 shows the wells used in calibration for wells completely screened in the Aromas Red Sands (layers 3-6). Figure 6-4 shows the wells used in calibration for wells at least partially screened in the Purisima Formation (layers 7-10).

Some of the recorded measurements appear to be anomalous and were excluded from model calibration. Anomalous measurements for which there is an obvious reason for the anomaly were excluded from all results. Otherwise, measurements are displayed on calibration hydrograph but excluded from other calibration results. Table 6-2 summarizes the excluded data.

Model results were recorded for at least five times. Model results from cell centers were interpolated from the five time steps to measurement dates and calibration well locations. Because many groundwater wells are screened across multiple model layers, composite groundwater levels were calculated from model results by layer. The composite groundwater levels were averages of screened layer results weighted by layer transmissivity, the percentage of screen in each layer multiplied by the horizontal hydraulic conductivity of each layer. Table 6-3 shows the percentage of screen in each layer for each calibration well.

Table 6-1. Calibration Data by Water Resource Agency Source

Water Resource Agency	Well Groups	Well Numbers/Names	Model Layers	Total Number of Measurements	Date Range
CWD	Cox Production Wells	2, 3, 5	7 - 8	571	10/1983 - 9/2009
	Rob Roy Production Wells	4, 10, 12	6 - 9	724	10/1983 - 9/2009
	Black Monitoring Well	Black	5 - 6	67	7/1984 - 9/2009
SqCWD	Service Area III Production Wells	Aptos, Bonita, Cliff, Country Club, San Andreas, Seascape	6 - 10	2,123	10/1983 - 9/2009
	Service Area IV Production Wells	Altivo, Sells	5 - 6	1,205	10/1983 - 9/2009
	Rob Roy 12 Monitoring Wells	CWD-A, B, C	5 - 9	522	7/1993 – 9/2009
	Coastal Monitoring Wells	SC-A1, SC-A2, SC-A3, SC-A4, SC-A8	3 - 10	2,818	2/1989 – 9/2009
	Monitoring Wells Adjacent to Production Wells	SC-20 (Polo), SC-A5 (Seascape), SC-A6 (Bonita), SC-A7 (Sells)	5 - 9	1,111	2/1989 – 9/2009
PVWMA	Private Wells Monitored by PVWMA	A, B (coded for privacy)	4 - 5	33	5/1994 – 9/2009
County	Private Wells Monitored by Environmental Health Services	Identified by Township-Range-Section	5 - 9	81	5/2008 – 9/2009

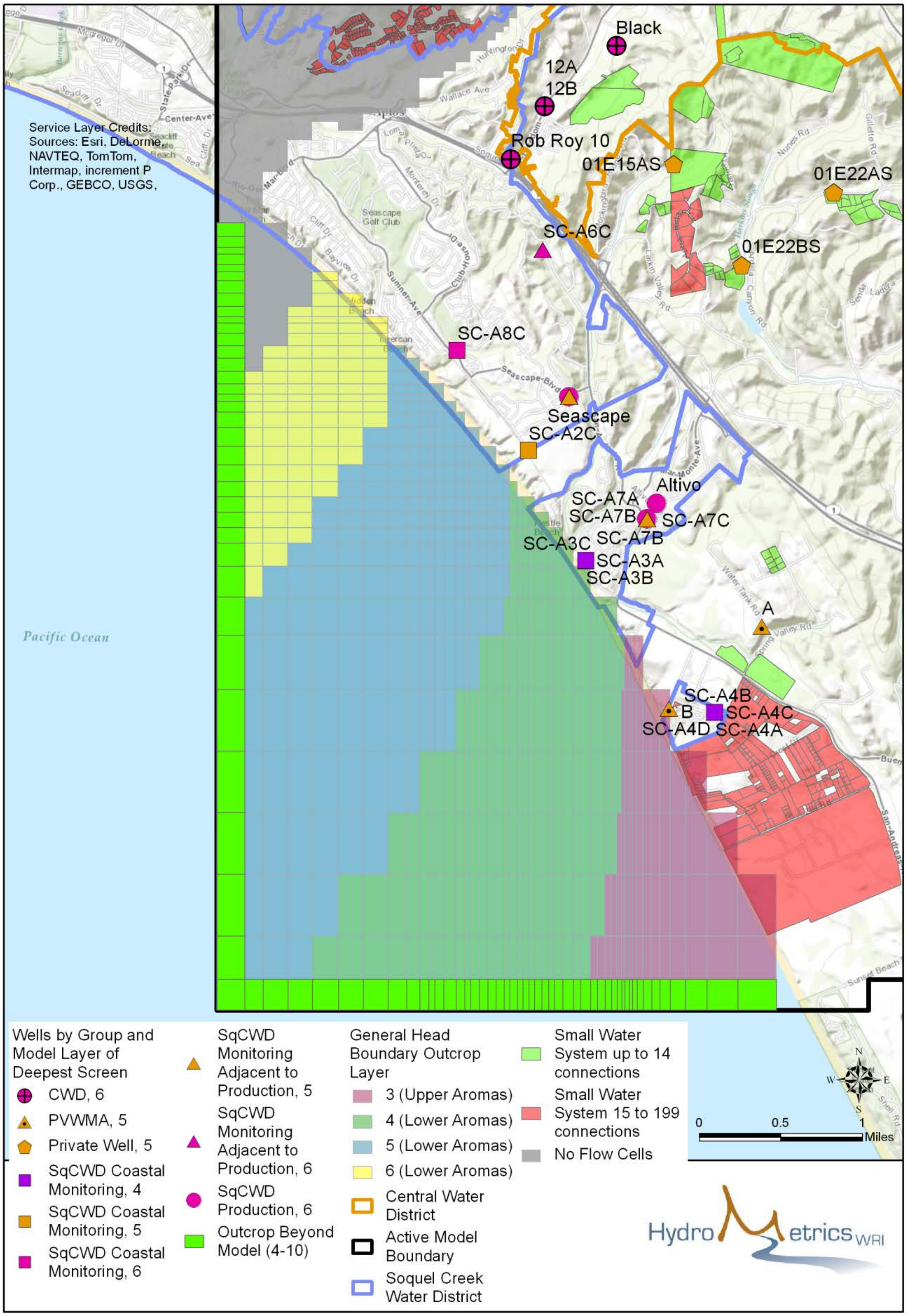

Figure 6-3. Aromas Calibration Wells and Seabed Outcrop Layers

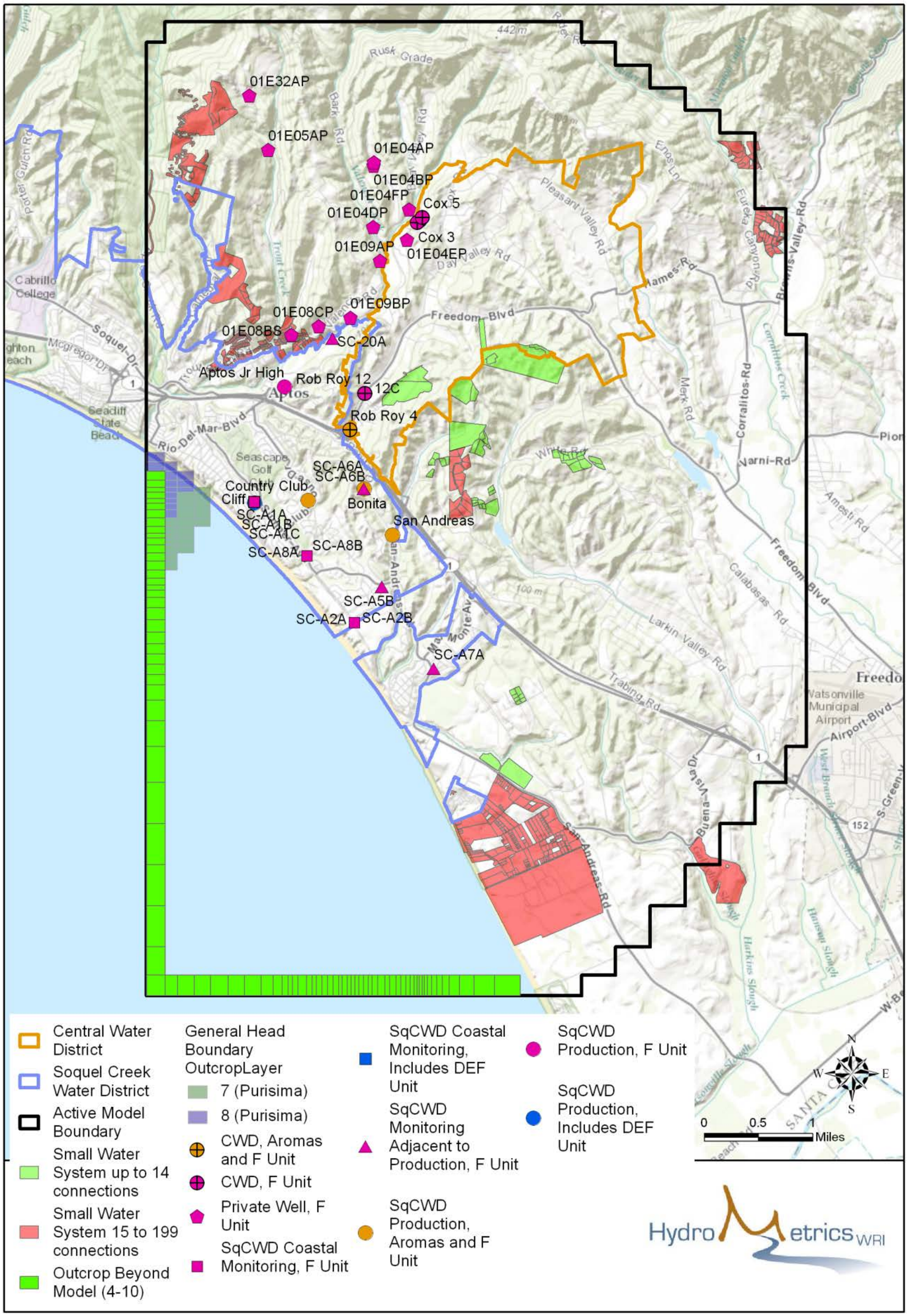

Figure 6-4. Purisima Calibration Wells and Seabed Outcrop Layers

Table 6-2. Summary of Measurements Excluded from Calibration Results

Well	Excluded Date Range	Excluded Elevation Range (ft msl)	Excluded Count	Explanation for Exclusion	On Hydrographs
Cox 5	10/1988-12/1989	172.75	6	DTW=0	No
Rob Roy 4	2/1989-6/1992	-6.5 – 8.0	97	No explanation for sustained drop (not observed in Rob Roy 10)	Yes
Rob Roy 10	10/1988-4/1999	156.0	6	DTW=0	No
Rob Roy 12A	7/1993-6/1995	19.4 – 24.2	9	20+ feet lower than rest of record	Yes
Aptos	8/25/2008	0.0	1	~10 feet lower than next lowest record	Yes
Bonita	6/1984-10/1994	-8.8 – -5.8 and 29.2 – 33.2	5	~10 feet lower or higher than rest of record	Yes
San Andreas	10/11/2002	-3.5	1	~7 feet lower than rest of record	Yes
Seascape	1/26/2008	-4.5	1	8+ feet lower than measurements since 2005	Yes
SC-A1C	3/1/1983	18.3	1	~10 feet higher than rest of record	No
SC-A4A	5/1/1999	-7.4	1	~5 feet lower than measurements since 1991	Yes
SC-A4B	5/1999-6/1999	-5.1 – -8.3	3	~3 feet lower than rest of record	Yes
SC-A4C	8/21/2009	-25.2	1	>25 feet lower than rest of record	No
SC-A5A	2/1989, 7-9/1991	13.5 – 21.6	4	>5 feet higher than rest of record	Yes
SC-A5B	2/1989, 7-9/1991, 8/2008, 8/2009	14.9 – 24.2, -14.6 – -28.0	6	>8 feet higher and >12 feet lower than rest of record	Yes
SC-A5C	2/1989, 7-9/1991, 9/2007-7/2009	14.0 – 23.5, -5.1 – -1.7	8	>5 feet higher than rest of record and >6 feet lower than measurements since 2002	Yes
SC-A5D	2/1989, 7-9/1991	14.3 – 21.9	4	>4 feet higher than rest of record	Yes
SC-A6C	12/2004-8/2009	35.8 - 217	30	High water levels likely not representative of regional aquifer	Yes for <60
PVWMA B	5/1998	-10.3	1	>10 feet lower than rest of record	No

Well	Screen Intervals: Number	Max Elevation (ft msl)	Min Elevation (ft msl)	Total Length (ft)	Screen Percentages by Model Layer: 1	2	3	4	5	6	7	8	9	10
Cox 2	1	195		140	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%
Cox 3	1	155	7.6	147.4	0%	0%	0%	0%	0%	0%	78%	22%	0%	0%
Cox 5	2	121.6	59.5	49.5	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%
Rob Roy 4	1	6	-204	210	0%	0%	0%	0%	0%	59%	34%	7%	0%	0%
Rob Roy 10	3	-4	-109	50	0%	0%	0%	0%	0%	100%	0%	0%	0%	0%
Rob Roy 12	3	-58	-358	160	0%	0%	0%	0%	0%	31%	32%	6%	31%	0%
Rob Roy 12A	1	47.65	27.65	20	0%	0%	0%	0%	51%	49%	0%	0%	0%	0%
Rob Roy 12B	1	-52.37	-72.37	20	0%	0%	0%	0%	0%	100%	0%	0%	0%	0%
Rob Roy 12C	2	-152.33	-322.33	40	0%	0%	0%	0%	0%	0%	50%	0%	50%	0%
Black	4	40	-100	50	0%	0%	0%	0%	20%	80%	0%	0%	0%	0%
Aptos	1	-60	-197	137	0%	0%	0%	0%	0%	0%	0%	65%	35%	0%
Bonita	7	-103.83	-496.83	241	0%	0%	0%	0%	0%	19%	34%	23%	24%	0%
Cliff	1	-130.75	-330.75	200	0%	0%	0%	0%	0%	0%	0%	30%	61%	9%
Country Club	4	-56.87	-297.87	169	0%	0%	0%	0%	0%	28%	22%	50%	0%	0%
San Andreas	3	-105.5	-449.5	240	0%	0%	0%	0%	0%	36%	25%	39%	0%	0%
Seascape	3	-141.46	-272.46	76	0%	0%	0%	0%	0%	100%	0%	0%	0%	0%
Altivo	3	-154.44	-304.44	100	0%	0%	0%	0%	31%	69%	0%	0%	0%	0%
Sells	3	-89.48	-320.48	120	0%	0%	0%	0%	75%	25%	0%	0%	0%	0%
SC-A1A	2	-395.24	-455.24	30	0%	0%	0%	0%	0%	0%	0%	0%	0%	100%
SC-A1B	2	-195.24	-330.24	40	0%	0%	0%	0%	0%	0%	0%	0%	57%	43%
SC-A1C	1	-105.24	-125.24	20	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%
SC-A8A	1	-387.98	-407.98	20	0%	0%	0%	0%	0%	0%	0%	0%	100%	0%
SC-A8B	1	-318.24	-338.24	20	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%
SC-A8C	1	-67.9	-87.9	20	0%	0%	0%	0%	0%	100%	0%	0%	0%	0%
SC-A2A	1	-332.76	-352.76	20	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%
SC-A2B	1	-292.76	-312.76	20	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%
SC-A2C	1	-12.76	-32.76	20	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
SC-A3A	1	-186.91	-206.91	20	0%	0%	0%	0%	72%	28%	0%	0%	0%	0%
SC-A3B	2	-126.91	-166.91	30	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
SC-A3C	1	-21.91	-41.91	20	0%	0%	0%	100%	0%	0%	0%	0%	0%	0%
SC-A4A	1	-334.11	-354.11	20	0%	0%	0%	0%	0%	100%	0%	0%	0%	0%
SC-A4B	1	-294.11	-314.11	20	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
SC-A4C	2	-104.11	-224.11	40	0%	0%	0%	50%	50%	0%	0%	0%	0%	0%
SC-A4D	1	-34.11	-54.11	20	0%	0%	88%	12%	0%	0%	0%	0%	0%	0%
SC-A5A	1	-474.68	-494.68	20	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%
SC-A5B	1	-404.68	-424.68	20	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%
SC-A5C	3	-134.68	-254.68	60	0%	0%	0%	0%	0%	100%	0%	0%	0%	0%
SC-A5D	1	-34.68	-54.68	20	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
SC-A6A	1	-467.39	-477.39	10	0%	0%	0%	0%	0%	0%	0%	0%	100%	0%
SC-A6B	1	-227.64	-230.64	3	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%
SC-A6C	1	-117.39	-120.39	3	0%	0%	0%	0%	0%	100%	0%	0%	0%	0%
SC-A7A	1	-489.37	-492.37	3	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%
SC-A7B	1	-369.36	-372.36	3	0%	0%	0%	0%	0%	100%	0%	0%	0%	0%
SC-A7C	1	-209.36	-212.36	3	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
SC-A7D	1	-89.35	-92.35	3	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
SC-20A	1	-180.84	-200.84	20	0%	0%	0%	0%	0%	0%	0%	0%	100%	0%
SC-20B	1	14.02	-15.98	30	0%	0%	0%	0%	0%	0%	30%	70%	0%	0%
703	1	-123.5	-223.5	100	0%	0%	0%	16%	84%	0%	0%	0%	0%	0%
99141	1	-156	-206	50	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
01E22BS	1	-30	-150	120	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
01E22AS	1	-40	-80	40	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
01E15AS	1	-41	-42	1	0%	0%	0%	0%	100%	0%	0%	0%	0%	0%
01E09AP	1	9	-11	20	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%
01E08CP	1	108	-142	250	0%	0%	0%	0%	0%	0%	29%	68%	3%	0%
01E08BS	1	-101	-102	1	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%
01E05AP	1	265	125	140	0%	0%	0%	0%	0%	0%	0%	0%	100%	0%
01E04FP	1	162	142	20	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%
01E04EP	1	102	101	1	0%	0%	0%	0%	0%	0%	100%	0%	0%	0%
01E04DP	1	120	35	85	0%	0%	0%	0%	0%	0%	24%	76%	0%	0%
01E04BP	1	297	213	84	0%	0%	0%	0%	0%	0%	0%	100%	0%	0%
01E04AP	1	283	-7	290	0%	0%	0%	0%	0%	0%	0%	79%	21%	0%
01E32AP	1	583	363	220	0%	0%	0%	0%	0%	0%	0%	33%	67%	0%
01E09BP	1	-145	-205	60	0%	0%	0%	0%	0%	0%	0%	35%	65%	0%

Table 6-3. Screen Layer Percentages for Calibration Wells

6.3 HYDROGEOLOGIC PROPERTY ESTIMATES

Several pumping tests at CWD and SqCWD wells provide information about aquifer properties. These tests include constant rate aquifer tests where a well is pumped at a constant rate and aquifer response is monitored at monitoring wells. Other tests include dye tracer studies during pumping that measure the flow in the well with depth. One measure of calibration is the degree to which the calibrated model simulates the hydrogeologic property estimates. Because these are only estimates, the model calibration is not expected to match the properties estimated by these tests. However, the general magnitude and pattern of properties should be reflected in the calibrated model.

Aquifer transmissivity is estimated based on drawdowns observed in monitoring wells during constant rate tests. Based on screen intervals, hydraulic conductivity is estimated. These estimates for hydraulic conductivity were used as estimates of hydraulic conductivities at pilot points near the test location. Matching these values was added as objectives during the calibration process along with water level observations. Two tests provided hydraulic conductivity estimates: a 6 hour aquifer test conducted at the Cox #3 well on September 12, 2012 (Section 2 of this report) and a 21.3 hour aquifer test conducted at the Polo Grounds well on May 28-29, 2009 (HydroMetrics LLC, 2009b) as summarized in Table 6-4.

Table 6-4. Hydraulic Conductivities Estimated from Constant Rate Aquifer Tests Used in Calibration

Pumped Well	Monitoring Wells	Analytical Method	Transmissivity (ft^2/d)	Screen Length	Hydraulic Conductivity (ft/d)	Model Layers
Cox #3	Cox #2, #5	Cooper-Jacob	470-488	143	3.4	7-8
Polo Grounds	SC-20A	Multi-Layer Unsteady Model	177	50	3.5	8
	SC-20B		1,986	40	49.7	9

Dye tracer flow profiles measure cumulative flow at different depths in the well during pumping. Based on these measurements, flow contribution by screen interval can be estimated. The estimated flow contribution percentages are assumed to reflect relative transmissivities between screen intervals. Based on screen interval lengths, relative hydraulic conductivities are calculated and used as estimates of hydraulic conductivity at pilot points near the test location. Matching these relative conductivities was given less importance than the estimates from constant rate tests due to greater uncertainty of the estimates. Hydraulic conductivity estimates from dye tracer flow profiles conducted at four wells were used: CWD's Rob Roy #12 well tested in 2012 (Section 2 of this report) and SqCWD's Bonita, San Andreas, and Altivo wells tested in 2008 (HydroMetrics LLC, 2009c). Table 6-5 summarizes the data from the dye tracer flow profiles used for calibration. Relative hydraulic conductivities shown in Table 6-5 are estimated based on flow distribution divided by screen interval percentage for each layer. To provide a basis for estimating relative hydraulic conductivities, the relative hydraulic conductivity of layer 6 is always set at one. Relative hydraulic conductivities of all other layers represent the estimated hydraulic conductivity of the layer divided by the estimated hydraulic conductivity of layer 6.

Table 6-5. Relative Hydraulic Conductivities Estimated from Dye Tracer Flow Profiles Used in Calibration

Pumped Well	Layer	Flow Distribution	Screen Interval Percentage	Relative Hydraulic Conductivity
Rob Roy 12	6	26%	31%	1
	7 and 8	34%	37%	1.09
	9	40%	31%	1.54
Bonita	6	29%	19%	1
	7	39%	34%	0.76
	8	20%	23%	0.60
	9	12%	24%	0.33
San Andreas	6	68%	36%	1
	7	23%	25%	0.49
	8	9%	39%	0.12
Altivo	5	45%	31%	1.85
	6	55%	69%	1

6.4 Calibration Results

6.4.1 Calibrated Property Values

Aquifer property values are adjusted during model calibration to improve the model's ability to simulate known conditions. Calibration of the model consisted of modifying the distribution and magnitude of horizontal hydraulic conductivity (Kx), vertical hydraulic conductivity (Kz via vertical anisotropy Kx/Kz), specific storage (Ss), and specific yield (Sy) values using the pilot point method discussed above. The final distributions of the aquifer property values are shown for each of the ten model layers in Figure 6-5 through Figure 6-14.

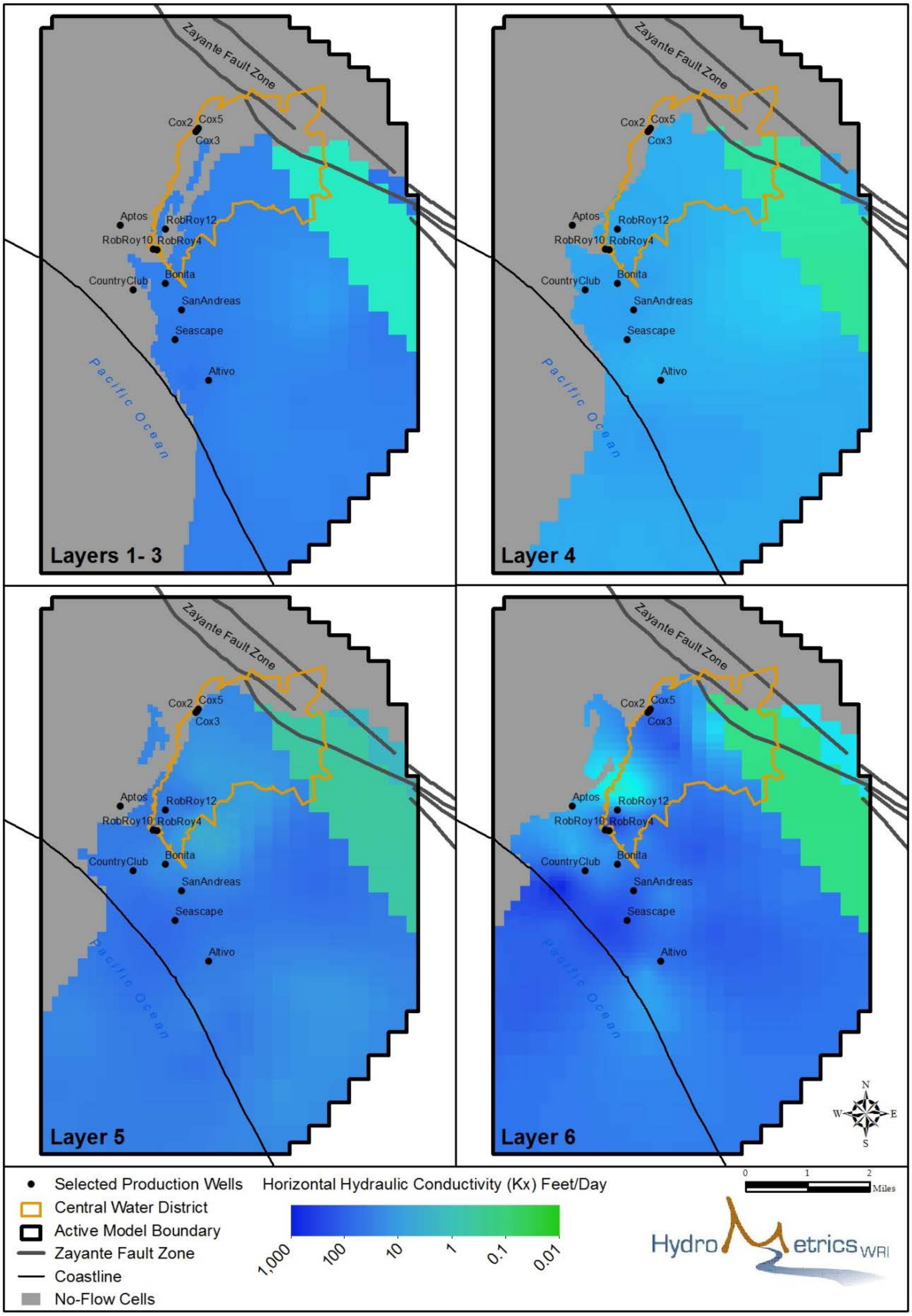

Figure 6-5. Horizontal Hydraulic Conductivity for Aromas Red Sands (Layers 1-6)

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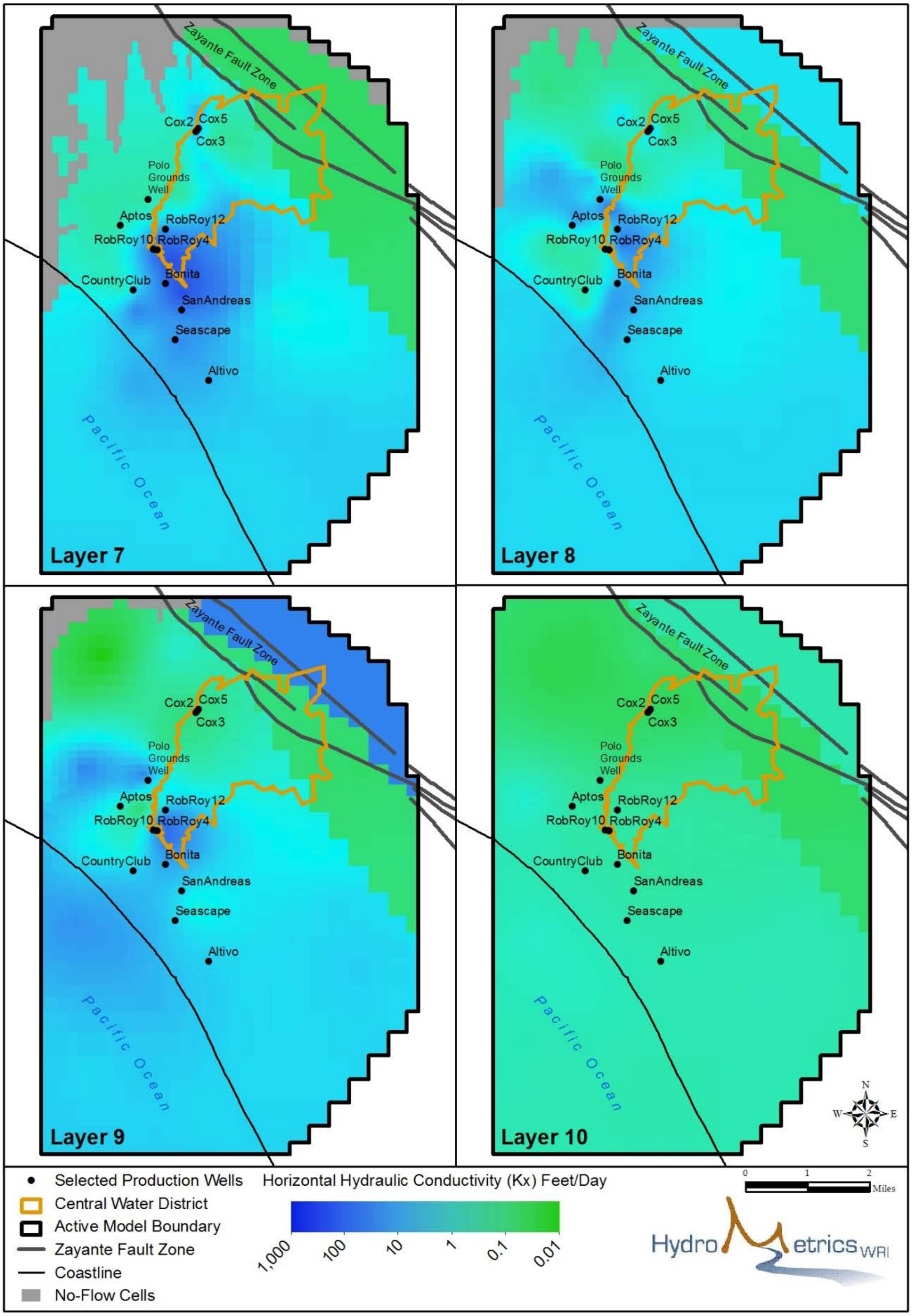

Figure 6-6. Horizontal Hydraulic Conductivity for Purisima Formation (Layers 7-10)

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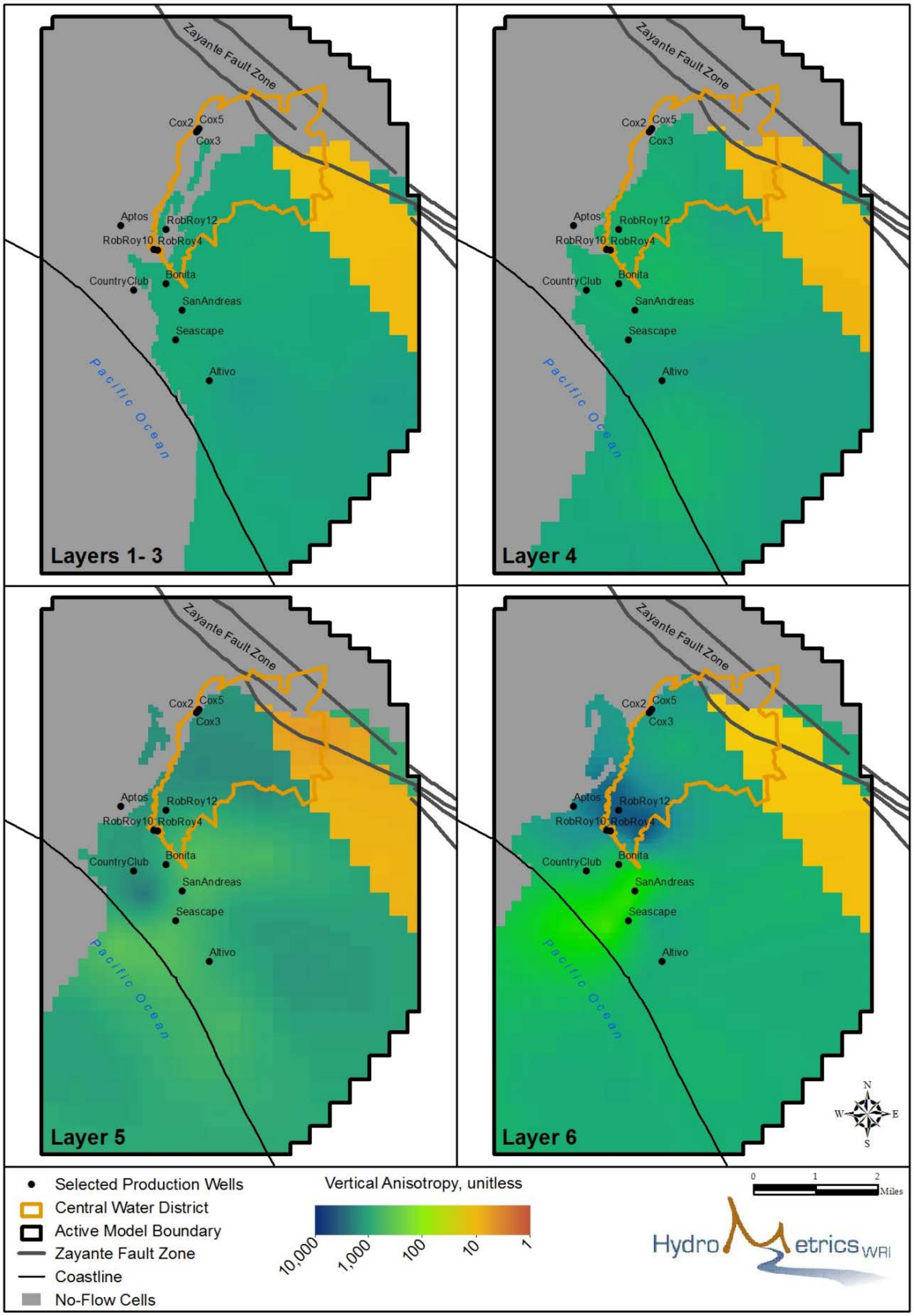

Figure 6-7. Vertical Anisotropy for Aromas Red Sands (Layers 1-6)

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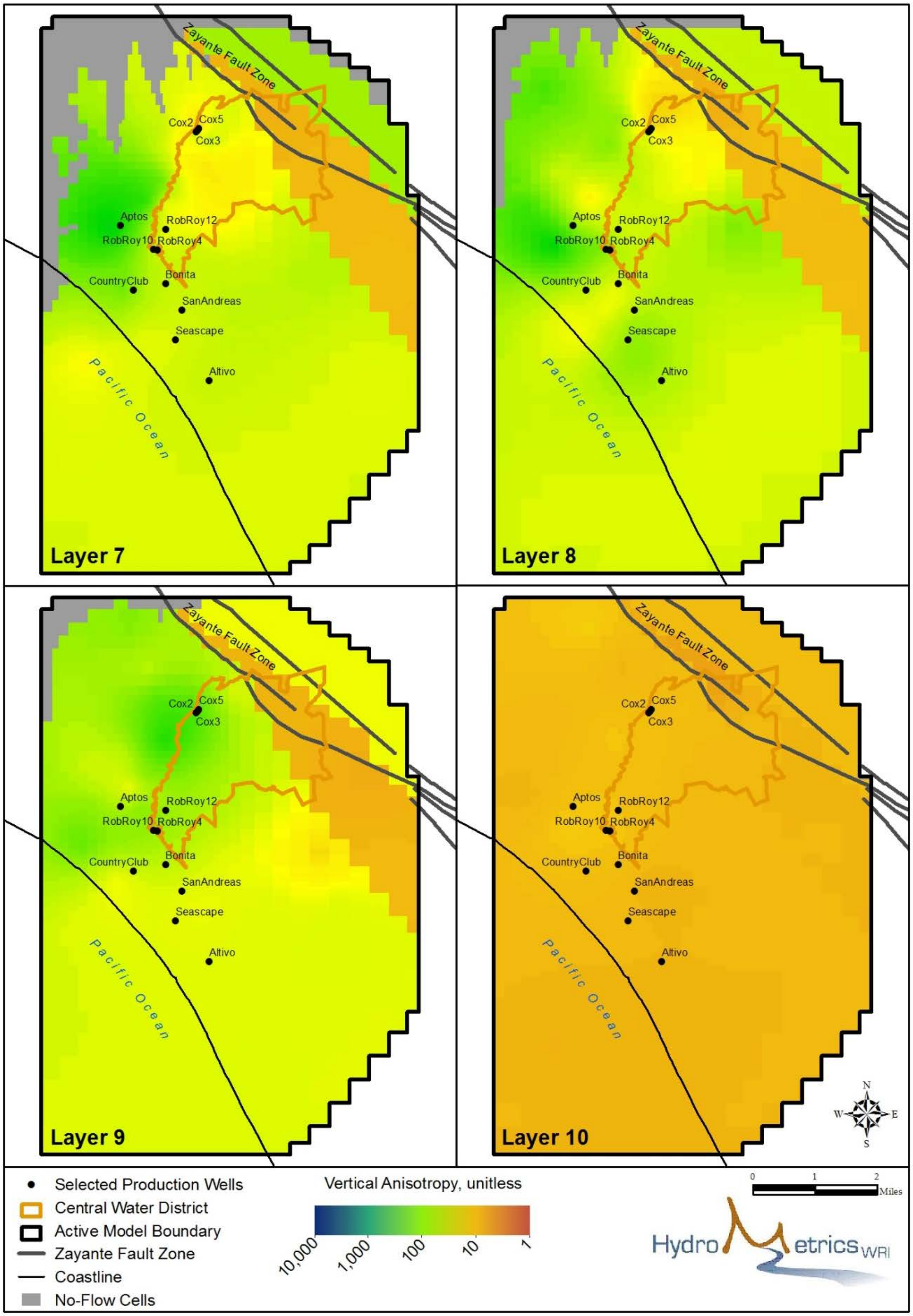

Figure 6-8. Vertical Anisotropy for Purisima Formation (Layers 7-10)

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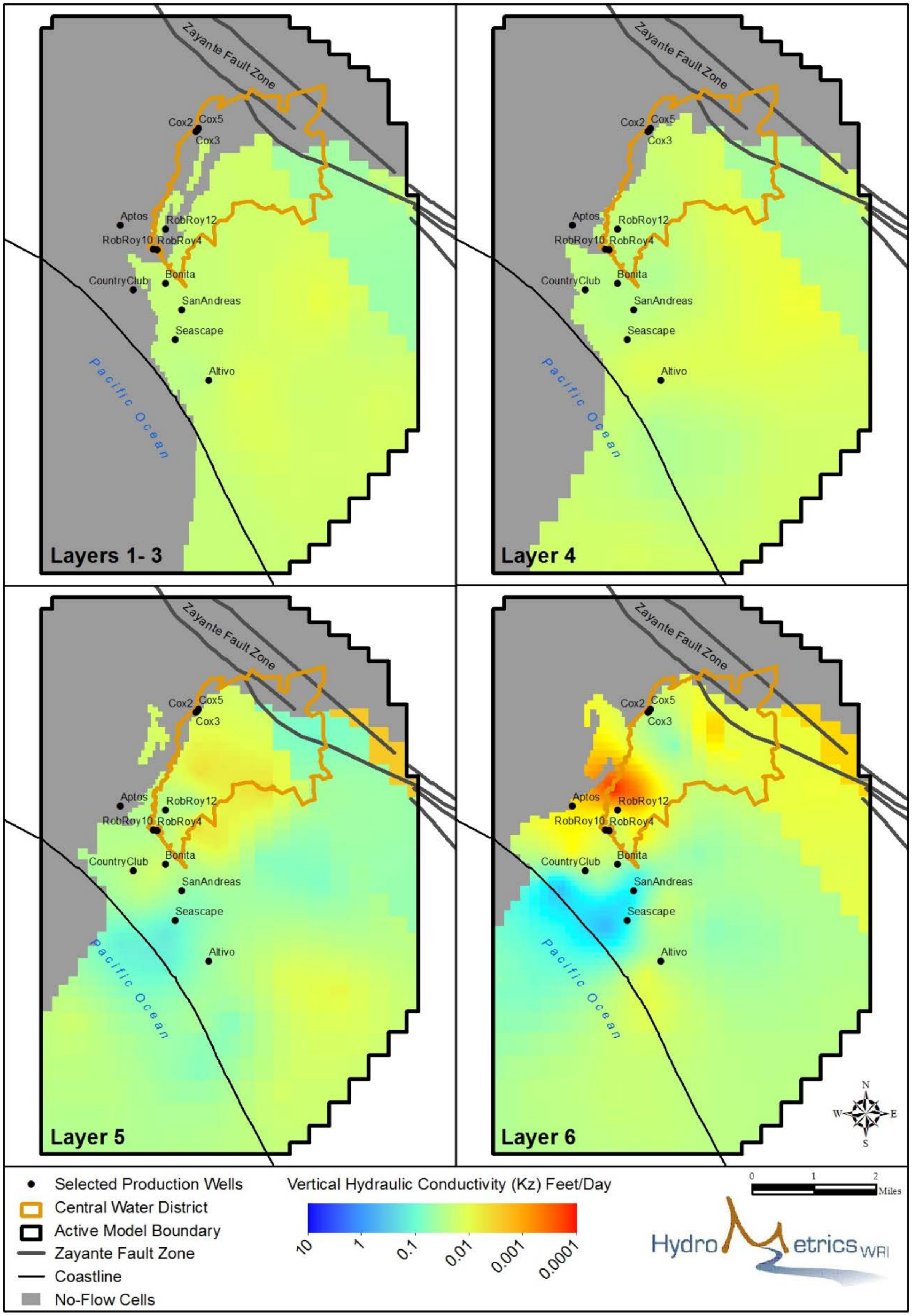

Figure 6-9. Vertical Hydraulic Conductivity for Aromas Red Sands (Layers 1-6)

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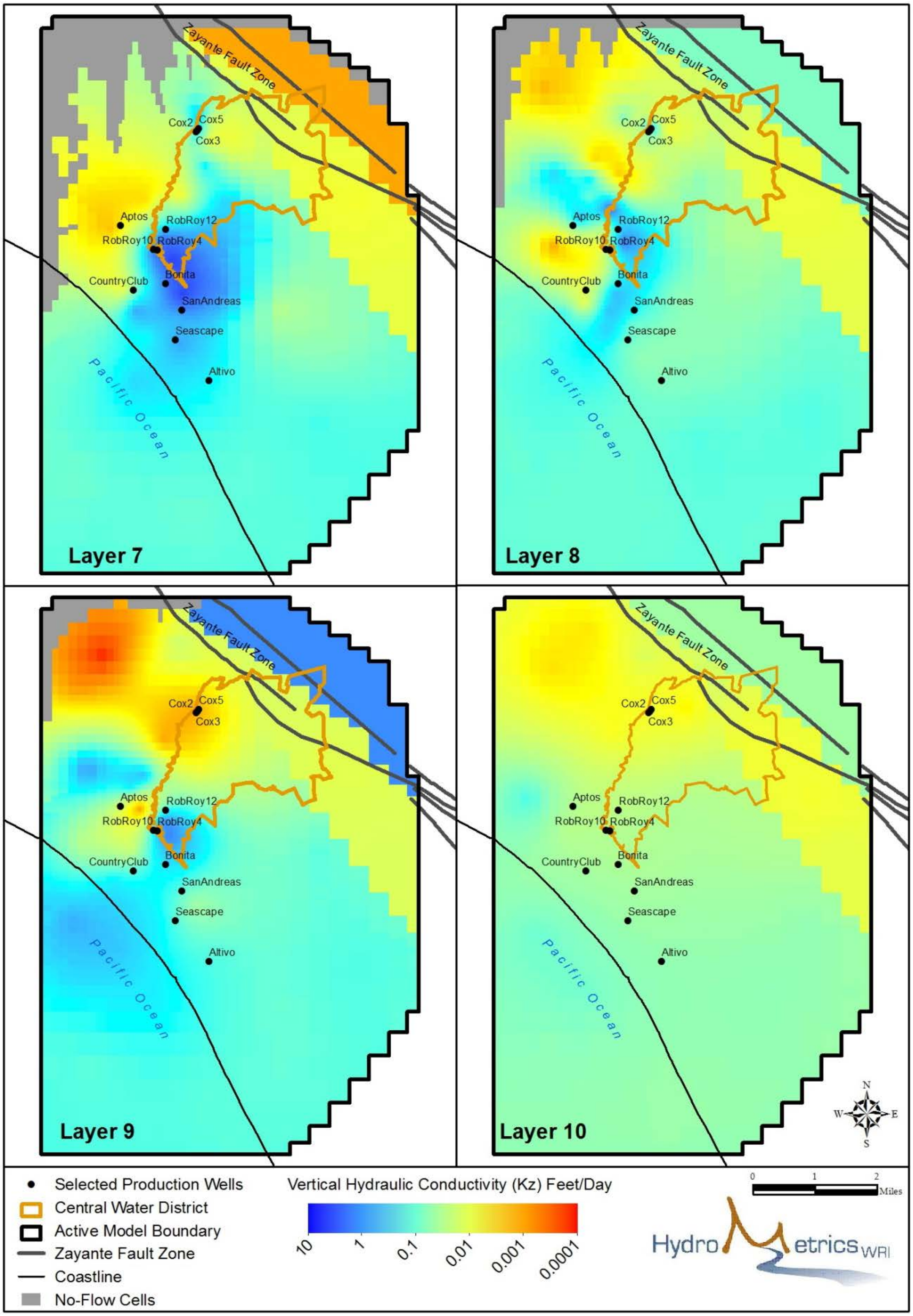

Figure 6-10. Vertical Hydraulic Conductivity for Purisima Formation (Layers 7-10)

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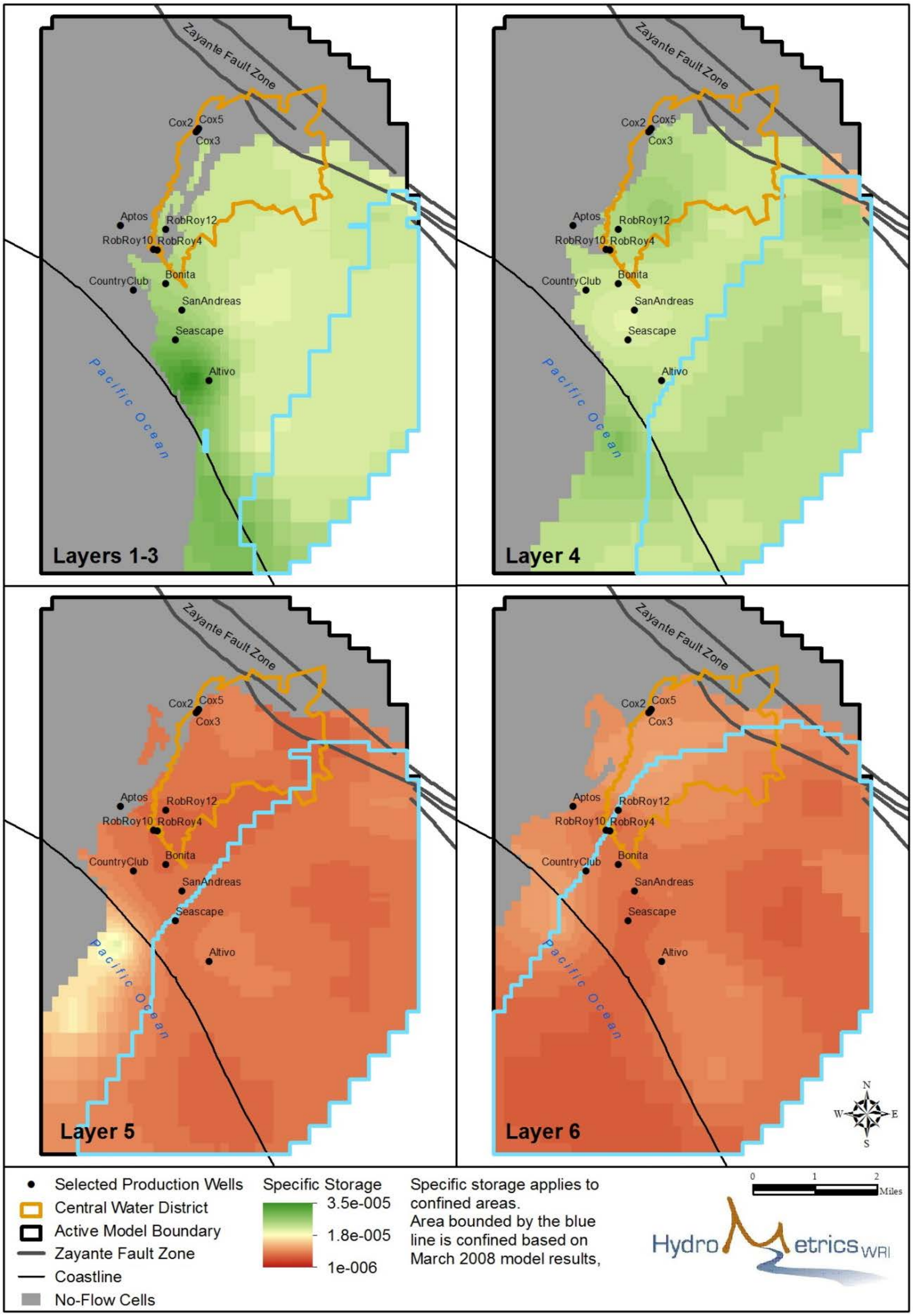

Figure 6-11. Specific Storage for Aromas Red Sands (Layers 1-6)

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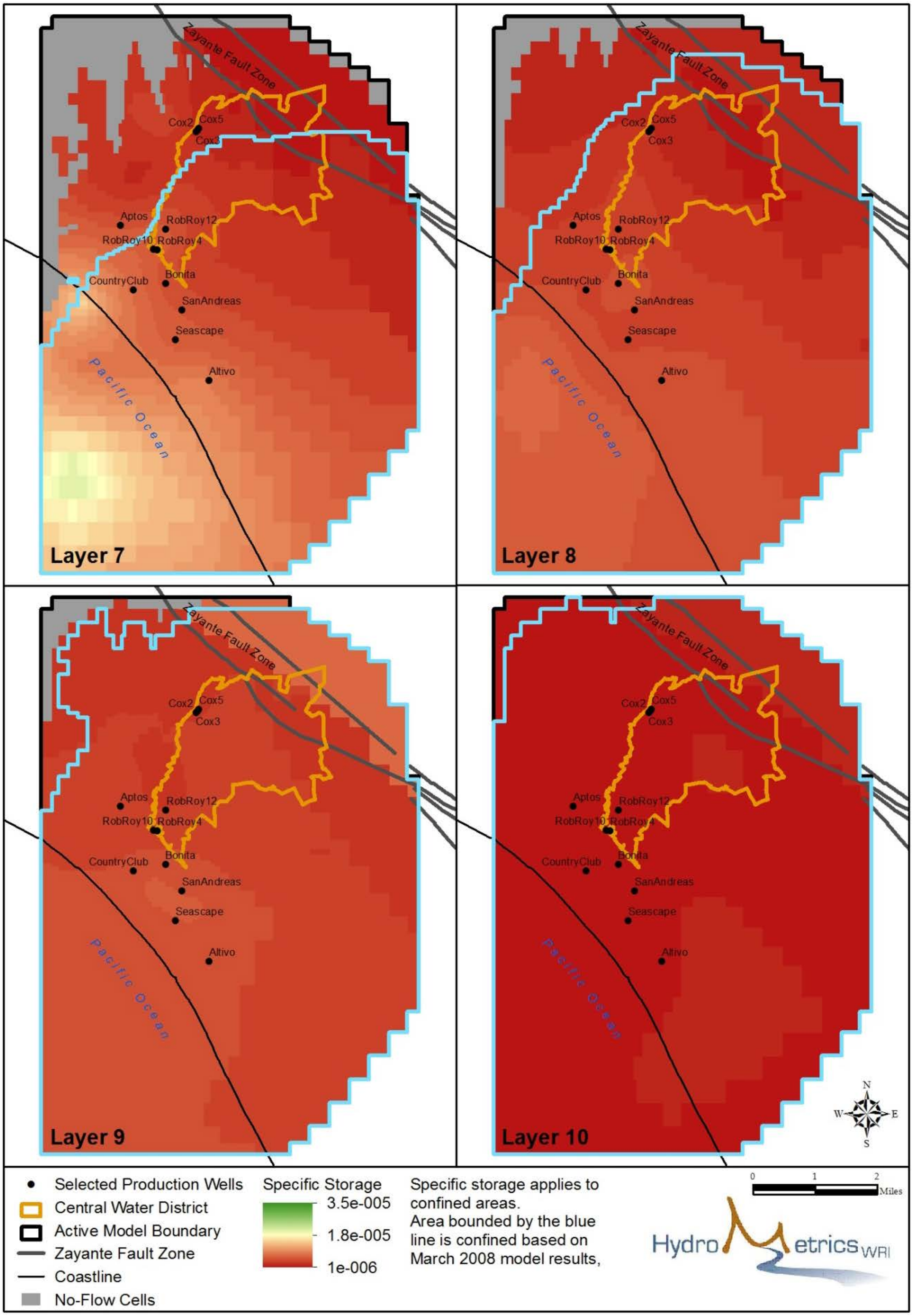

Figure 6-12. Specific Storage for Aromas Red Sands (Layers 7-10)

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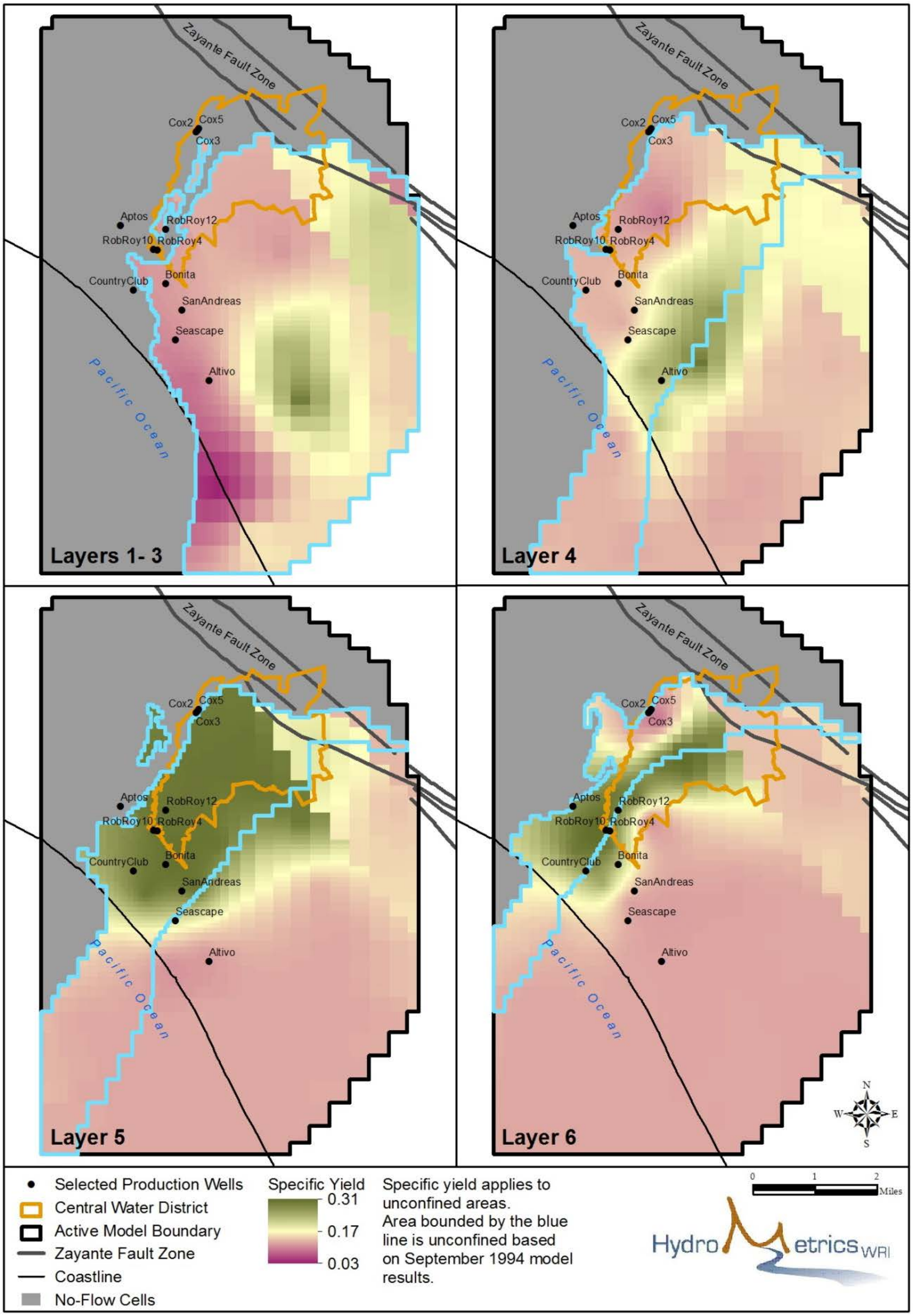

Figure 6-13. Specific Yield for Aromas Red Sands (Layers 1-6)

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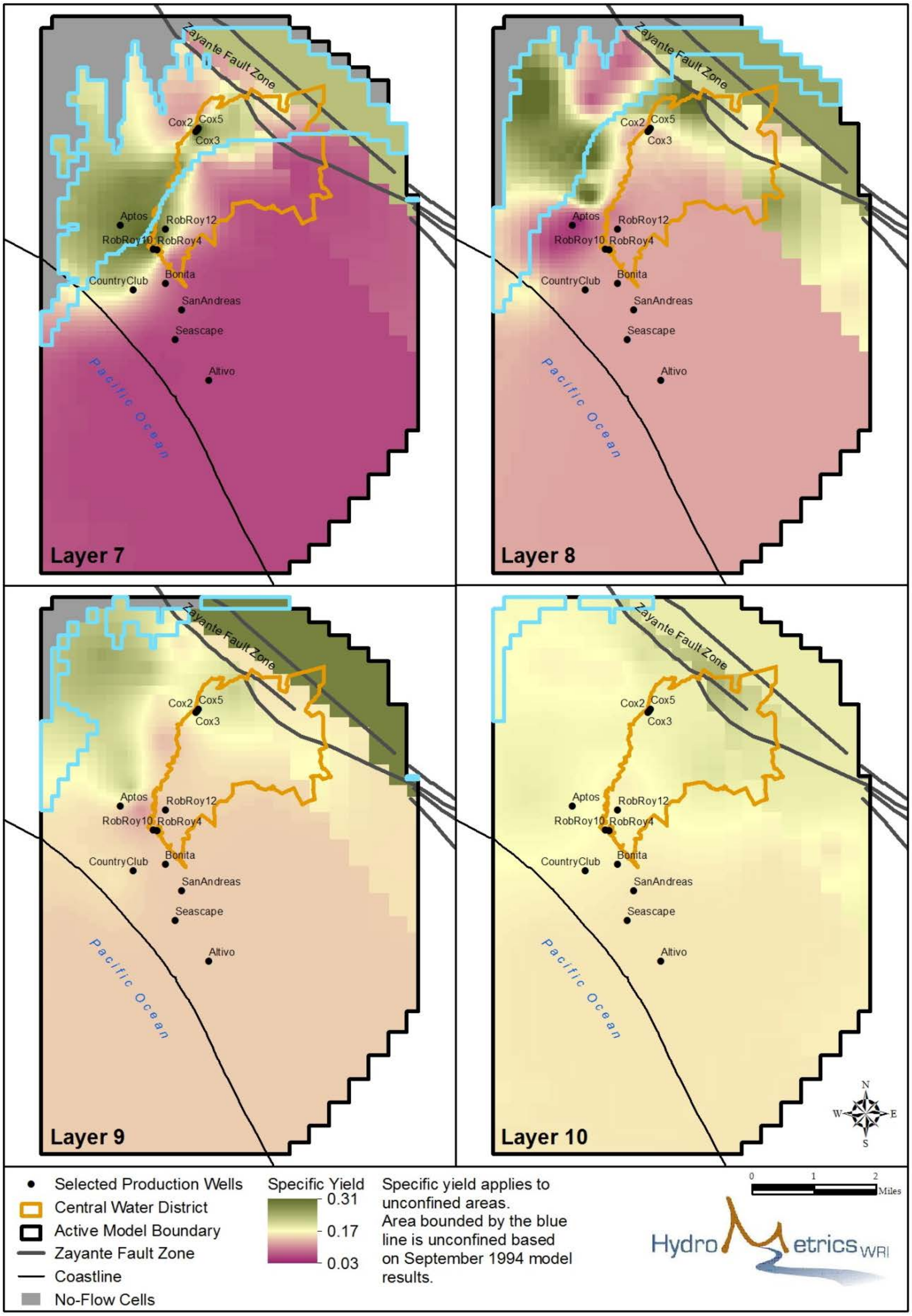

Figure 6-14. Specific Yield for Purisima Formation (Layers 7-10)

6.4.2 Calibrated Values for Seabed Outcrop Boundary Conditions

Table 6-6 shows the calibrated values for general head boundary conductances representing the model's seabed outcrop were calibrated for each layer (Figure 6-3 and Figure 6-4). The head boundaries at the model edge have lower conductances than general head boundaries in the interior because boundaries at the model edge represent an outcrop some distance beyond the model domain. Conductances for lower Aromas outcrop boundaries also are higher than conductances for the Purisima outcrop boundaries.

Table 6-6. Calibrated Seabed Outcrop General Head Boundary Conductances

Aquifer Unit	Model Layer	Conductance (ft^2/d)	
		Within Model Domain	**Beyond Model Domain**
Upper Aromas	3	848	N/A
Lower Aromas	4	1,355	148
	5	3,199	311
	6	6,710	397
Purisima F	7	1,144	34
	8	1,748	44
	9	N/A	44
Purisima DEF	10	N/A	18

6.4.3 Groundwater Elevation Results

Example maps of simulated piezometric surfaces for each model layer are displayed on Figure 6-15 through Figure 6-18. Maps of simulated piezometric surfaces are provided for September 1994 and March 2008. September 1994 is a period of relatively low groundwater elevations. March 2008 is a period of relatively high groundwater elevations.

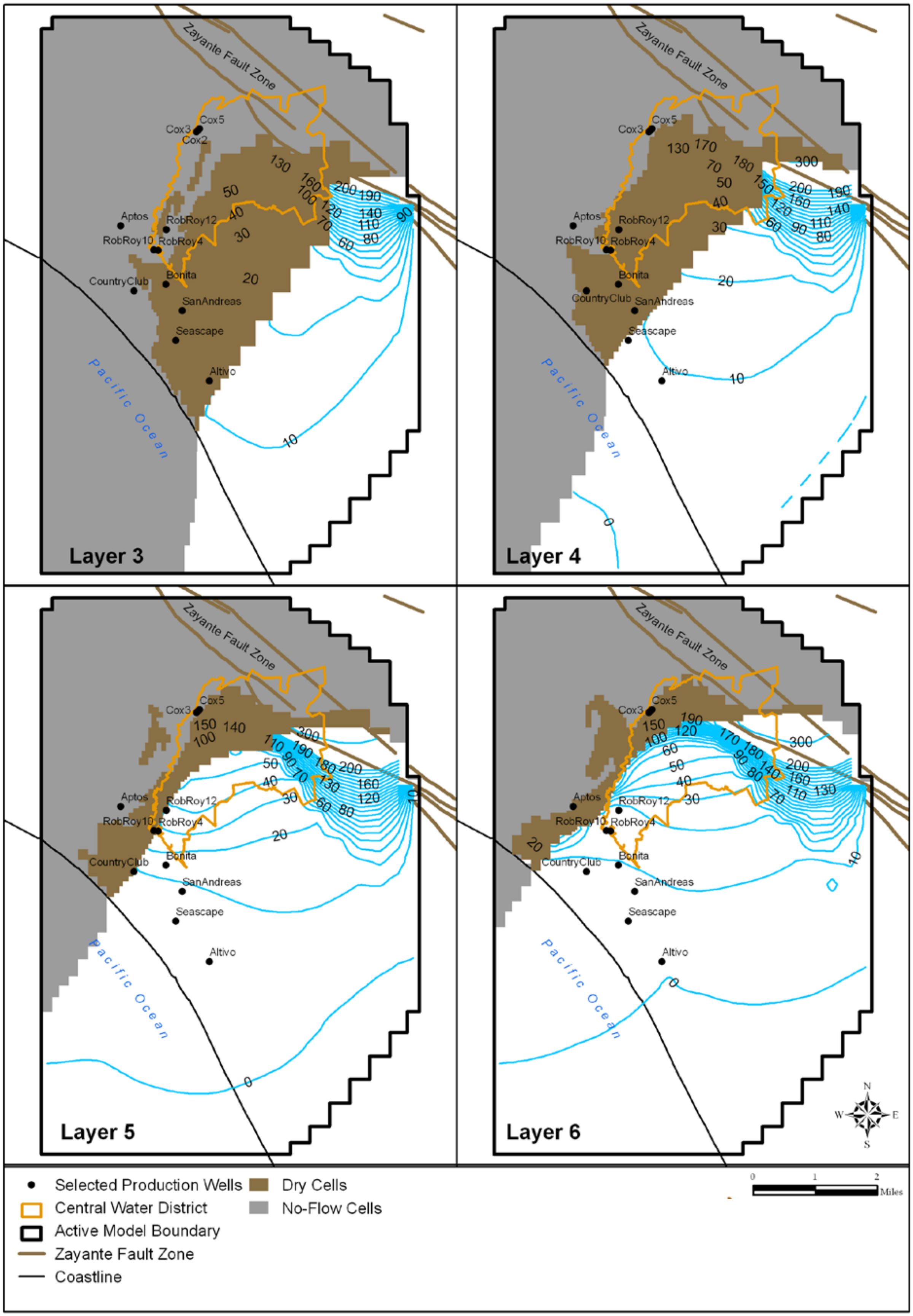

Figure 6-15. Modeled Groundwater Elevations (feet msl) in Aromas Red Sands for September 1994

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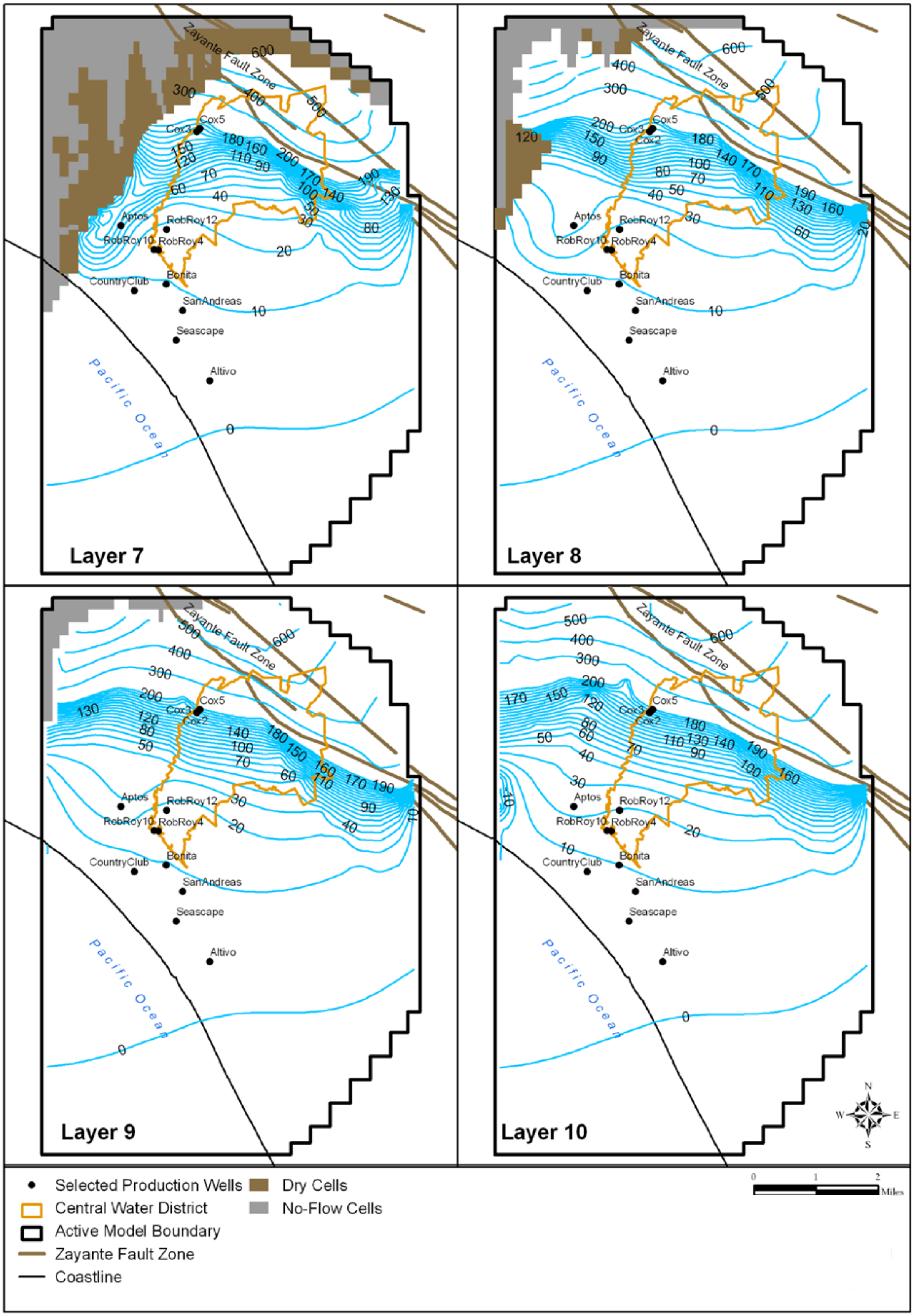

Figure 6-16. Modeled Groundwater Elevations (feet msl) in Purisima Formation for September 1994

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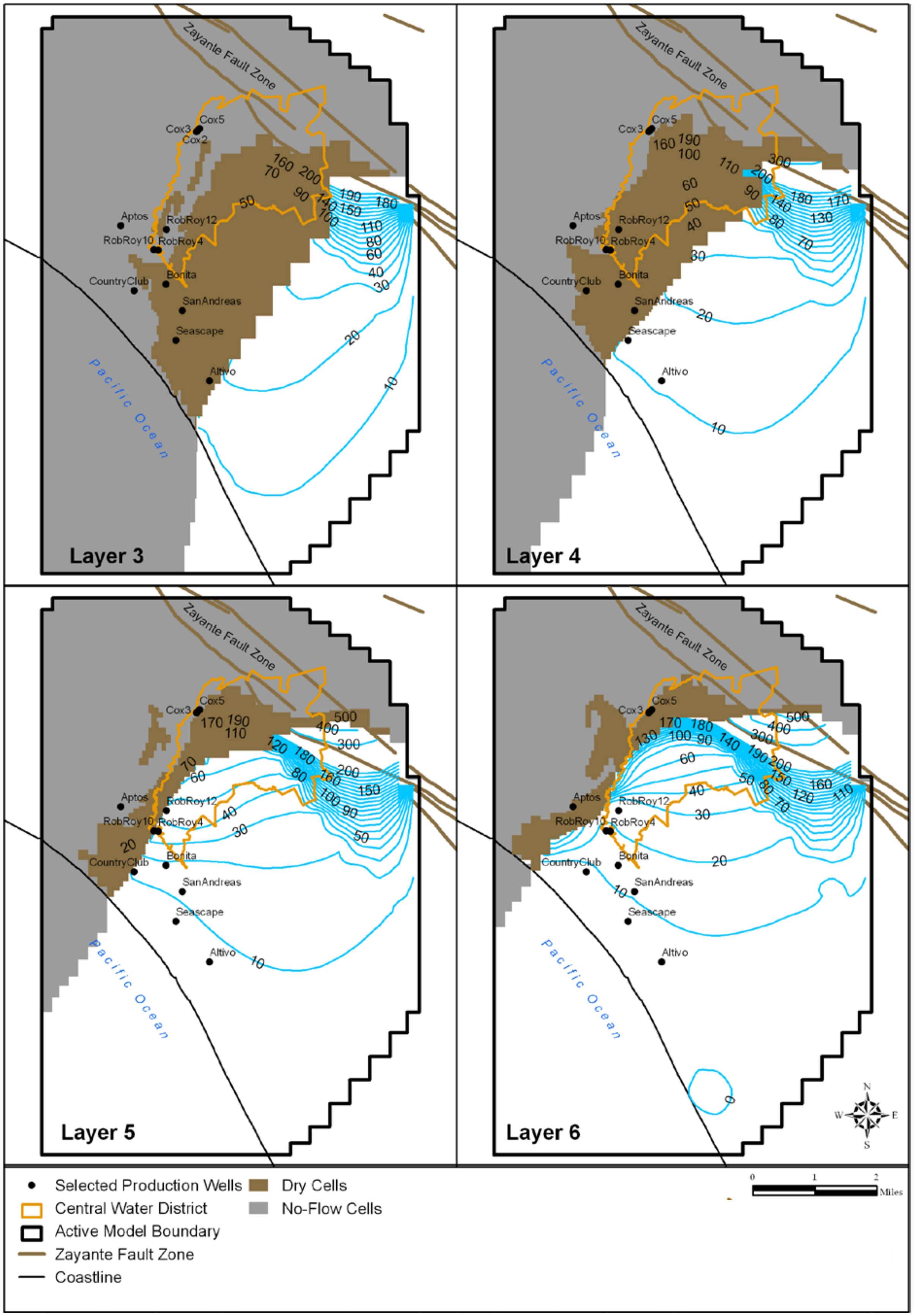

Figure 6-17. Modeled Groundwater Elevations (feet msl) in Aromas Red Sands for March 2008

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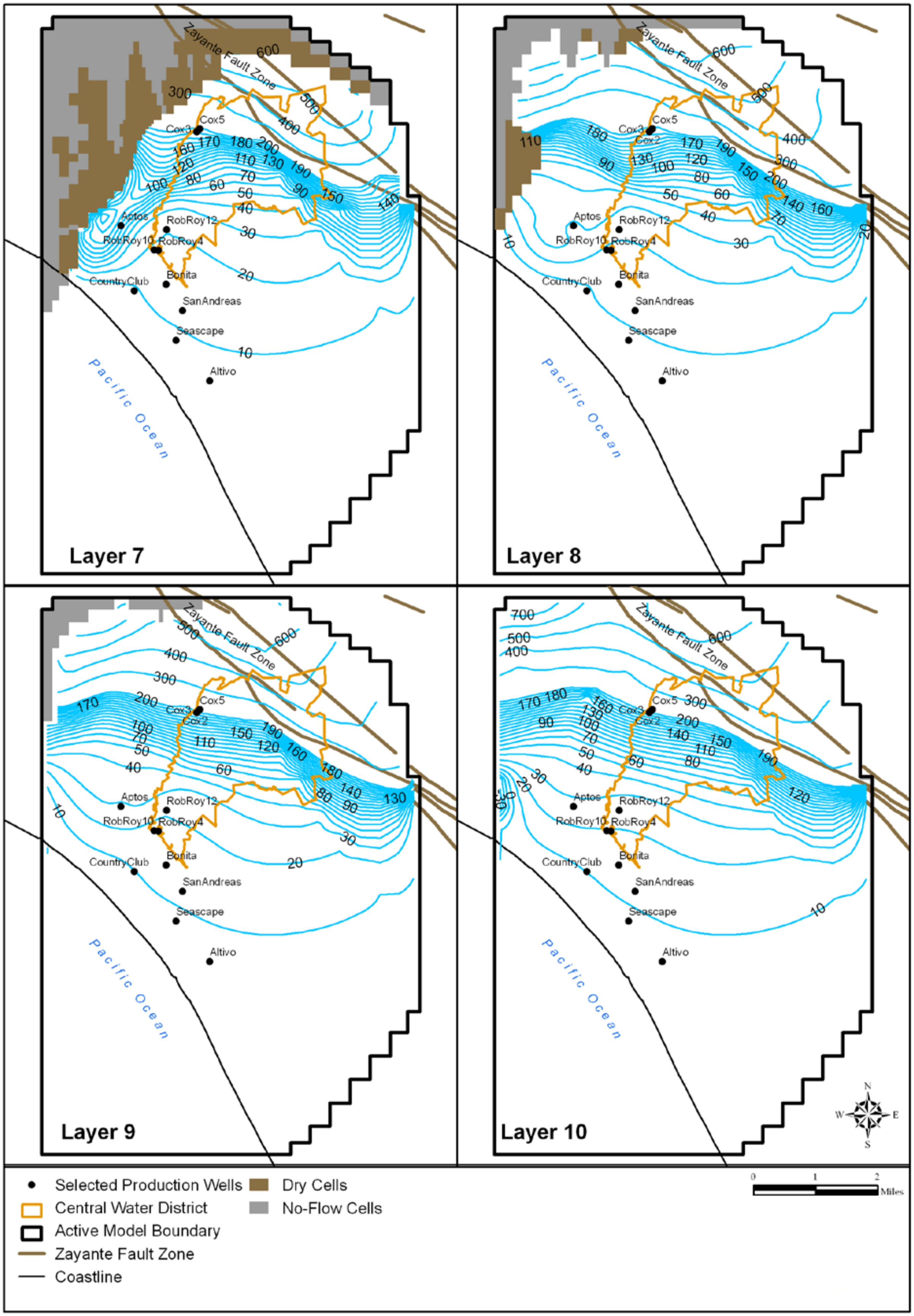

Figure 6-18. Modeled Groundwater Elevations (feet msl) in Purisima for March 2008

6.5 Calibration Evaluation

6.5.1 Comparison of Calibrated Parameter Values to Previous Conceptual Model Estimates

The conceptual model for updating CWD's groundwater model is focused on the water balance as discussed in the model setup in Section 5. For geology, the stratigraphic layering from the DWSAP model is used with layers 1 through 6 representing the Aromas Red Sands, layers 7 through 9 representing the Purisima F unit, and layer 10 representing the Purisima DEF unit (Johnson, 2009). The DSWAP model defined hydrogeologic parameters using homogeneous zones; the updated model allows for local heterogeneity within model layers. Table 6-7 compares the calibrated parameter ranges for horizontal hydraulic conductivity (Kx), vertical hydraulic conductivity (Kz), specific storage (Ss), and specific yield (Sy) for the area south of the Zayante Fault with previous estimates from Johnson et al., 2004. The 2004 estimates represent estimates for the full unit thicknesses while the model is discretized into thinner layers so there is more variation in the model.

Table 6-7. Calibrated Parameter Values South of Zayante Fault and Previous Conceptual Model Estimates

Unit (Layers)	Horizontal Hydraulic Conductivity (ft/d)		Vertical Hydraulic Conductivity (ft/d)		Specific Storage (ft^{-1})		Specific Yield	
	Model Range (Avg[1])	**2004 Estimate**	**Model Range (Avg[1])**	**2004 Estimate**	**Model Range (Avg[1])**	**2004 Estimate[2]**	**Model Range (Avg[1])**	**2004 Estimate**
Upper Aromas (1-3)	10-30 (18)	3-40	0.01-0.03 (0.02)	0.05 – 2	2×10^{-5} – 4×10^{-5} (2×10^{-5})	N/A	0.03 - 0.3 (0.15)	0.04-0.14
Lower Aromas (4-6)	0.6-200 (26)	6-50	1×10^{-4} – 0.6 (0.03)		5×10^{-6} – 4×10^{-5} (2×10^{-5})	6×10^{-8} – 4×10^{-5}	0.09 – 0.3 (0.18)	
Purisima F (7-9)	0.01-240 (8)	2-6	1×10^{-4} – 4 (0.1)	0.005 - 0.5	1×10^{-6} – 2×10^{-5} (5×10^{-6})	5×10^{-8} – 4×10^{-5}	0.03 – 0.3 (0.09)	0.01-0.10
Purisima DEF (10)	0.04-1.4 (0.5)	2-6	3×10^{-3} – 0.1 (4×10^{-2})		5×10^{-7} – 2×10^{-6} (1×10^{-6})	3×10^{-8} – 2×10^{-5}	0.06 – 0.07 (0.06)	

[1] Area-weighted mean

[2] Storativity estimate divided by estimated average thickness for unit (Johnson et al., 2004)

6.5.2 Comparison of Calibrated Hydraulic Conductivities to Estimates from Aquifer Tests and Dye Tracer Tests

During calibration, modeled hydraulic conductivities at pilot points near pumping test locations (Figure 6-1 and Figure 6-2) were compared to hydraulic conductivities estimated from constant rate aquifer tests (Table 6-4) and relative hydraulic conductivities estimated from dye tracer flow profiles (Table 6-5).

Table 6-8 shows the hydraulic conductivities modeled at pilot points compared to hydraulic conductivities estimated from constant rate aquifer tests at the Cox #3 and Polo Grounds wells. This comparison shows a good match between the calibrated model and estimates.

Table 6-8. Comparison of Modeled Hydraulic Conductivities to Estimates from Constant Rate Aquifer Tests

<table>
<tr><th>Pumped Well</th><th>Estimated Hydraulic Conductivity (ft/s)</th><th>Model Layer</th><th>Range of Modeled Hydraulic Conductivity (ft/s)</th></tr>
<tr><td rowspan="2">Cox #3</td><td rowspan="2">3.4</td><td>7</td><td>3.9-5.3</td></tr>
<tr><td>8</td><td>3.1-3.4</td></tr>
<tr><td rowspan="2">Polo Grounds</td><td>3.5</td><td>8</td><td>3.6</td></tr>
<tr><td>49.7</td><td>9</td><td>49</td></tr>
</table>

Table 6-9 shows the hydraulic conductivities modeled at pilot points compared to relative hydraulic conductivities estimated at dye tracer profiles. The modeled relative conductivities in these locations do not consistently match the estimates from the dye tracer profiles. Matching these relative conductivities was given less importance than the conductivities from constant rate tests due to greater uncertainty of the estimates. Calibrating to groundwater levels was given priority in these locations.

Table 6-9. Comparison of Modeled Hydraulic Conductivities to Relative Hydraulic Conductivities Estimated from Dye Tracer Flow Results

Pumped Well	Layer	Estimated Relative Hydraulic Conductivity	Modeled Hydraulic Conductivities (ft/s)	Modeled Relative Hydraulic Conductivities
Rob Roy 12	6	1	118	1
	7	1.09	89	0.75
	8	1.09	86	0.73
	9	1.54	87	0.73
Bonita	6	1	24	1
	7	0.76	20-26	0.83-1.1
	8	0.60	22-24	0.92-1.0
	9	0.33	12	0.51
San Andreas	6	1	59	1
	7	0.49	32-33	0.56-0.57
	8	0.12	8.8	0.15
Altivo	5	1.85	38	1.4
	6	1	26	1

6.5.3 Groundwater Elevation Calibration

Flow model calibration is commonly evaluated by comparing simulated water elevations with observed groundwater elevations from monitoring and production wells. Hydrographs of simulated groundwater elevations should generally match the trends and fluctuations observed in measured hydrographs. Furthermore, the average errors between observed and simulated groundwater elevations should be relatively small and unbiased. The target well locations used for calibration of the regional groundwater flow model are shown in Figure 6-3 and Figure 6-4. For wells screened over multiple model layers, simulated groundwater levels in each of the layers are weighted by layer transmissivity and averaged before comparing with measured data.

Example hydrographs showing both observed and simulated groundwater elevations are shown in Figure 6-19 through Figure 6-28. These example hydrographs were chosen to demonstrate the model's accuracy in various parts of the Aromas area of the Soquel-Aptos Basin. The hydrographs show that the model accurately simulates both the magnitude of groundwater fluctuations and trends observed in monitoring well data for much of the area. However, the model does not match trends observed in monitoring well data in the Seascape area and areas further southeast as exemplified by the hydrograph of monitoring well SC-A2 (Figure 6-27). The complete set of hydrographs showing both observed and simulated groundwater elevations are included in an Attachment to this report.

Various graphical and statistical methods can be used to demonstrate the magnitude and potential bias of the calibration errors. Figure 6-29 shows simulated groundwater elevations plotted against observed groundwater elevations 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 groundwater level differences, the results will diverge from this 45º line. The line drawn on Figure 6-29 demonstrates that the results lie close to a 45º line, suggesting that the model results are not biased towards overestimating or underestimating average groundwater level differences.

Figure 6-29 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 groundwater elevations for all data on Figure 6-29,

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

Where h_m is the measured groundwater elevation, h_s is the simulated groundwater elevation, and n is the number of observations.

The mean absolute error is the average of the absolute differences between measured and simulated groundwater elevations.

$$MAE = \frac{1}{n}\sum_{i=1}^{n}\left|h_m - h_s\right|_i$$

The standard deviation of the errors is one measure of the spread of the errors around the 45º line in Figure 6-29. The population standard deviation is used for these calculations.

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

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 6-29, and is calculated as the square root of the average squared errors.

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

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 RMSE should be less than 10% of the total head range in the model. The RMSE of 4.0 feet is approximately 0.7% of the total head range of 609 feet. A second general rule that is occasionally used is that the mean absolute error should be less than 5% of the total head range in the model. The mean absolute error of 2.9 feet is approximately 0.4% 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 6-30. This figure is a graph of observed groundwater elevations versus model residual (simulated elevation minus observed elevation). 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 6-30 show that the calibrated model results are generally unbiased.

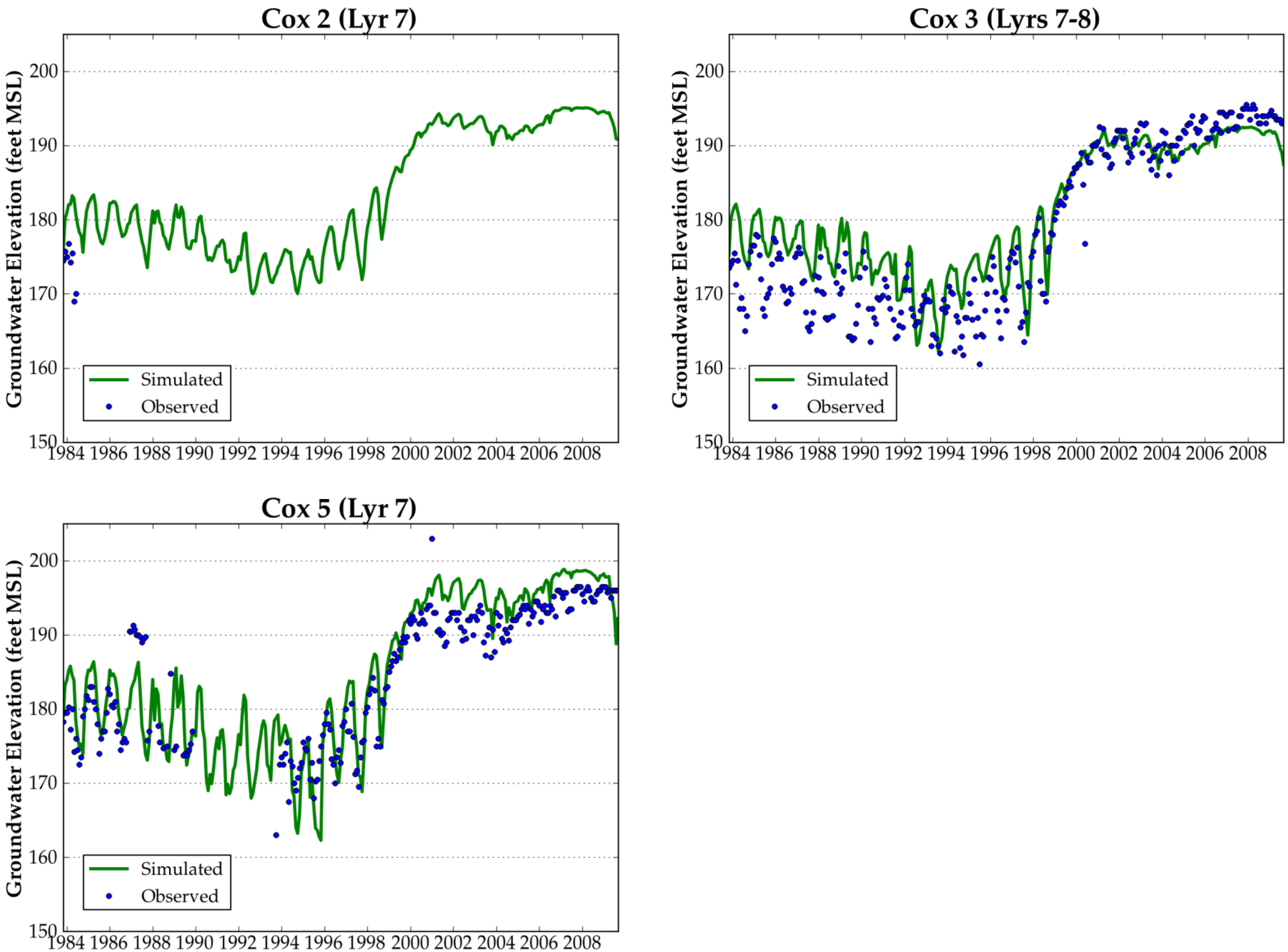

Figure 6-19. Hydrographs for Cox Well Field

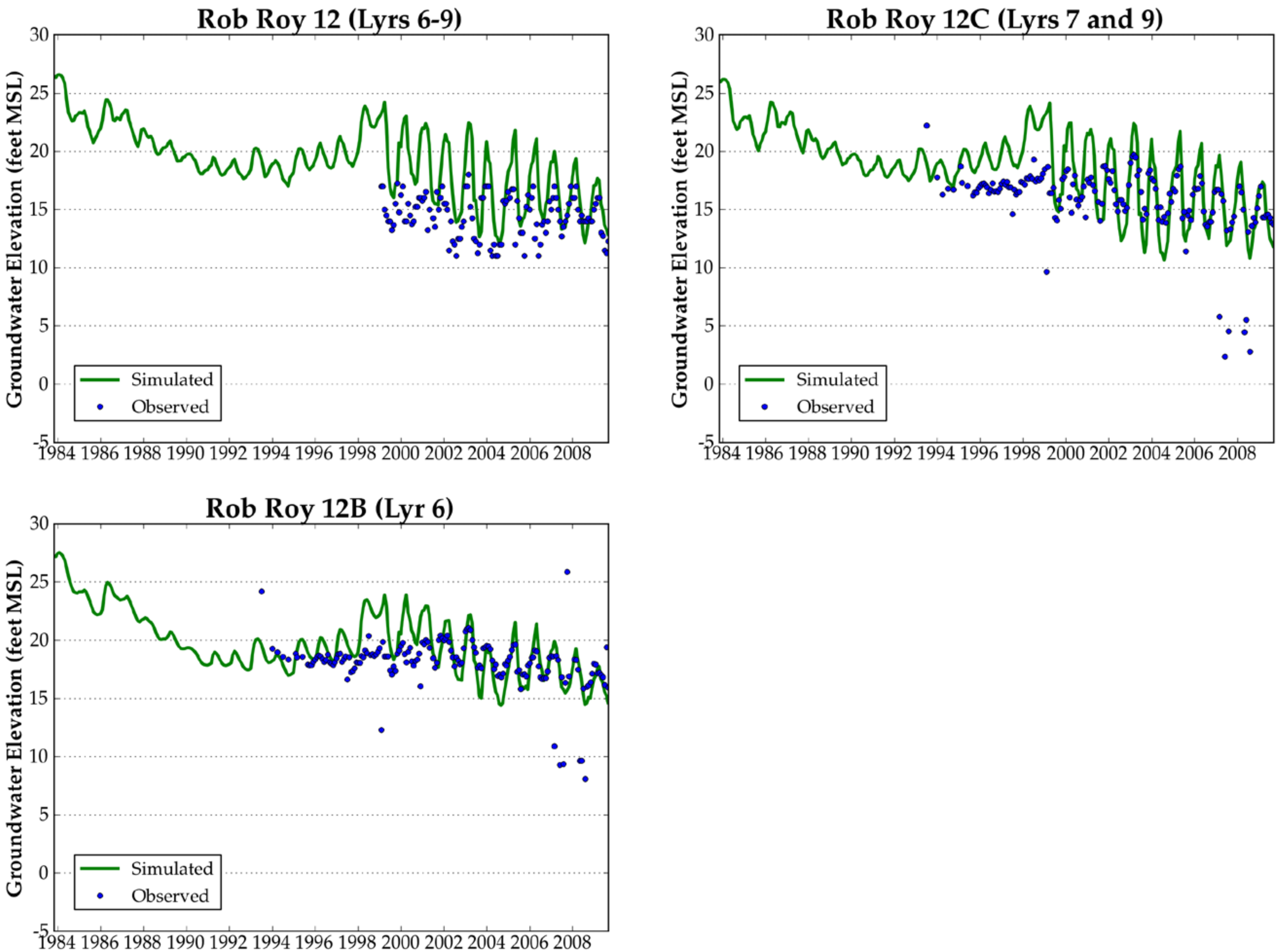

Figure 6-20. Hydrographs for Rob Roy 12 Wells Screened in Similar Layers

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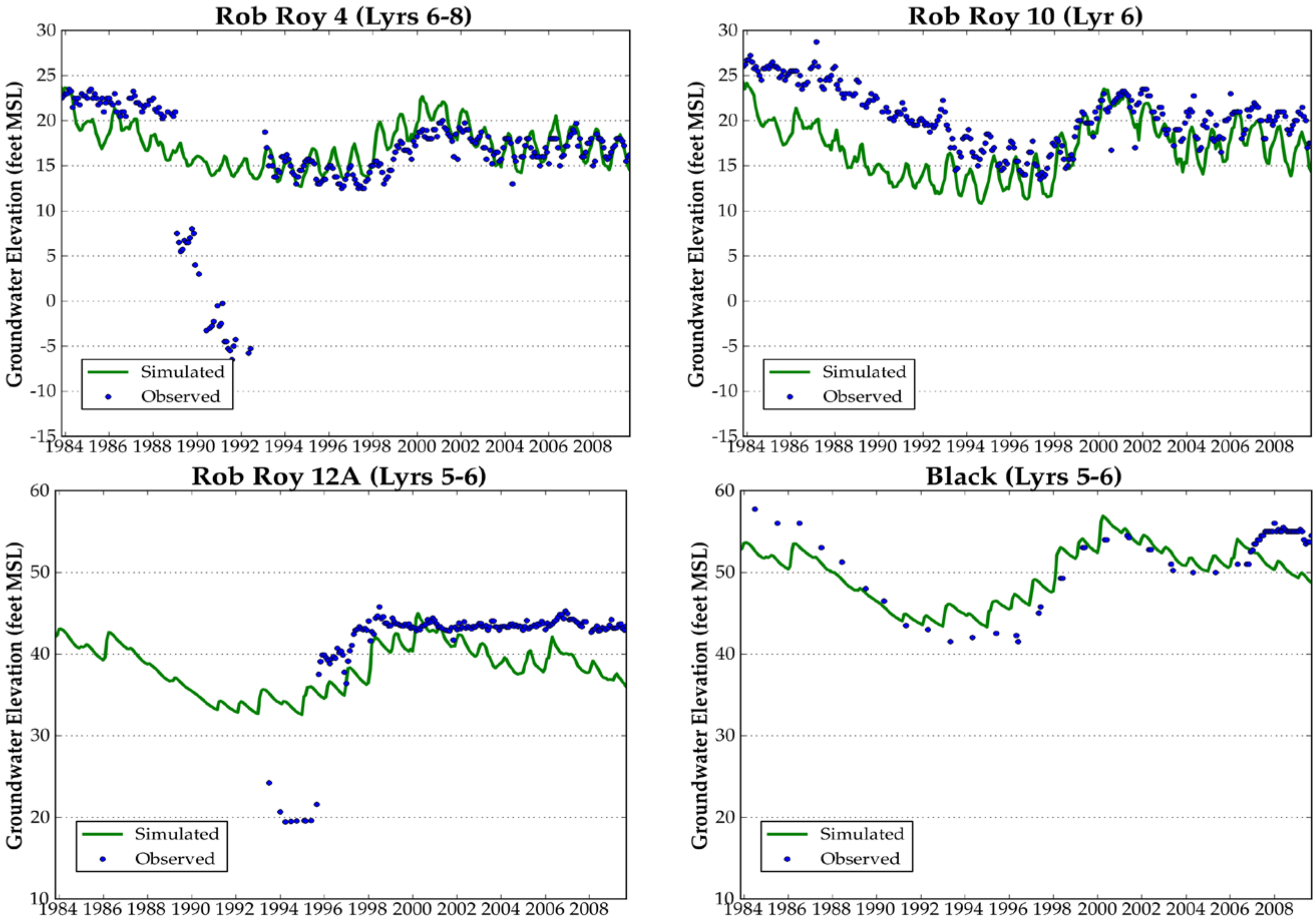

Figure 6-21. Hydrographs for Rob Roy 4, Rob Roy 10, Rob Roy 12A and Black Well

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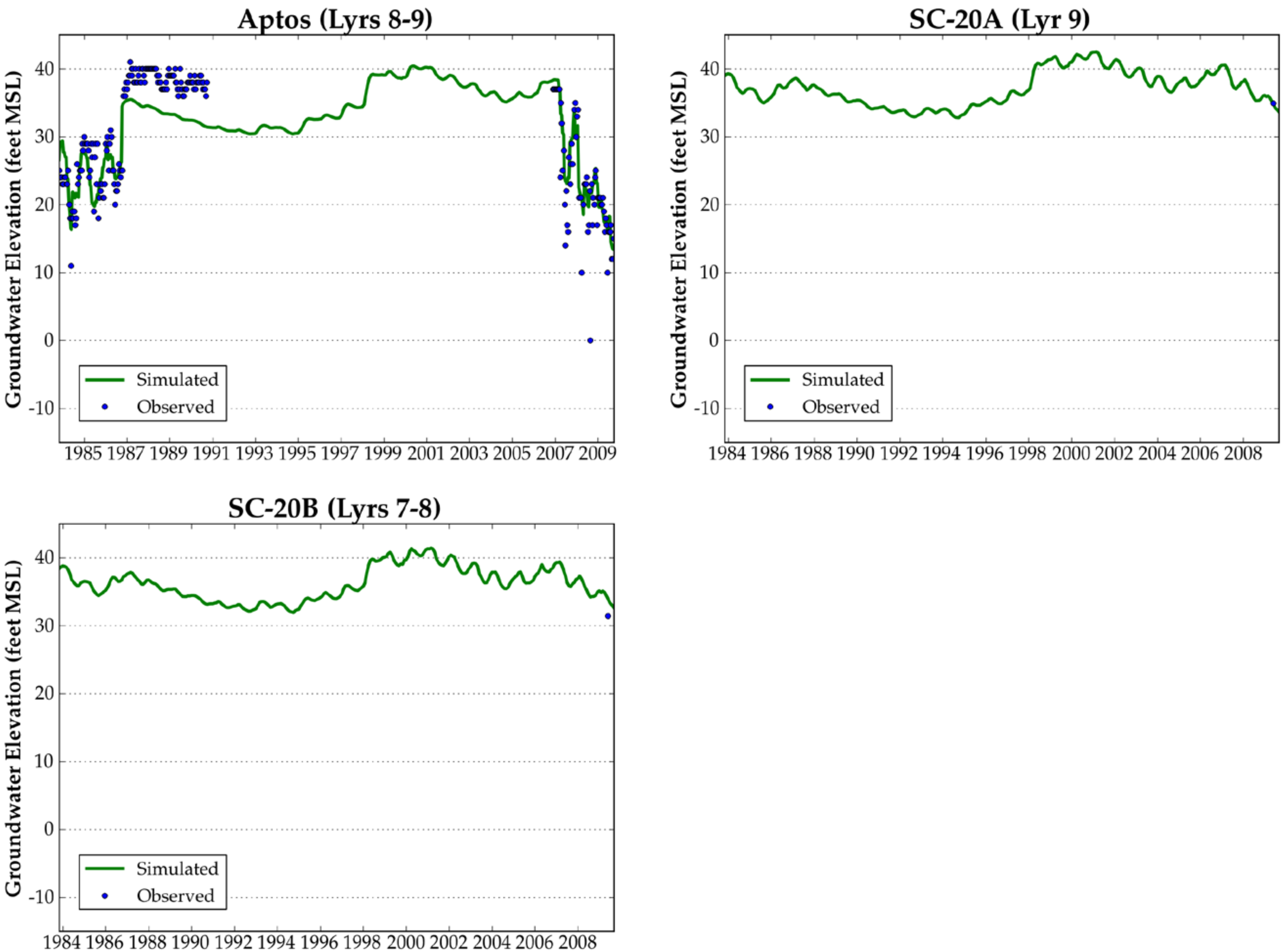

Figure 6-22. Hydrographs for Aptos Jr. High and Polo Grounds Wells

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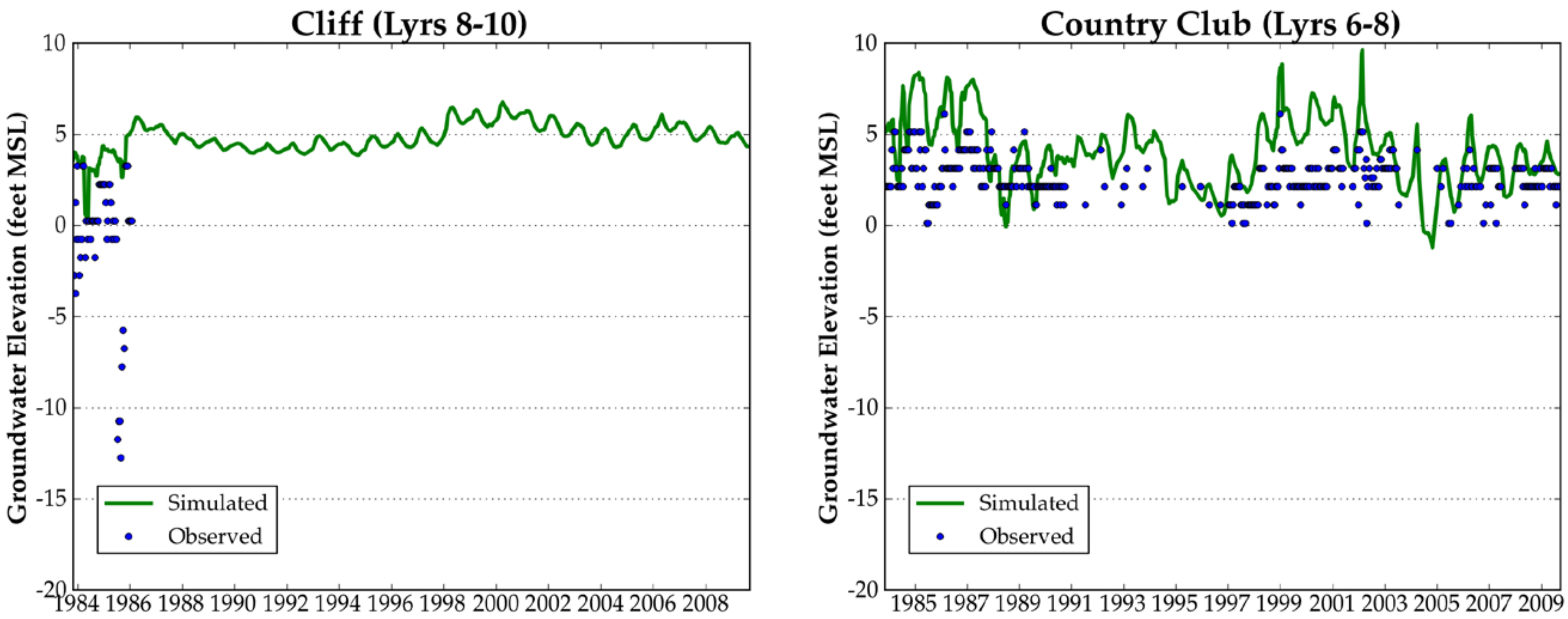

Figure 6-23. Hydrographs for Cliff and Country Club Wells

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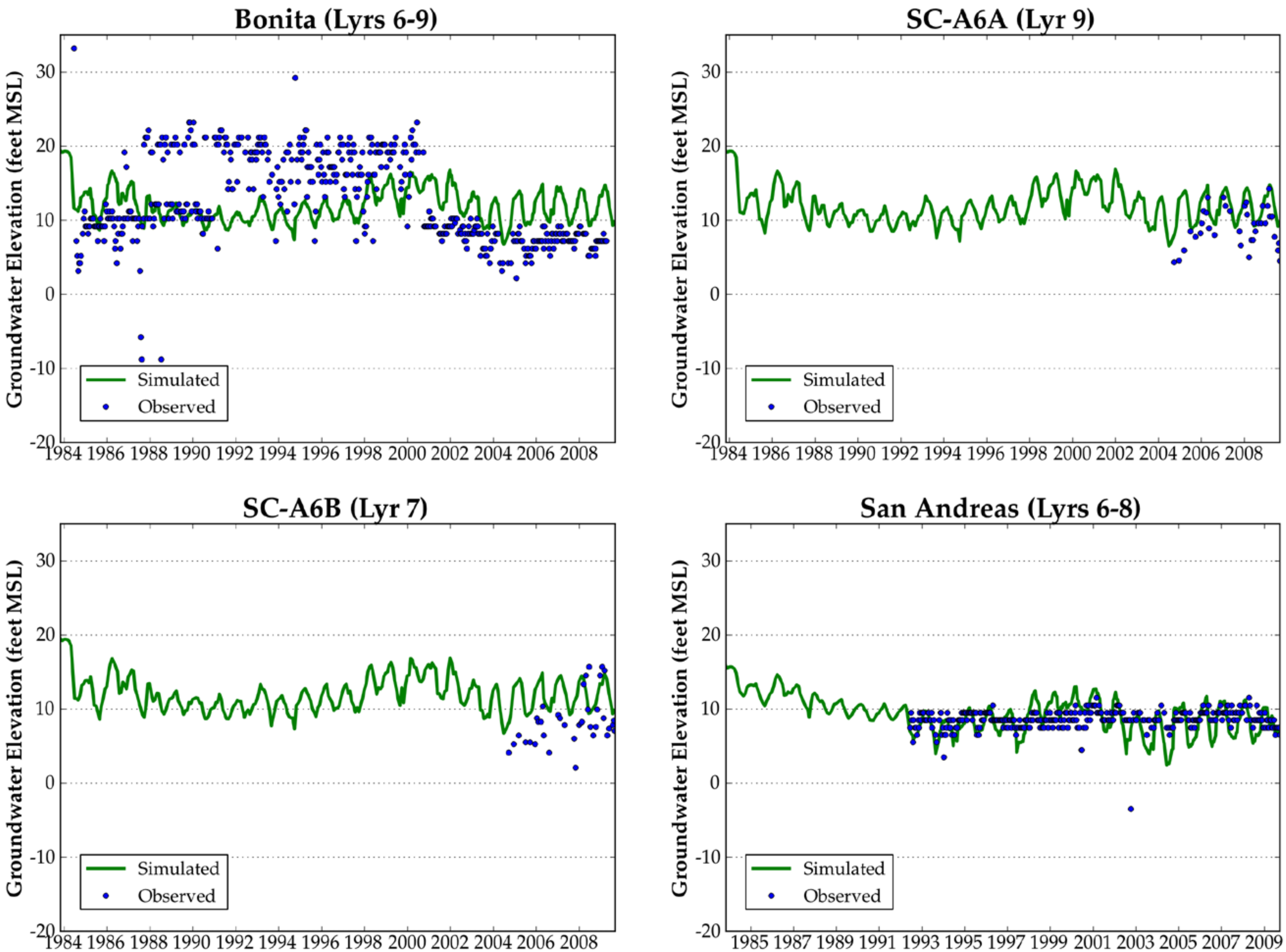

Figure 6-24. Hydrographs for Bonita and San Andreas Wells Screened in Similar Layers

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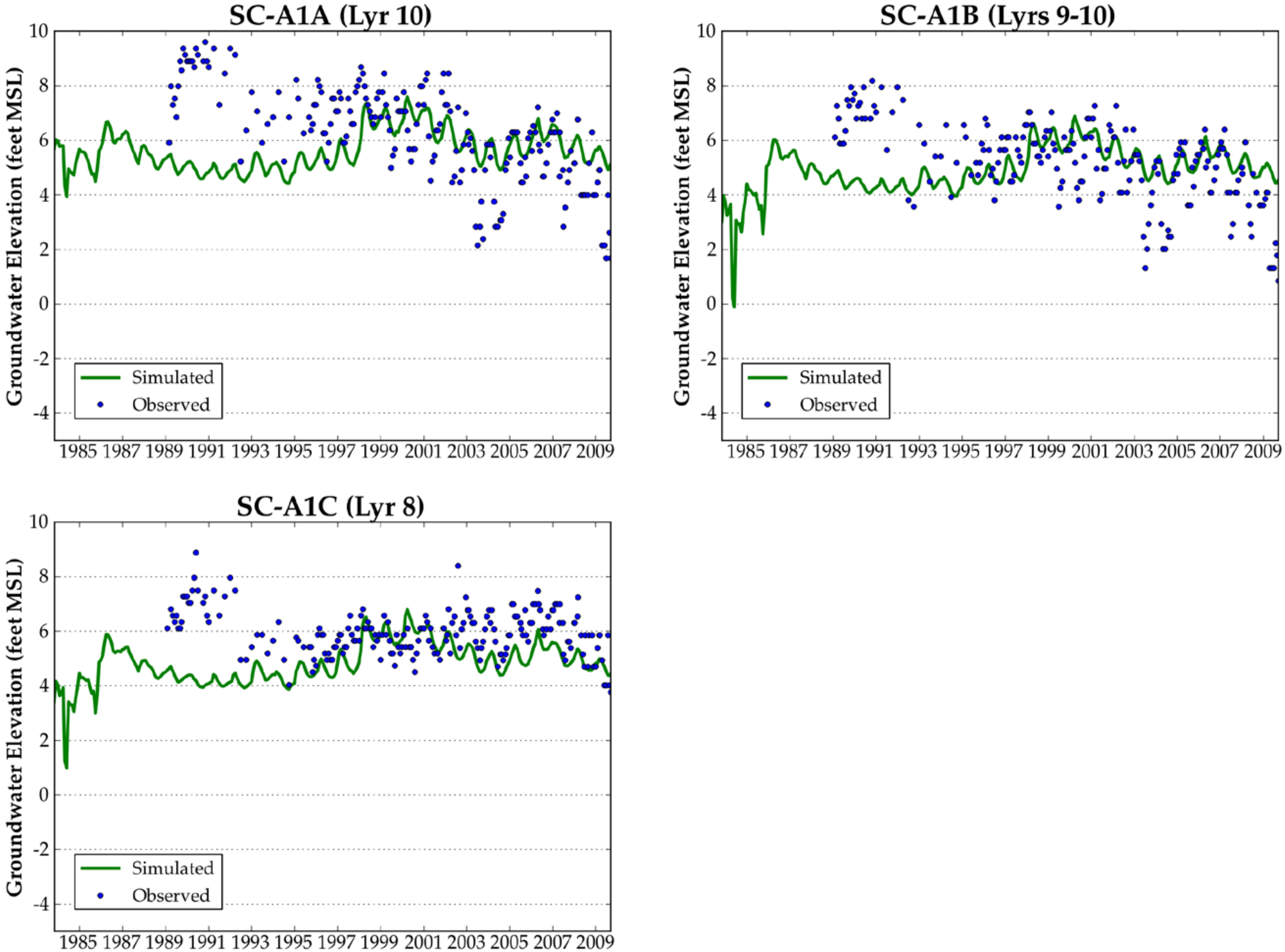

Figure 6-25. Hydrographs for SC-A1 (Cliff Drive) Wells

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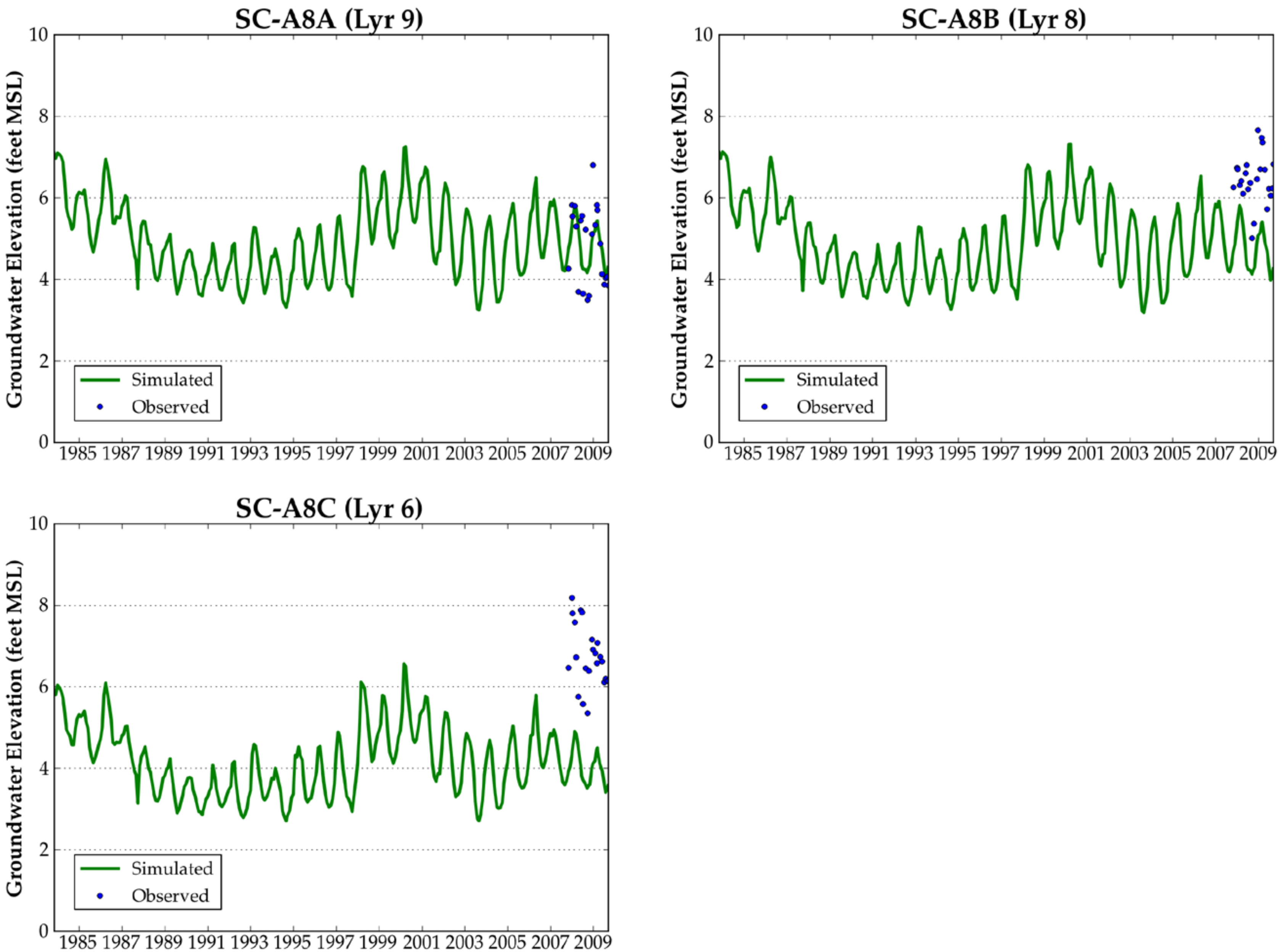

Figure 6-26. Hydrographs for SC-A8 (Dolphin and Sumner) Wells

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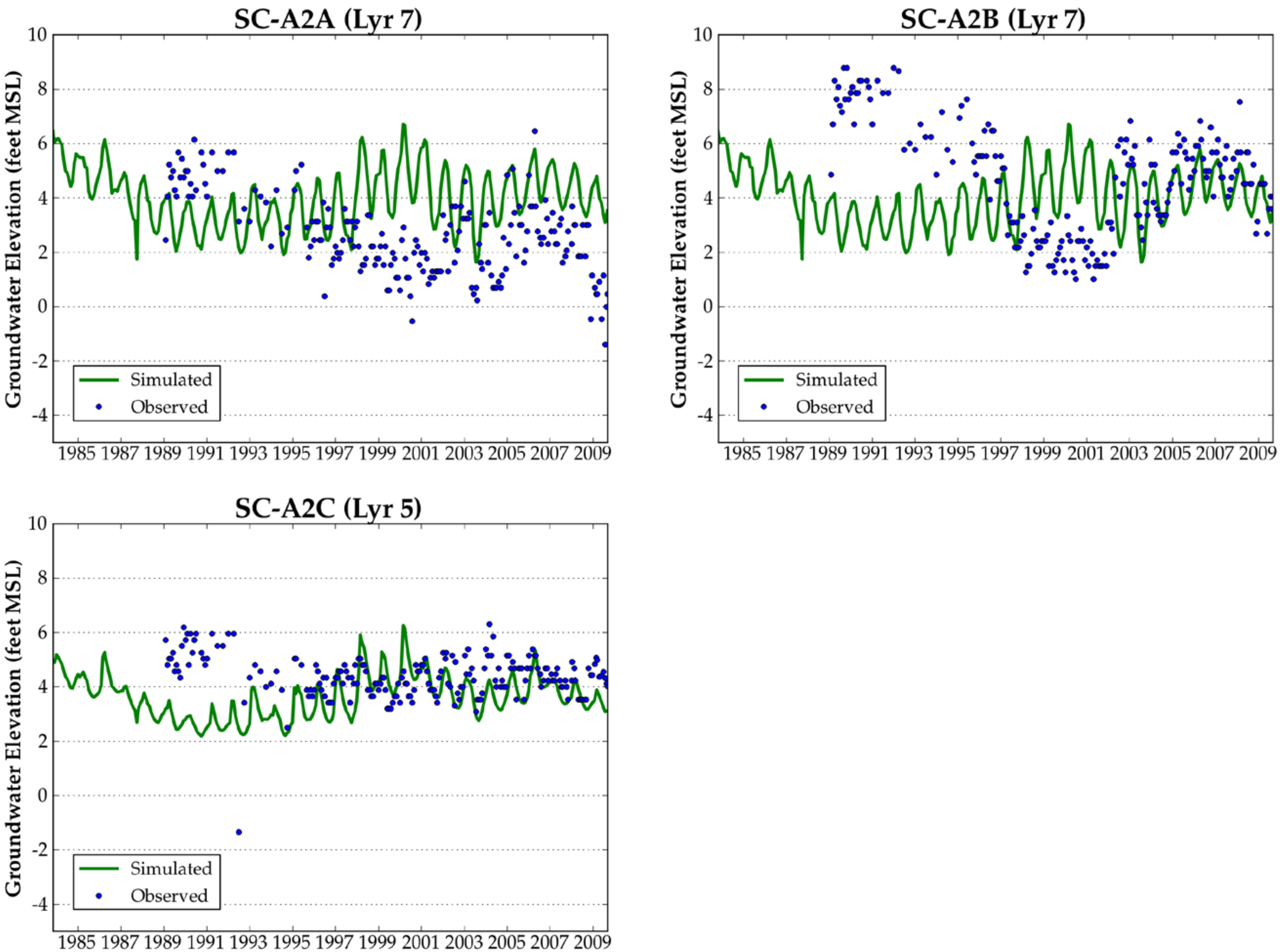

Figure 6-27. Hydrographs for SC-A2 (Dolphin and Sumner) Wells

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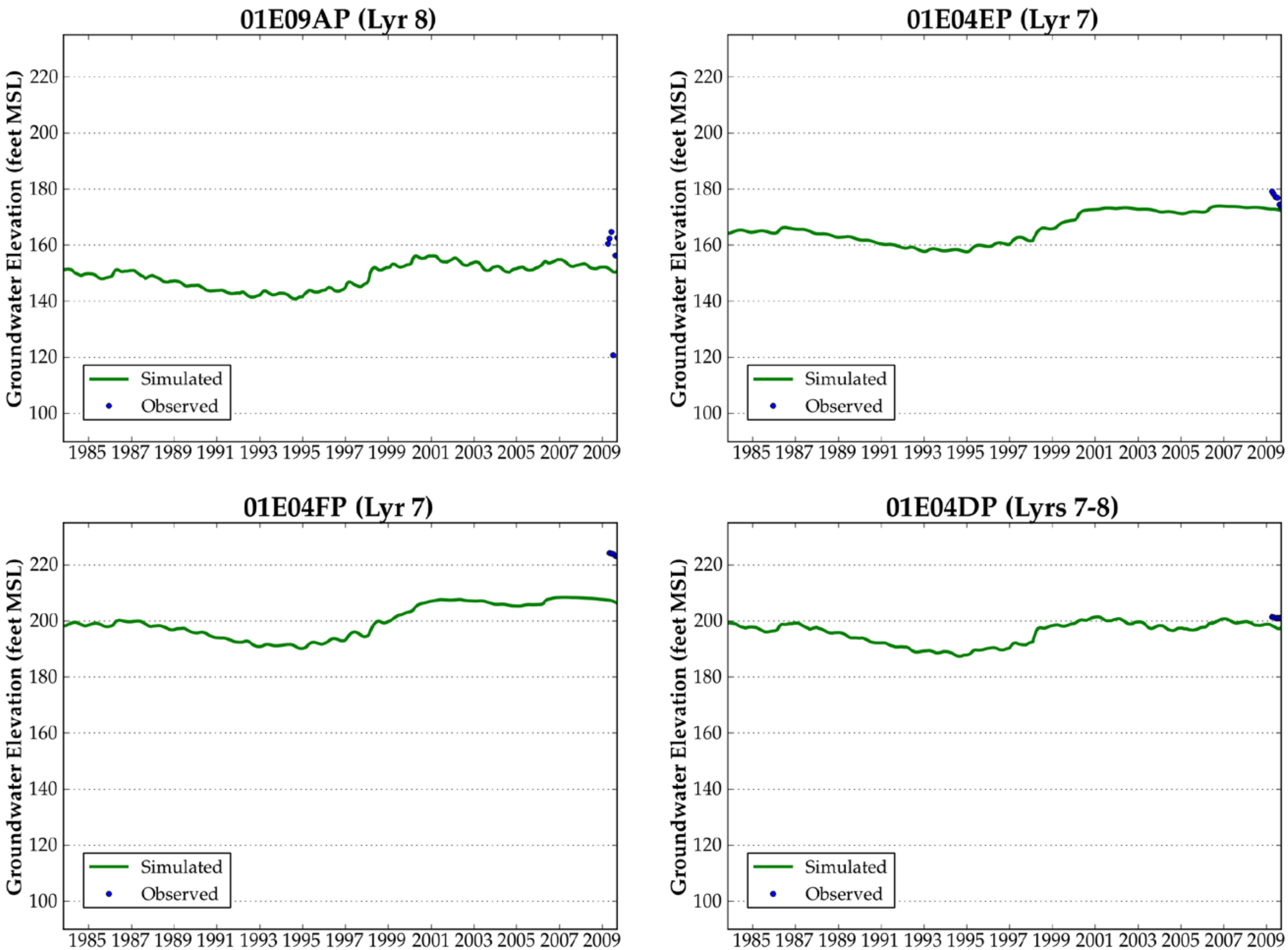

Figure 6-28. Hydrographs for Private Wells near Cox Well Field

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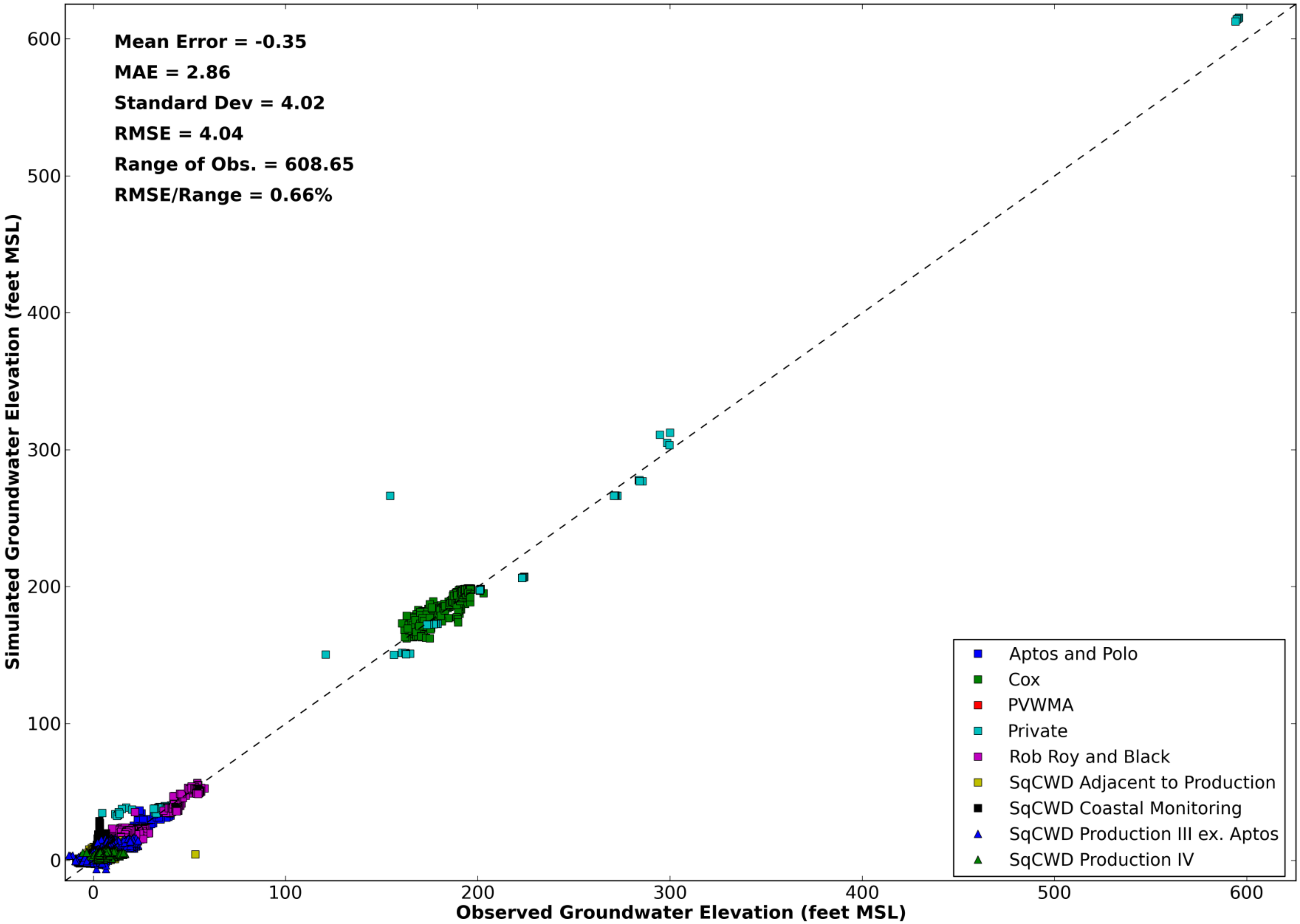

Figure 6-29. Observed vs. Simulated Groundwater Elevations

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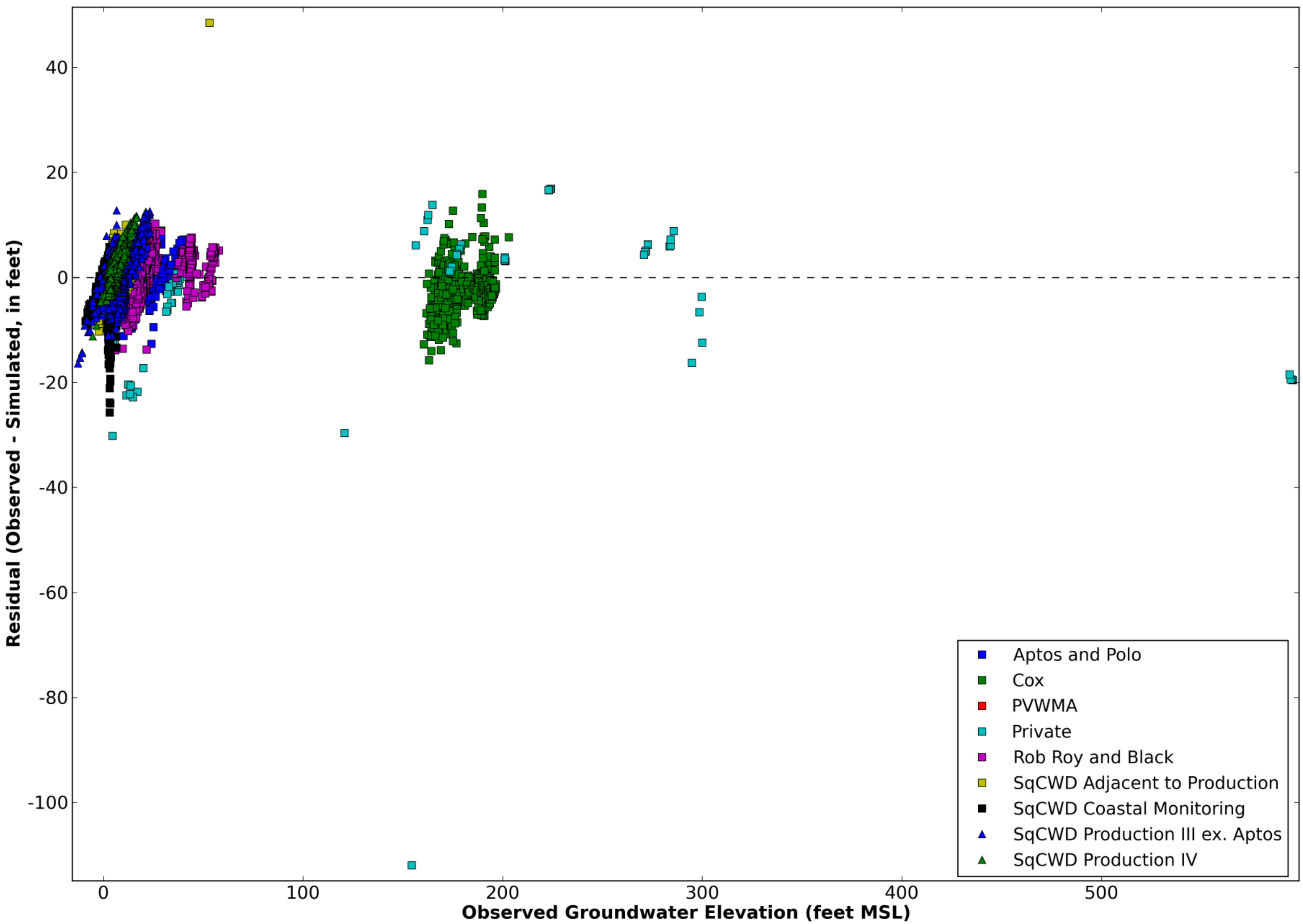

Figure 6-30. Observed vs. Model Residual Groundwater Elevations

6.6 CALIBRATED MODEL WATER BALANCE

The water balance of the calibrated model is evaluated based on subbasins as defined by watersheds used in the PRMS model (HydroMetrics WRI, 2011) that provides the recharge input for the model. Figure 6-31 shows the subbasins. The evaluation focuses on the subbasins where the model is best calibrated: Valencia Creek and Rio del Mar Area. Figure 6-32 through Figure 6-34 show the water balances in each subbasin with outflows as bars and inflows as lines.

6.6.1 VALENCIA CREEK SUBBASIN

The Valencia Creek subbasin includes CWD's Cox and Rob Roy well fields, as well as SqCWD's Aptos Jr. High and Polo Grounds wells. Figure 6-32, however, shows that municipal production is a relatively small fraction of the water balance in the Valencia Creek subbasin. Most of the outflow from the subbasin is to adjacent subbasins.

The inflows vary more over time than outflows in the subbasin. The inflow pattern is evident in the observed data and simulated results at the Black monitoring well (Figure 6-21).

Recharge from the PRMS model was applied to the groundwater model assuming that there are no flows between streams and aquifers. Figure 6-33 shows stream leakage to and from Valencia Creek. Annual leakage to the Creek is no greater than 2% of total inflows to the subbasin, so the inconsistency with the recharge assumption is minor.

6.6.2 RIO DEL MAR AREA SUBBASIN

The Rio del Mar Area subbasin includes SqCWD Country Club and Bonita wells. Figure 6-34 shows that the majority of outflow is offshore, and not from pumping. A substantial amount of outflow is necessary to protect the Aromas area from seawater intrusion (HydroMetrics WRI, 2012), but the amount of outflow in this subbasin may be more than necessary as annual average groundwater levels at the SC-A1 wells are 2-3 feet above protective elevations.

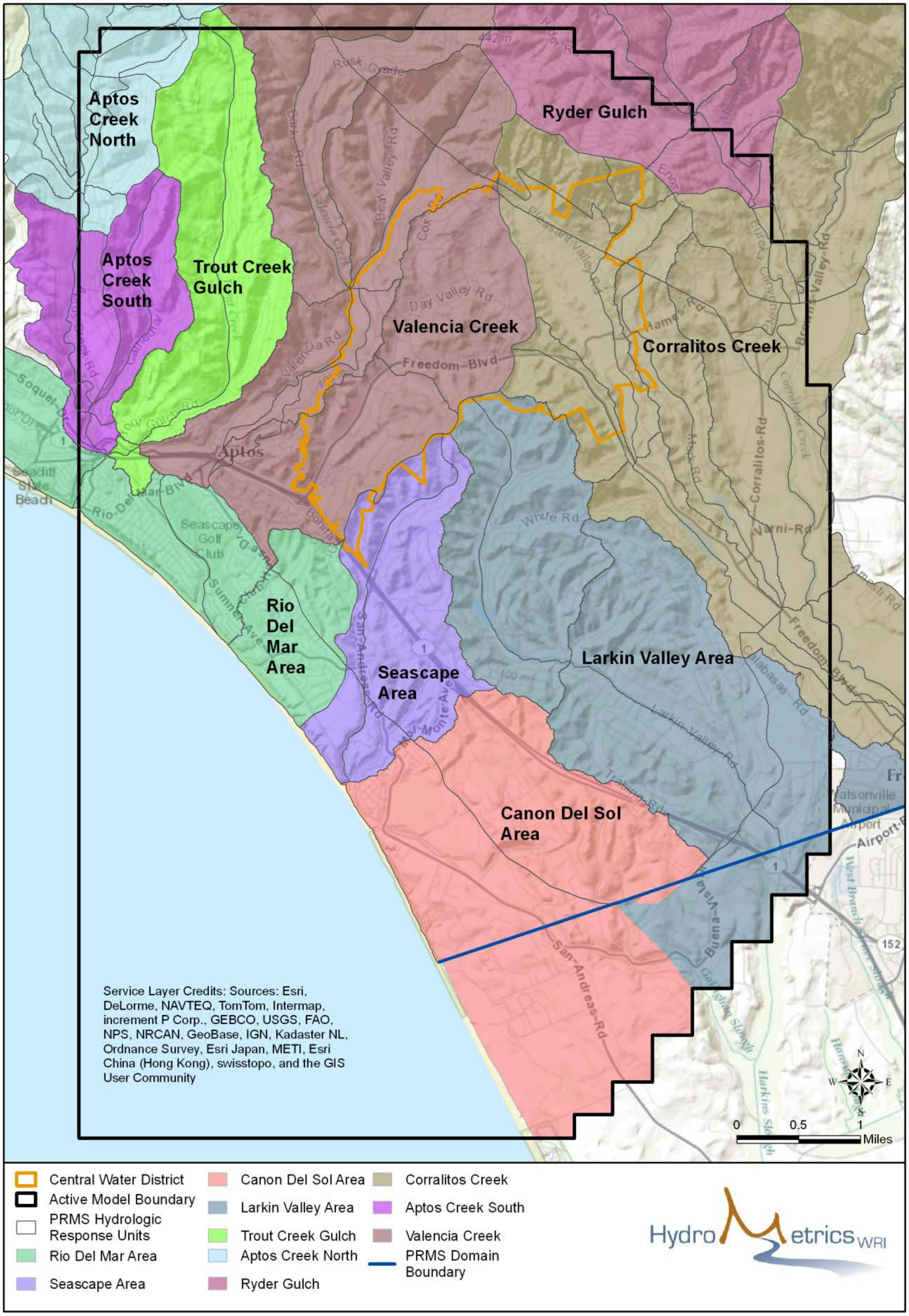

Figure 6-31. Subbasins Used for Water Balance Evaluation

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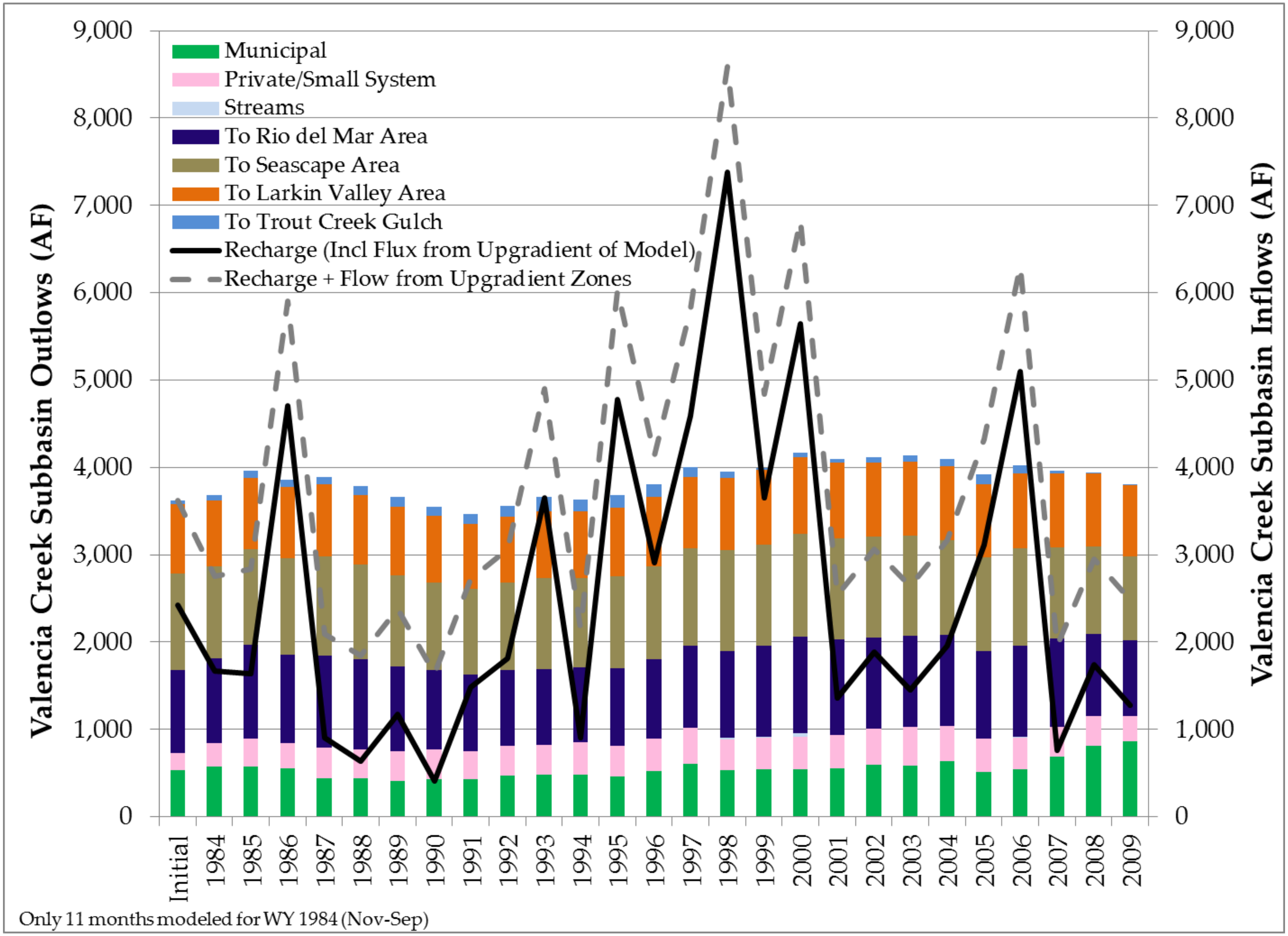

Figure 6-32. Calibrated Model Water Balance for Valencia Creek Subbasin

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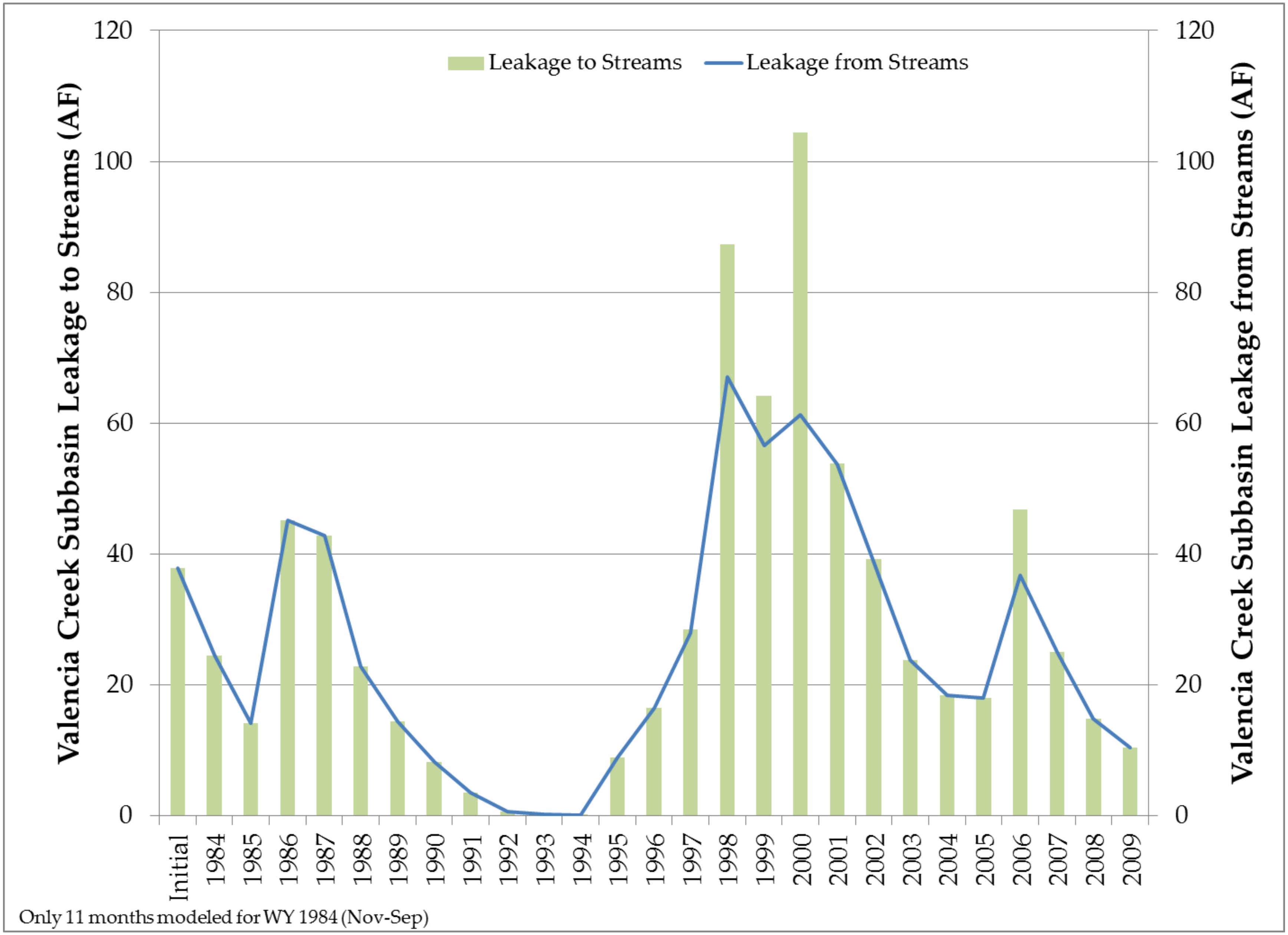

Figure 6-33. Calibrated Model Stream Leakage for Valencia Creek Subbasin

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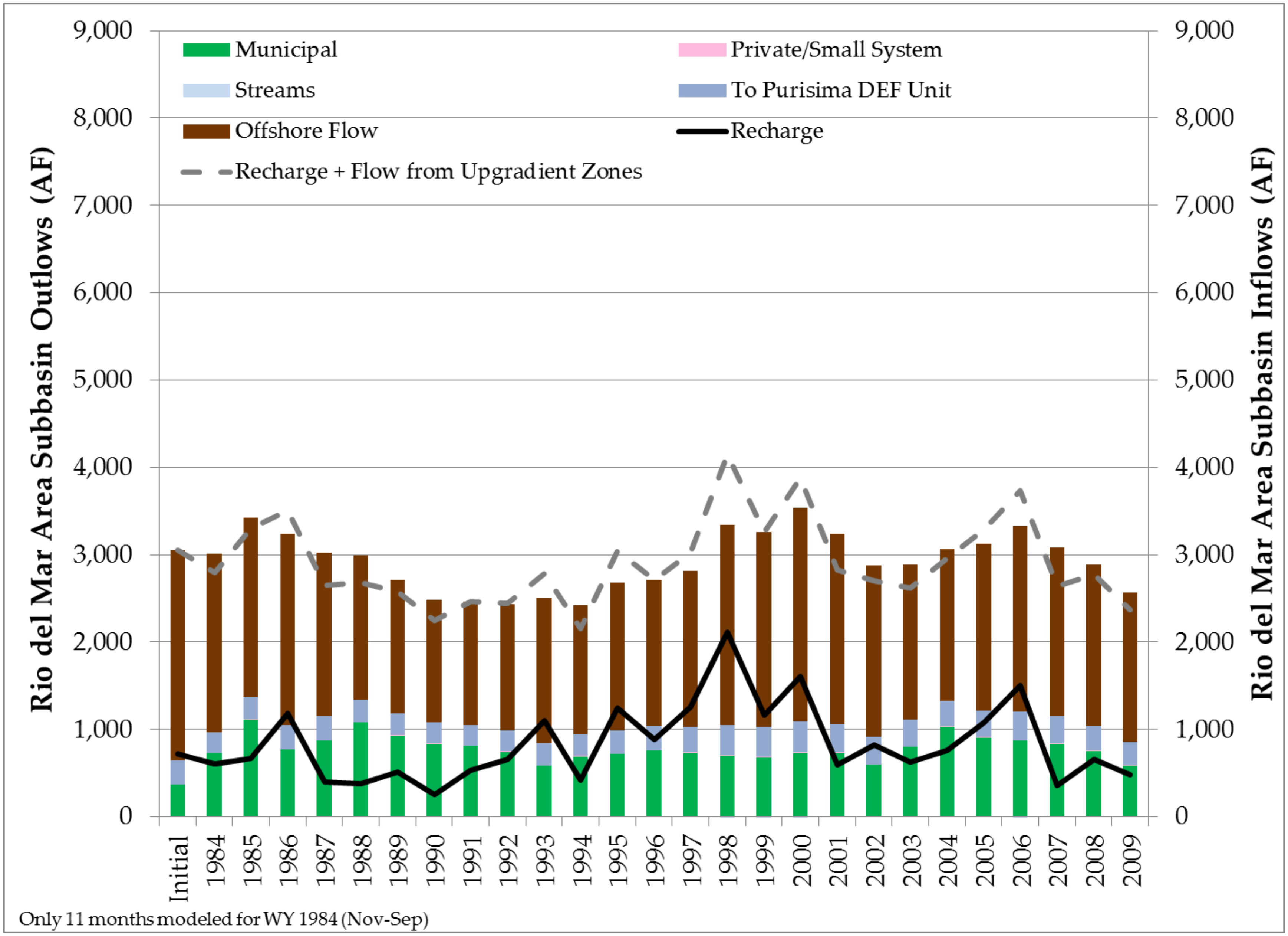

Figure 6-34. Calibrated Model Water Balance for Rio del Mar Subbasin

6.7 Implications of Calibration for Predictive Simulations

As detailed in Section 7 (Task 4.5), the model is used to evaluate potential changes to CWD's pumping distribution at the Cox and Rob Roy well fields. These include the following scenarios:

- Baseline: Project current pumping into the future
- Scenario 1: Maximize Cox pumping
- Scenario 2: Modify Rob Roy 12 to improve water quality
- Scenario 3: Maximize Rob Roy and Cox pumping

The model is well calibrated at the Cox (Figure 6-19) and Rob Roy (Figure 6-20) well fields over a time period when the distribution of pumping varied between the Cox and Rob Roy well fields. The model is also well calibrated at other wells in the Valencia Creek subbasin such as the Black monitoring well (Figure 6-21) and SqCWD's Aptos Jr. High well (Figure 6-22). Therefore, the model is the appropriate tool to evaluate the effects of pumping scenarios on wells within the Valencia Creek subbasin.

Observation data from private wells near the Cox well field only are available for the last two years of the calibration period. The calibrated model approximates recent groundwater levels at these wells (Figure 6-28), but the response at these wells to changes in pumping at the Cox well field have not been calibrated. Evaluating relative effects of pumping scenarios at this group of wells is appropriate based on calibration to data at other wells in the subbasin, but results at specific private wells are subject to a reasonable level of uncertainty.

The model adequately simulates observed groundwater level trends in the Rio del Mar area (Figure 6-25 and Figure 6-26). Therefore, evaluating effects of pumping scenarios on groundwater level trends at wells in the area, including coastal monitoring wells SC-A1 and SC-A8, is appropriate.

Streamflows for the different scenarios should be evaluated to make sure that they are consistent with the modeling assumption that there is no leakage between the aquifer and streams. Maximum annual stream leakage simulated to and from the Creek should not be much greater than 2% of total inflows to the subbasin, in line with the results of the calibrated model.

The model does not adequately simulate observed groundwater level trends in the Seascape area (Figure 6-27) and areas to the south and east. The model should not be used to evaluate groundwater management in this area without further modifications.

The calibrated model files are provided on compact disk in Appendix E.

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SECTION 7
GROUNDWATER MANAGEMENT ANALYSIS (TASK 4.5)

This section provides results of groundwater modeling scenarios using the updated (Section 5, Task 4.1) and calibrated (Section 6, Task 4.1) groundwater model. The key groundwater management strategy being evaluated is to shift pumping away from the Rob Roy well field and to the Cox well field due to the presence of chromium VI in the Rob Roy well field. CWD would like to operate the two well fields in a manner such that pumping is reduced at the Rob Roy well field, water quality is maximized, and treatment costs are minimized. Implementing this strategy may have additional benefits to the overall basin management by allowing for recovery of groundwater levels in the Aromas Formation, which would reduce the potential for seawater intrusion, a basin management objective in the GMP.

The section was distributed to the Technical Advisory Committee (TAC) for review as a draft technical memorandum on January 23, 2014 and is being issued as a final technical memorandum as part of the grant scope (HydroMetrics WRI, 2014).

7.1 ANALYSIS APPROACH

The objective of the groundwater management modeling scenarios is to evaluate the feasibility of this pumping redistribution strategy and broadly assess the potential environmental impacts of the strategy. Therefore, the modeling scenarios evaluate the changes in groundwater levels in the basin for three different scenarios that represent the potential range in pumping for the Rob Roy and Cox well fields. For these scenarios, other model conditions are held the same so that differences between the scenarios represent only the effects of shifting pumping. Even though some of the assumptions for other model conditions do not accurately simulate future conditions, keeping them consistent for all scenarios allows for a comparison of the different groundwater management alternatives. The common assumptions for all of the model scenarios are discussed below.

7.1.1 INITIAL CONDITIONS AND SIMULATION TIME PERIOD

The initial groundwater elevations are based on the final groundwater elevations simulated by the calibration model (Section 6) which runs to September 30, 2009. It will be several years before groundwater management alternatives can be implemented, but it is assumed that groundwater conditions at that time are similar to conditions at the end of 2009. It is estimated that management alternatives can be implemented by the start of Water Year 2016 so the simulations represent Water Years 2016-2041.

7.1.2 Hydrology and Boundary Conditions

The long-term hydrology is a repeat of the hydrology from the regional PRMS areal recharge model and other boundary conditions used for groundwater model calibration in Task 1.1. The hydrologic period from 1984 to 2009 includes the full range of hydrologic conditions, with periods of extended drought and above average rainfall. This allows for the scenarios to include an assessment over this range of conditions.

One adjustment to the hydrology from the groundwater model calibration is made for the upgradient flux west of the Zayante fault. A seven-year running average of upgradient recharge from the PRMS recharge model (HydroMetrics WRI, 2011) is still used, but it is assumed that hydrology for the six years prior to the start of the scenario period can be represented by PRMS results for Water Years 2004-2009.

Boundary conditions representing groundwater inflow from and outflow to areas outside of the model domain repeat the conditions used in the calibration model for 1984 to 2009 to be consistent with the hydrologic conditions. Repeating the condition for the Purisima DEF boundary to the west assumes that SqCWD groundwater pumping at the Aptos Creek and T. Hopkins wells are similar to what occurred from 1984 to 2009. Repeating the condition for the Pajaro Valley condition to the east assumes that Pajaro Valley conditions are similar to what occurred from 1984 to 2009.

7.1.3 Non-CWD Pumping

The existing private well and small water system pumping repeat what was used in the calibration model. Private well pumping is assumed to be relatively stable over time, and so it will be held constant throughout the simulations.

SqCWD pumping is based on the pumping distribution plans that have been developed to meet pumping goals and demand outlined in the 2012 Integrated Resources Plan (HydroMetrics WRI, 2013b). These plans include pumping for each well in SqCWD Service Areas 3 and 4, including the recently added Polo Grounds well. The plans include five years of pumping to meet projected 2015 demand, then twenty years of limited pumping to recover the basin beginning in 2020, and one year (2041) of limited pumping at the Aromas area post-recovery pumping goal.

HydroMetrics WRI (2013b) includes several pumping distribution plans. The simulations are based on plans that do not include pumping at the Altivo well, which has the highest chromium VI concentrations. It is assumed that SqCWD will shift pumping from the Altivo well to the San Andreas and Bonita wells where SqCWD is planning on installing chromium VI treatment. In addition to the Altivo well, there is no pumping assumed at the Seascape and Sells wells. There have been high salt concentrations directly below the Seascape well and nitrate concentrations exceeding the drinking water standard at the Sells well.

Annual demand for simulation water year 2020 is lower than the projected 2015 demand used for simulation water years 2016-2019 because simulation water year 2020 is based on 1988 hydrology, which would trigger drought curtailment (HydroMetrics WRI, 2011). After simulation water year 2020, it is assumed that supplemental supply and drought curtailment will allow SqCWD to meet its reduced pumping goals in all years.

Seasonal pumping for simulation water years 2016-2019 are based on average seasonal distribution for water years 2005-2012. Seasonal pumping for simulation water years 2020 and 2041 are based on the average seasonal distribution for water years 2005-2012 with 15% drought curtailment applied from May to October. Simulation water years 2020 and 2014 are based on 1988 and 2009 hydrology respectively, years which would trigger drought curtailment (HydroMetrics WRI, 2011). HydroMetrics WRI (2013b) specifies the seasonal distribution for the recovery period (simulation water years 2021-2040), which does not change whether supplemental supply or drought curtailment is used to reduce pumping from May to October.

Table 7-1 shows the assumed monthly pumping at the SqCWD wells. [1]

[1] All pumping amounts shown in Table 7-1 are rounded to the nearest acre-foot so totals calculated from the monthly values may vary from the annual totals due to rounding.

Table 7-1. Monthly Pumping in acre-feet for SqCWD Wells

Simulation Water Years (Pumping Goal Set, HydroMetrics WRI 2013c)	Basis for Seasonal Distribution	Well	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total
2016-2019 (Set 1B: Meet 2015 Demand 1,330 acre-feet per year)	WY 2005-2012	Aptos Jr. High	15	12	11	11	10	11	12	16	18	20	18	17	**170**
		Polo Grounds	28	22	20	20	18	21	23	30	33	37	35	31	**320**
		Country Club	20	16	15	15	13	15	17	22	24	26	25	22	**230**
		Bonita	27	21	19	19	18	20	22	29	32	35	33	30	**305**
		San Andreas	27	21	19	19	18	20	22	29	32	35	33	30	**305**
2020 (Set 1D: Drought Curtailment on 2015 Demand 1,220 acre-feet per year)	WY 2005-2012 with drought curtailment May-October	Aptos Jr. High	15	12	11	11	10	11	12	16	18	20	18	17	**170**
		Polo Grounds	28	22	20	20	18	21	23	30	33	37	35	31	**320**
		Country Club	20	16	15	15	13	15	17	22	24	26	25	22	**230**
		Bonita	27	21	19	19	18	20	22	18	21	24	22	19	**250**
		San Andreas	27	21	19	19	18	20	22	18	21	24	22	19	**250**
2021-2040 (Set 2B : Recovery 600 acre-feet per year)	HydroMetrics WRI, 2013b	Aptos Jr. High	25	11	0	0	0	0	12	25	24	25	25	24	**170**
		Polo Grounds	31	18	13	13	11	13	19	31	30	31	31	30	**270**
		Country Club	7	3	0	0	0	0	4	7	7	7	7	7	**50**
		Bonita	8	3	0	0	0	0	4	8	8	8	8	8	**55**
		San Andreas	8	3	0	0	0	0	4	8	8	8	8	8	**55**
2041 (Set 3B : Post – Recovery 1,200 acre-feet per year)	WY 2005-2012 with drought curtailment May-October	Aptos Jr. High	14	13	12	12	11	12	14	15	17	18	17	16	**170**
		Polo Grounds	27	25	22	22	20	23	26	28	31	34	32	29	**320**
		Country Club	17	16	15	15	13	15	17	18	20	23	21	19	**210**
		Bonita	21	19	17	17	16	18	20	22	24	27	25	23	**250**
		San Andreas	21	19	17	17	16	18	20	22	24	27	25	23	**250**

7.2 GROUNDWATER MANAGEMENT SCENARIOS

The groundwater management model scenarios include a Baseline Simulation and three predictive scenarios.

Model scenario results are best evaluated by comparing the relative change between scenarios rather than the absolute results of a single scenario. The Baseline Simulation provides a projection based on current pumping conditions. Comparing the results of the Baseline Simulation to the results of each of the three predictive scenarios provide the primary basis for evaluating the effects of shifting pumping between the Rob Roy and Cox well fields.

The three predictive scenarios are used to evaluate the range of potential pumping for each of the well fields. These will include maximizing pumping at the Cox well field, modifying the Rob Roy #12 well to improve water quality, and maximizing pumping at both the Cox and Rob Roy well fields. Table 7-2 shows the simulated monthly pumping at the well fields for the baseline and three predictive scenarios.[2] The simulation model files are provided on compact disk in Appendix E.

7.2.1 BASELINE SIMULATION

The purpose of the Baseline Simulation is to project current conditions into the future. In this case, annual pumping at each CWD well is constant and is assumed to equal the average pumping at that well from 2005 to 2011. At the Rob Roy well field, annual pumping is 529 acre-feet per year (AFY), while at the Cox well field it is 23 AFY, for a combined annual pumping by CWD of 552 AFY.

7.2.2 SCENARIO 1: SHIFT PUMPING TO NEW COX WELL

For Scenario 1, it is assumed that the majority of the CWD pumping is shifted from the Rob Roy well field to the Cox well field. The draft Tasks 2 and 3.1 memo recommending a replacement well at Cox estimates that the new well can pump 324 gpm 15 hours per day. This is equivalent to an annual capacity of 327 acre-feet. However, demand is below the capacity of this well in January and February, so the new Cox well is pumped 324 AFY in this scenario. If monthly demand exceeds the capacity of the new Cox well, the Rob Roy #12 well provides the remaining amount (229 AFY) for a total of 552 AFY. The new Cox well would supply approximately 45% of the

[2] All pumping amounts shown in Table 7-2 and the text below are rounded to the nearest acre-foot so totals calculated from the monthly values shown may vary from the annual totals due to rounding.

total CWD demand from May-October and approximately 90% of the total CWD demand from November-April.

7.2.3 Scenario 2: Improve Rob Roy #12 Water Quality

For Scenario 2, it is assumed that the upper two screened intervals in the Rob Roy #12 well would be blocked off by a liner. This would leave pumping available from only the lower screen. The water from these upper screened intervals has the highest chromium VI concentrations, so it is assumed that blocking this flow would improve water quality from this well. According to data collected for Section 4, about 40% of the total flow (236 out of 590 gpm) in the Rob Roy #12 well comes from the lower screen. For Scenario 2, monthly pumping is limited at Rob Roy #12 to 236 gpm 15 hours per day.

The replacement well at Cox would pump at 324 gpm for 15 hours per day to meet monthly demand as in Scenario 1, resulting in a total of 324 AFY. Rob Roy #12 would meet additional monthly demand up to its pumping limit, resulting in a total of 138 AFY. Any remaining monthly demand would be met by Rob Roy #4 and #10. 90 AFY would be required from these two wells. The maximum monthly production required of Rob Roy #4 and #10 is similar to the 2005-2011 monthly averages pumped at the wells for July-September.

Since this scenario was developed and implemented, DPH proposed a drinking water standard of 10 µg/L for chromium VI. Chromium VI concentrations at the Rob Roy #12 well have been around 4 µg/L. Therefore, it is unnecessary to lower chromium VI concentrations at Rob Roy #12 to meet the proposed drinking water standard. In addition, chromium VI concentrations at Rob Roy #4 and #10 were recently measured as 11 µg/L so this scenario may not have lower average system chromium VI concentrations than Scenario 1.

Even though there does not appear to be a reason to implement this scenario, results of the scenario provide information about the effects of changing vertical distribution of flow at the Rob Roy #12 well that may have future application.

7.2.4 Scenario 3: Maximize Rob Roy and Cox Pumping

For Scenario 3, the pumping is increased to represent the maximum potential pumping for the two CWD well fields. The pumping for the Cox well field would be pumped at the full recommended replacement well capacity of 327 AFY. For the Rob Roy well field, it is assumed that only the Rob Roy #12 well is used but at the 590 gpm rate observed during the well profiling test. Although pumping durations over the long term have not been evaluated at Rob Roy #12 against operational constraints such as the pump intake depth, it is assumed the well can pump 590 gpm for 15 hours per day. Rob Roy #12 would be pumped at 595 AFY in this scenario. The total pumping from the two wells is 921 AFY.

The maximum pumping scenario is for informational purposes only, since this is more water than is needed by CWD, and it is unclear that the Rob Roy #12 pumping rate meets operational constraints over the full year. It is possible that extra water could be provided utilizing inter-ties to water users outside CWD; however, the provision of how and where the extra water is used is not addressed. While it is possible that recipients of the water may change pumping at non-CWD wells in the model or wells outside the model that affect model boundary conditions, Scenario 3 will not include any changes at non-District wells or boundary conditions. The background pumping conditions of all of the model scenarios remain the same to provide a consistent basis for comparison.

7.2.5 ESTIMATED SYSTEM WATER QUALITY FOR SCENARIOS

Water quality is not directly modeled by the groundwater model. Average system chromium VI concentrations for each scenario can be estimated based on concentrations from samples taken on October 8, 2013 and Task 1.2 findings (Table 7-3).

Scenario 1 improves average system concentration of chromium VI the most. Even though the modifications at Rob Roy #12 lower concentrations at that well, the need to use Rob Roy #4 and Rob Roy #10 results in a higher system concentration for chromium VI under Scenario 2 than Scenario 1. Scenario 3 has a majority of flow from Rob Roy #12 but is able to reduce average system concentration by replacing Rob Roy 4 & 10 pumping with Cox pumping. Different pressure zones in the system will result in non-uniform concentrations throughout the system so pumping Rob Roy 4 & 10 using their current configurations should be minimized to ensure concentrations remain under 10 μg/L throughout the system.

Pumping the Cox well field at the higher amounts in the scenarios will result in higher iron and manganese concentrations requiring treatment. Design for a Cox treatment plant is provided in Section 9.2 (Kennedy/Jenks, 2012).

Table 7-2. Monthly Pumping in acre-feet for Groundwater Management Modeling Scenarios

	Well	Basis	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Total
CWD Demand		WY 2005-2011 Average	50	34	28	26	24	29	37	52	63	74	72	64	552
Baseline	Cox #3 & #5	WY 2005-2011 Average	1	1	1	1	1	1	1	2	4	4	3	2	23
	Rob Roy #12	WY 2005-2011 Average	34	29	23	22	19	23	26	37	43	45	42	38	379
	Rob Roy #4 & #10	WY 2005-2011 Average	15	4	4	4	3	5	10	13	16	25	26	24	150
Scenario 1	New Cox	Dry Season Capacity	28	27	28	26	24	28	27	28	27	28	28	27	324
	Rob Roy #12	Meet Remaining Demand	22	7	0	0	0	1	10	24	36	46	44	38	229
	Rob Roy #4 & #10	Backup or Destroyed	0	0	0	0	0	0	0	0	0	0	0	0	0
Scenario 2	New Cox	Dry Season Capacity	28	27	28	26	24	28	27	28	27	28	28	27	324
	Rob Roy #12	Flow Only from Lower Screen	20	7	0	0	0	1	10	20	20	20	20	20	138
	Rob Roy #4 & #10	Meet Remaining Demand	2	0	0	0	0	0	0	4	16	26	24	18	90
Scenario 3	New Cox	Dry Season Capacity	28	27	28	28	25	28	27	28	27	28	28	27	327
	Rob Roy #12	Pumping Rate During Profiling	51	49	51	51	46	51	49	51	49	51	51	49	595
	Rob Roy #4 & #10	Backup or Destroyed	0	0	0	0	0	0	0	0	0	0	0	0	0

Table 7-3. Estimated Average System Hexavalent Chromium Concentrations by Scenario

		Baseline: 2005-2011 Average	Scenario 1: Shift from Rob Roy to Cox	Scenario 2: Modify Rob Roy #12	Scenario 3: Maximize Rob Roy and Cox
	Chromium VI Concentration (µg/L)	**Annual Pumping Distribution Percentage**			
Cox	ND < 1 (estimate 0.5)	4%	59%	59%	35%
Rob Roy #12	4.3	69%	41%	N/A	65%
Modified Rob Roy #12	ND < 1 (estimate 0.5)	N/A	N/A	25%	N/A
Rob Roy #4 & #10[3]	11	27%	0%	16%	0%
		Average System Chromium VI Concentration (µg/L)			
		6.0	2.1	2.2	3.0

[3] If water pumped from these wells exceed the final MCL, CWD will make the necessary modifications to meet the MCL by blending with water from Rob Roy #12.

7.3 ANALYSIS OF RESULTS

The simulated groundwater levels and water balance are evaluated to assess the groundwater management alternatives. The primary model result evaluated is the effect of the changes in pumping on the stability of long-term groundwater level trends in the aquifer. However, pumping impacts on groundwater levels at SqCWD and private wells, coastal groundwater levels, and leakage from streams are also considered. Coastal groundwater levels are used as the primary indicator of the risk of seawater intrusion.

Hydrographs of simulation results (Figure 7-1 through Figure 7-9) show Scenario 1 (blue line) and Scenario 3 (green dashes) against the Baseline Simulation (black dots), because Scenario 2 will not improve overall system water quality. In order to show the effect of changing the vertical distribution of pumping at Rob Roy #12, hydrographs for the Rob Roy area showing Scenario 1 (blue line) against Scenario 2 (green dashes) are presented (Figure 7-11 and Figure 7-12).

Table 7-4 shows the average difference in groundwater elevations at key wells for the three management alternative Scenarios versus the Baseline Simulation. Groundwater levels are generally lower at and near the Cox well field for all Scenarios versus the Baseline Simulation. Groundwater levels are generally higher from the Rob Roy well field to the coast for Scenarios 1 and 2 where Rob Roy pumping is reduced from the Baseline Simulation. Groundwater levels are generally lower at the Rob Roy well field to the coast for Scenario 3 where Rob Roy pumping is increased from the Baseline Simulation.

Table 7-4. Average Groundwater Level Difference (in feet) at Selected Wells for Scenarios 1-3 versus Baseline Simulation

Group	Well	Scenario 1: Shift from Cox to Rob Roy	Scenario 2: Modify Rob Roy 12	Scenario 3: Maximize Rob Roy and Cox
Cox	New Cox	-52	-52	-53
Rob Roy	Rob Roy 12	3.0	4.0	-3.2
	Rob Roy 4	2.2	1.9	0.0
	Rob Roy 10	3.1	2.4	0.7
	Modified Rob Roy 12	2.7	0.8	-3.3
Black	Black	0.0	0.0	-1.0
SqCWD Production Wells	Bonita	0.9	0.9	-0.7
	Country Club	0.5	0.5	-0.3
	Aptos Jr. High	0.5	0.6	-1.0
	SC-20A (Polo Grounds)	0.4	0.5	-1.4
	SC-20B (Polo Grounds)	0.6	0.7	-1.4
SqCWD Coastal Monitoring Wells	SC-A1A	0.2	0.2	-0.2
	SC-A1B	0.2	0.2	-0.2
	SC-A8A	0.3	0.3	-0.2
	SC-A8B	0.3	0.3	-0.2
Private Wells near CWD	01E09AP	-11	-11	-12
	01E04AP	-8	-8	-8
	01E04FP	-20	-20	-20
	01E04DP	-13	-13	-13

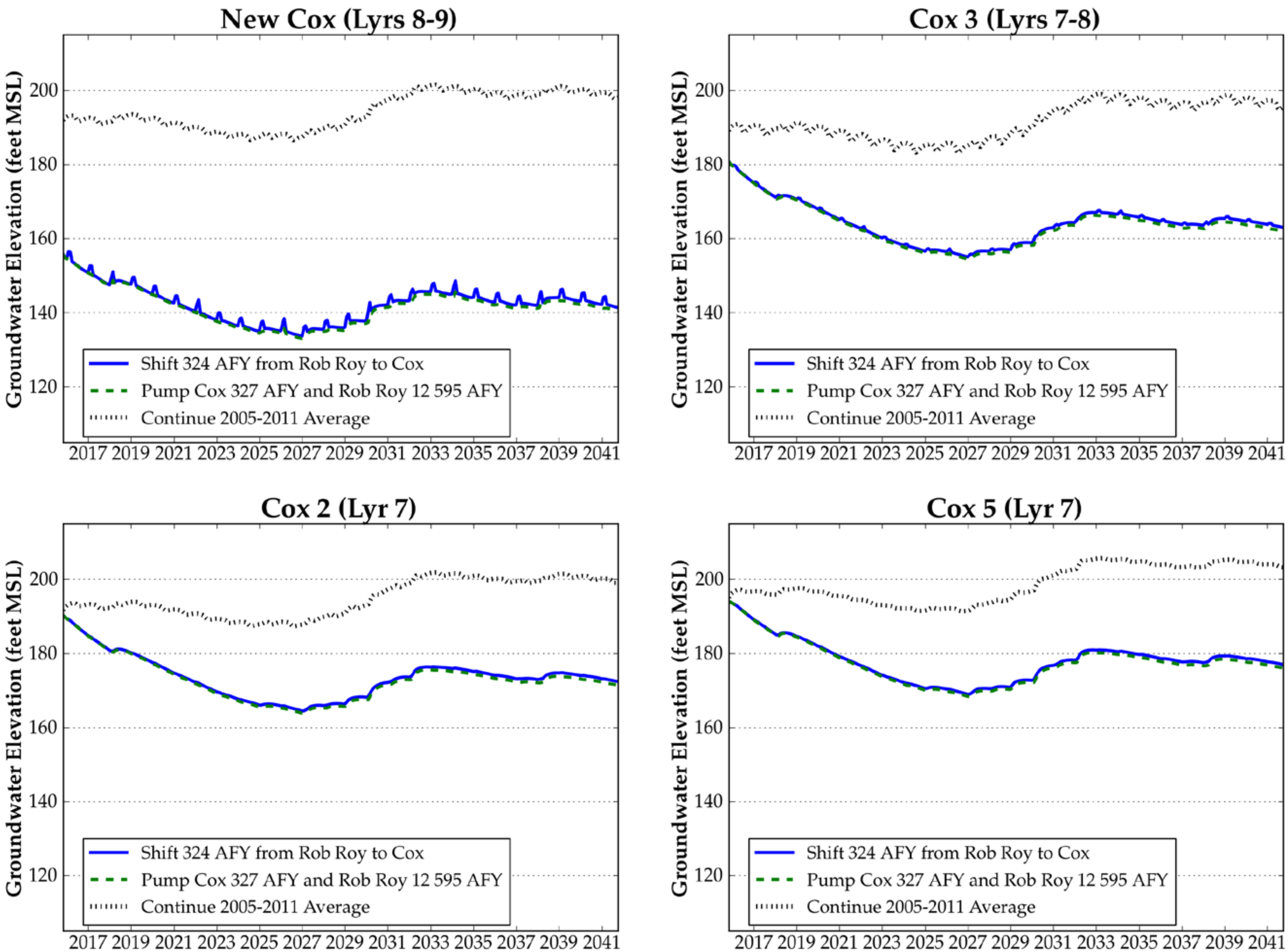

Figure 7-1. Scenarios 1 and 3 Hydrographs for Cox Well Field

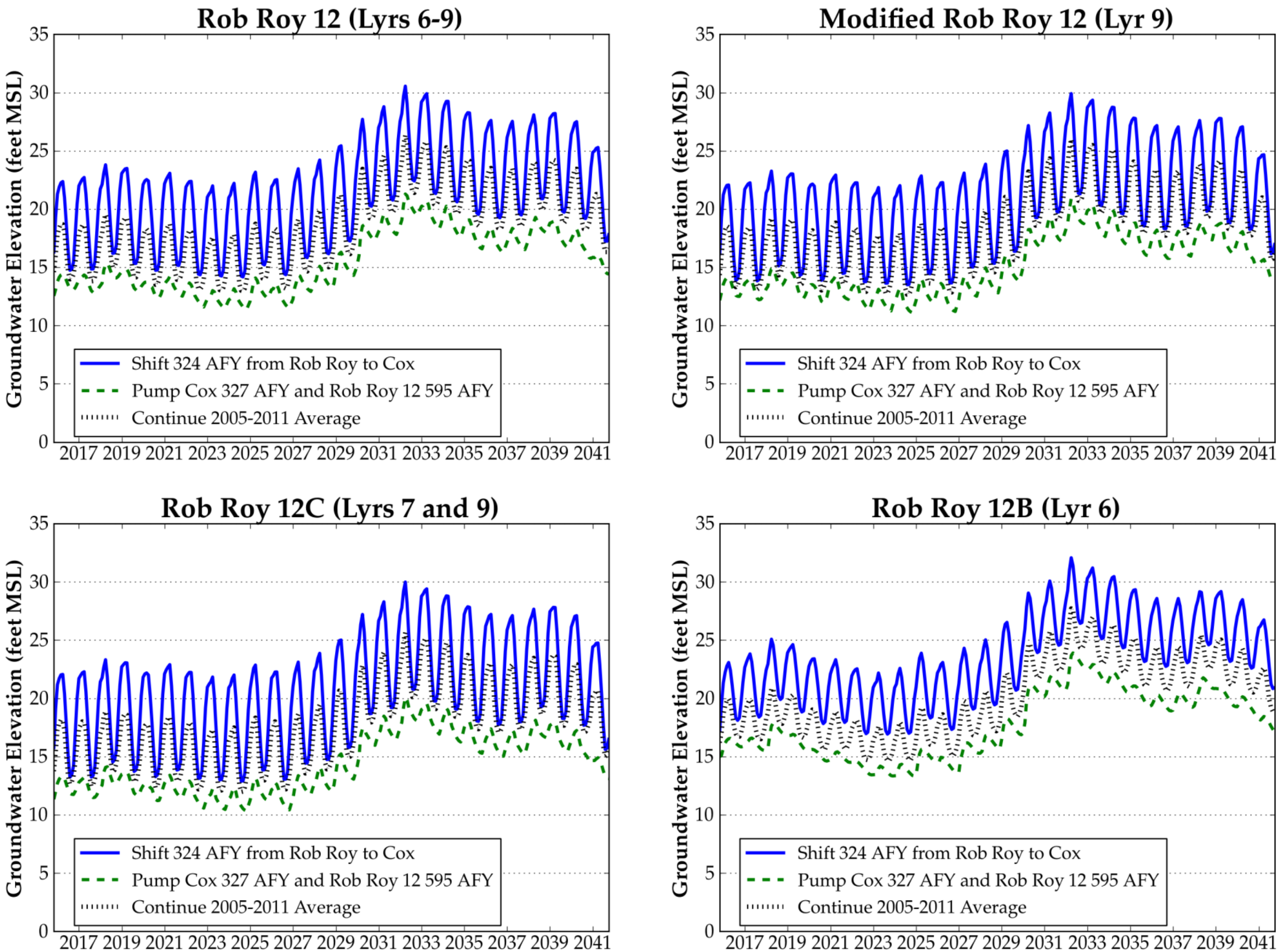

Figure 7-2. Scenarios 1 and 3 Hydrographs for Rob Roy 12 Wells Screened in Similar Layers

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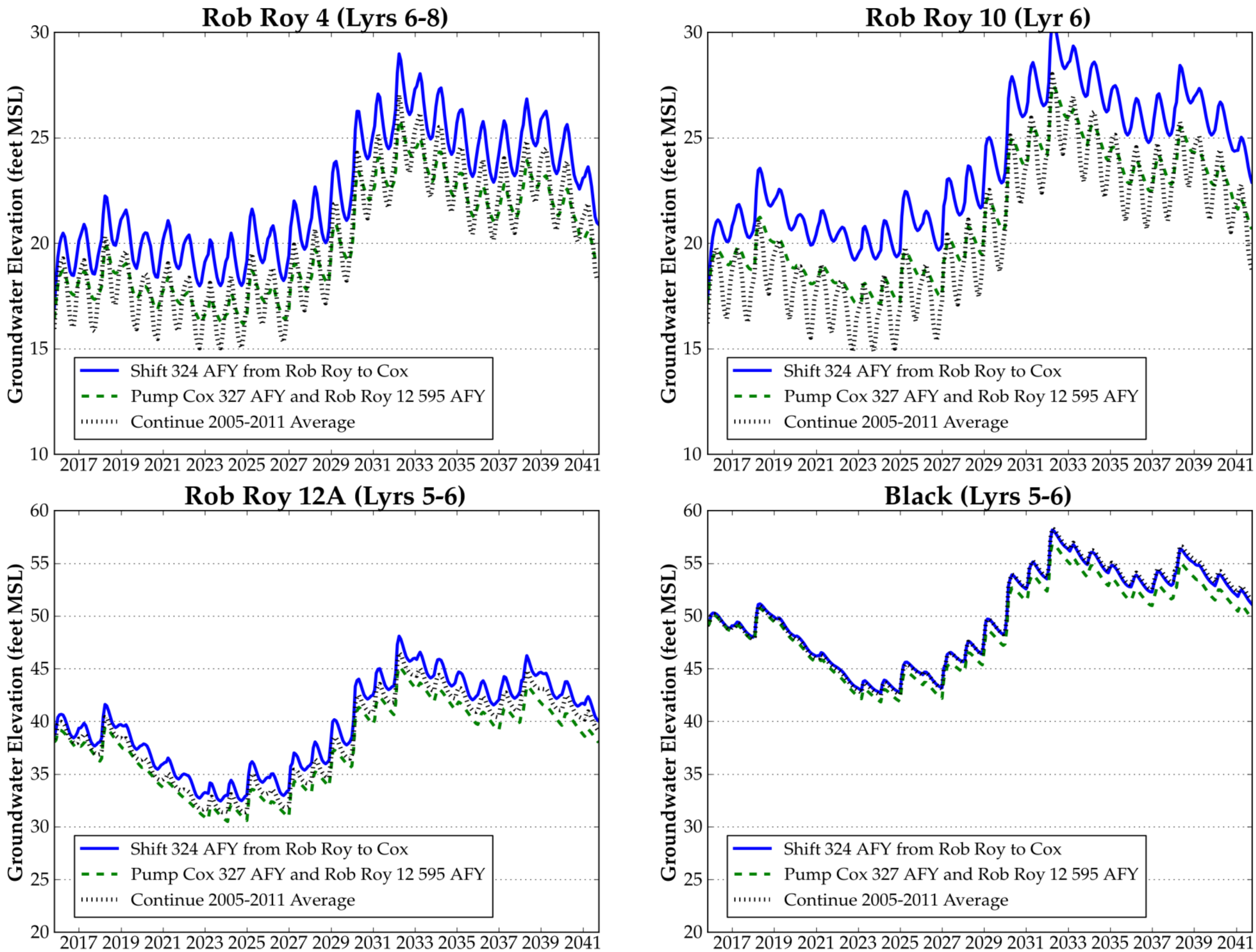

Figure 7-3. Scenarios 1 and 3 Hydrographs for Rob Roy 4, Rob Roy 10, Rob Roy 12A and Black Well

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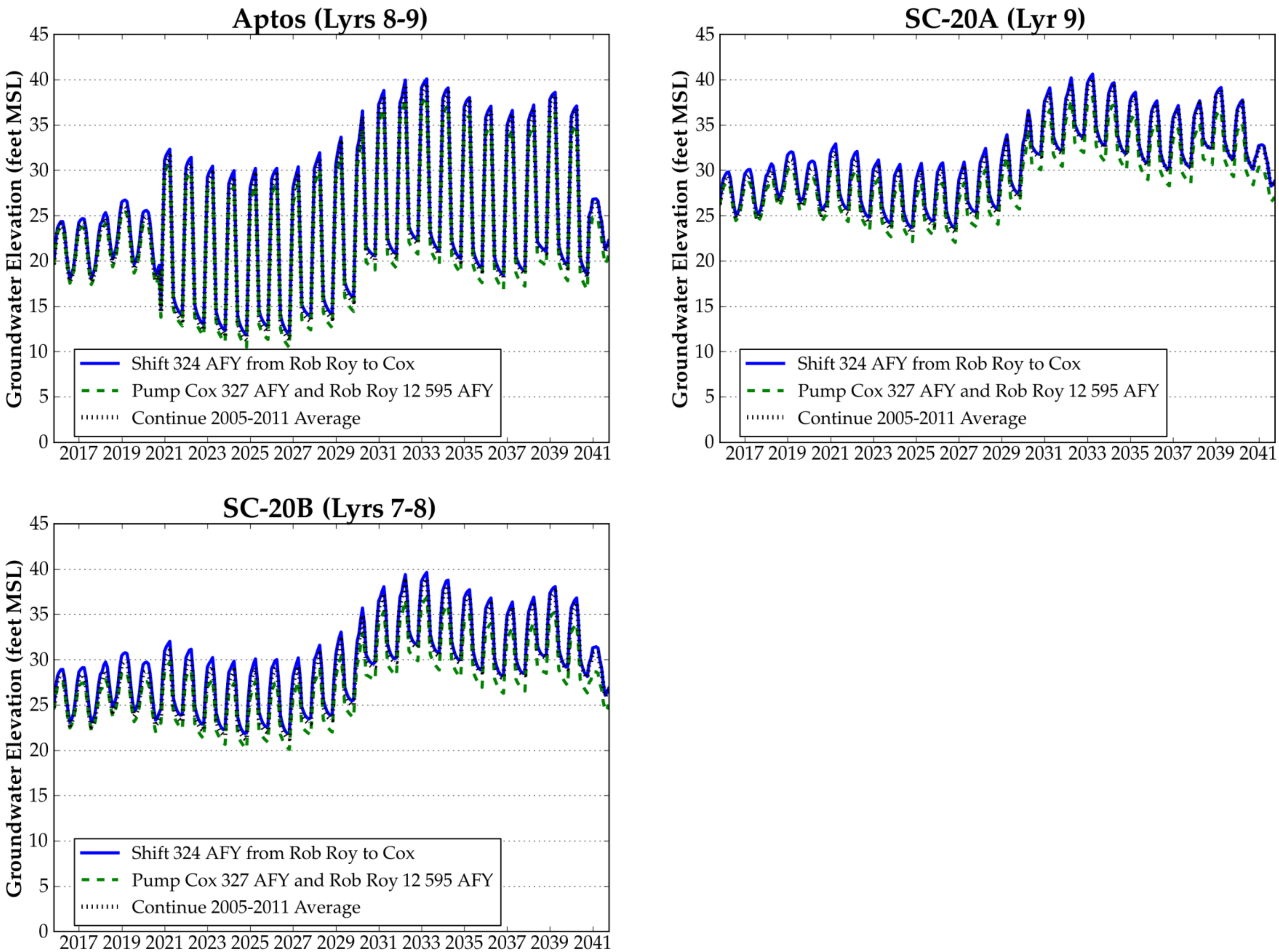

Figure 7-4. Scenarios 1 and 3 Hydrographs for Aptos Jr. High and Polo Grounds Wells

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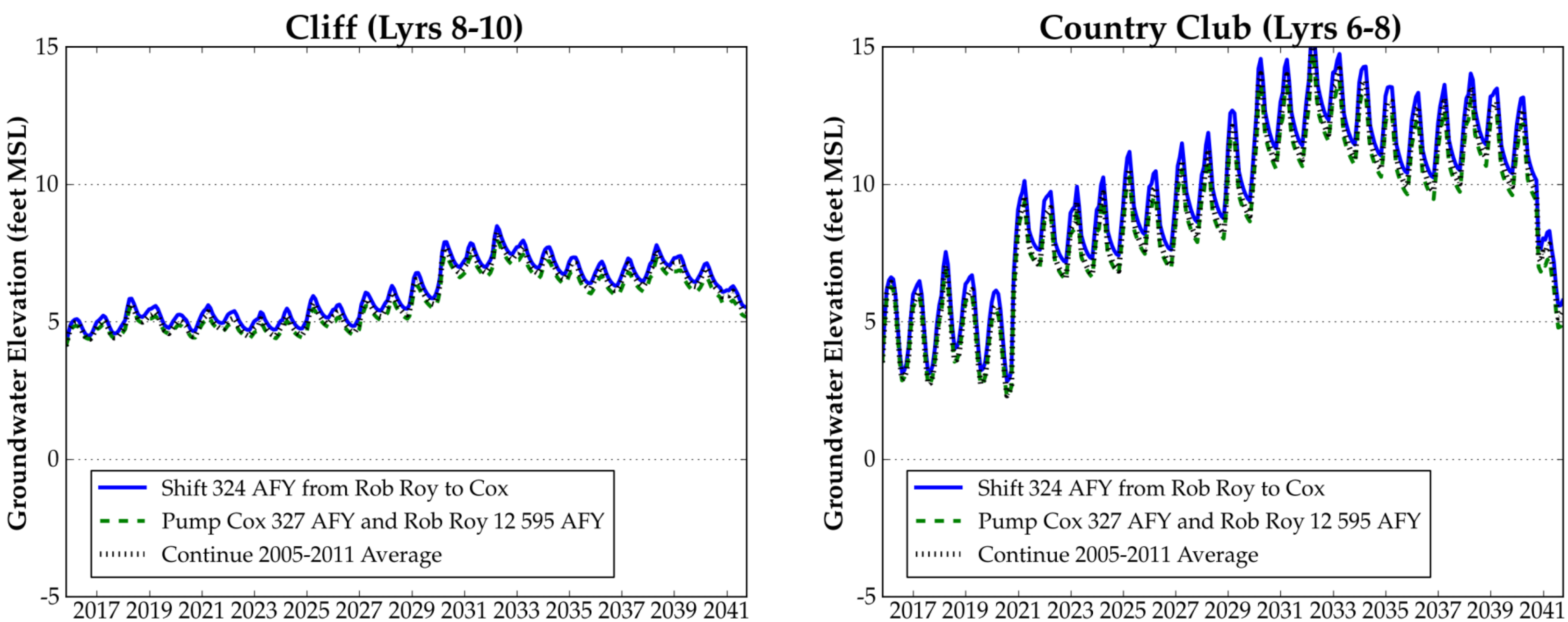

Figure 7-5. Scenarios 1 and 3 Hydrographs for Cliff and Country Club Wells

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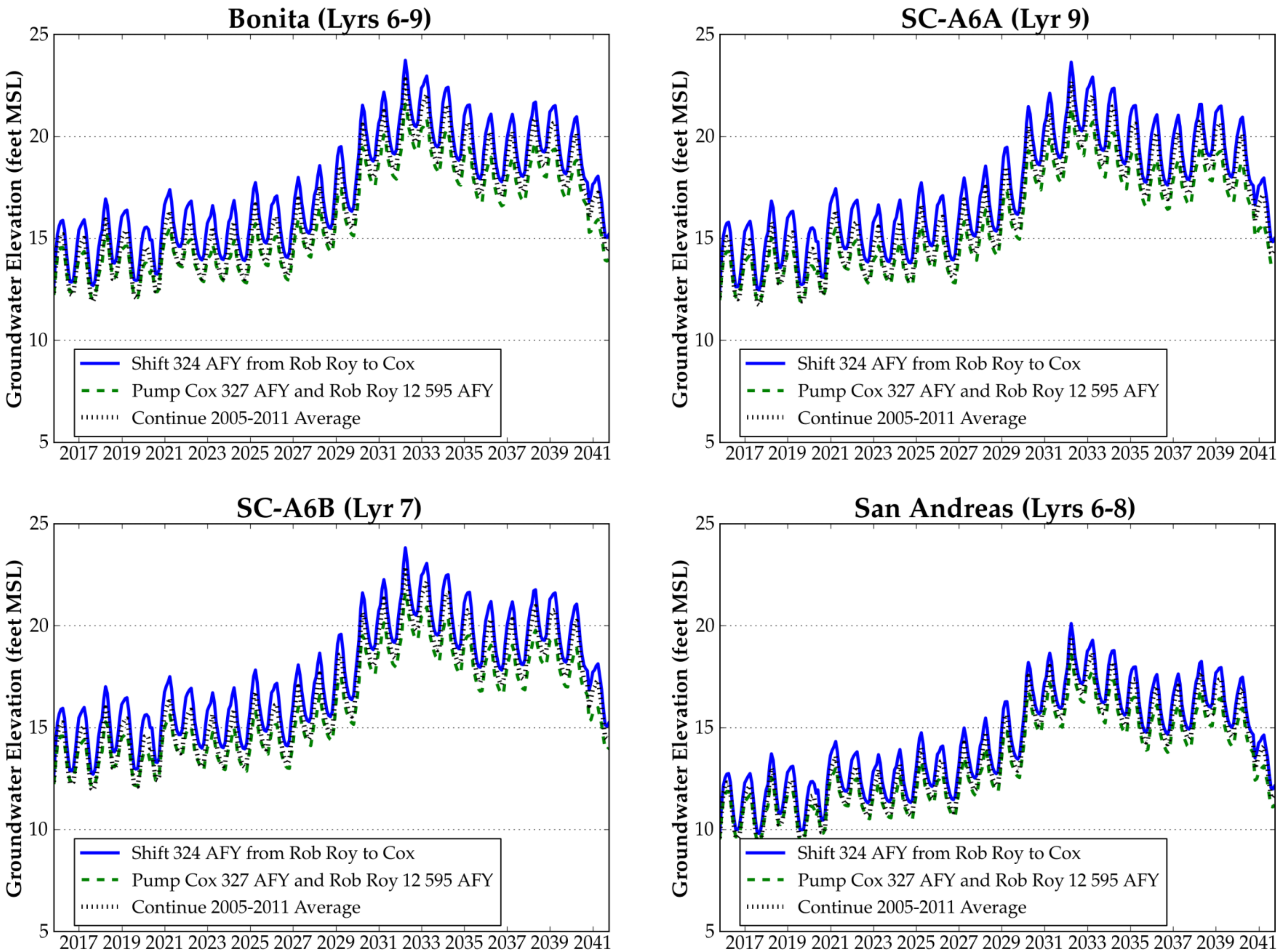

Figure 7-6. Scenarios 1 and 3 Hydrographs for Bonita and San Andreas Wells Screened in Similar Layers

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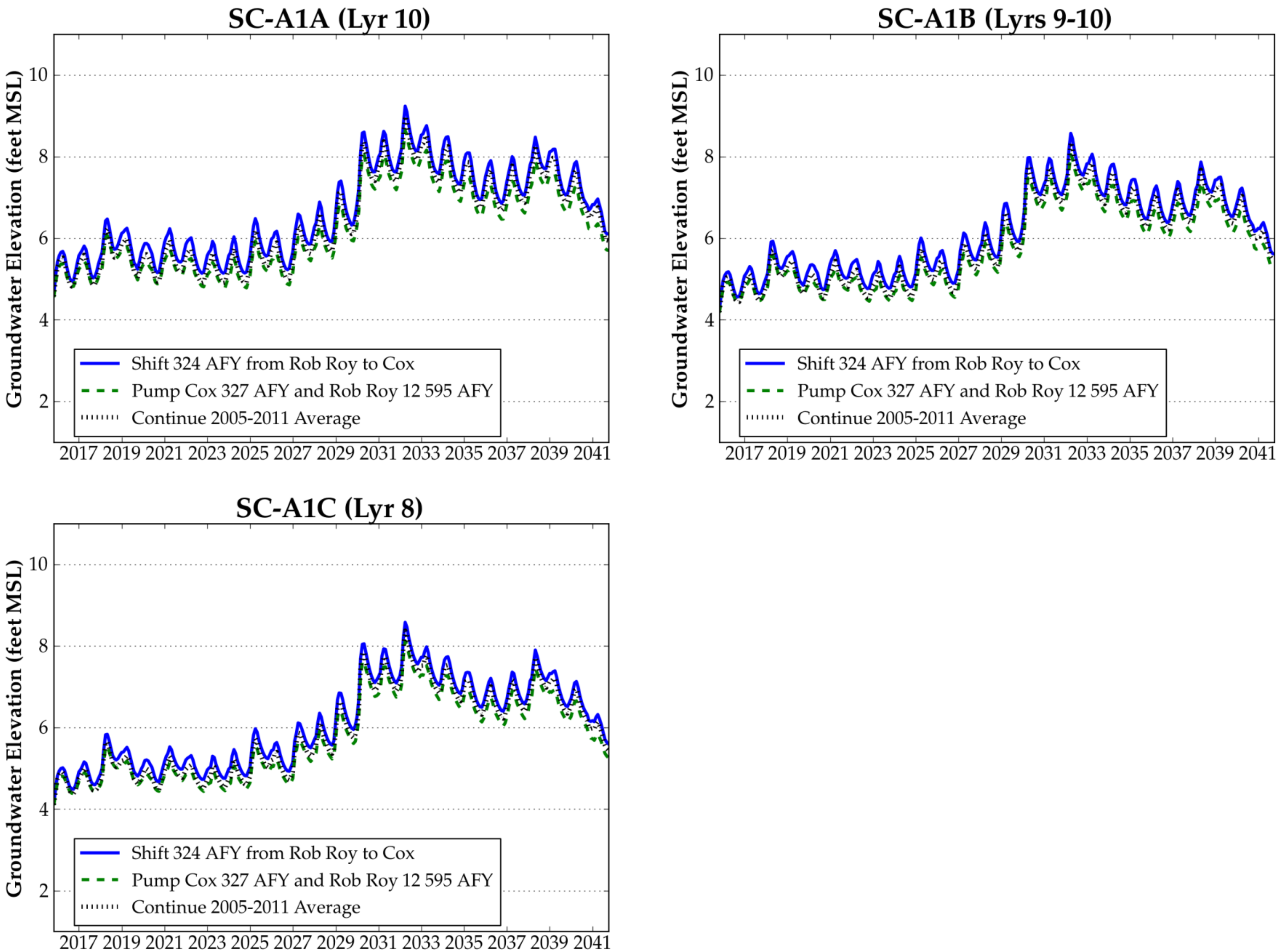

Figure 7-7. Scenarios 1 and 3 Hydrographs for SC-A1 (Cliff Drive) Coastal Monitoring Wells

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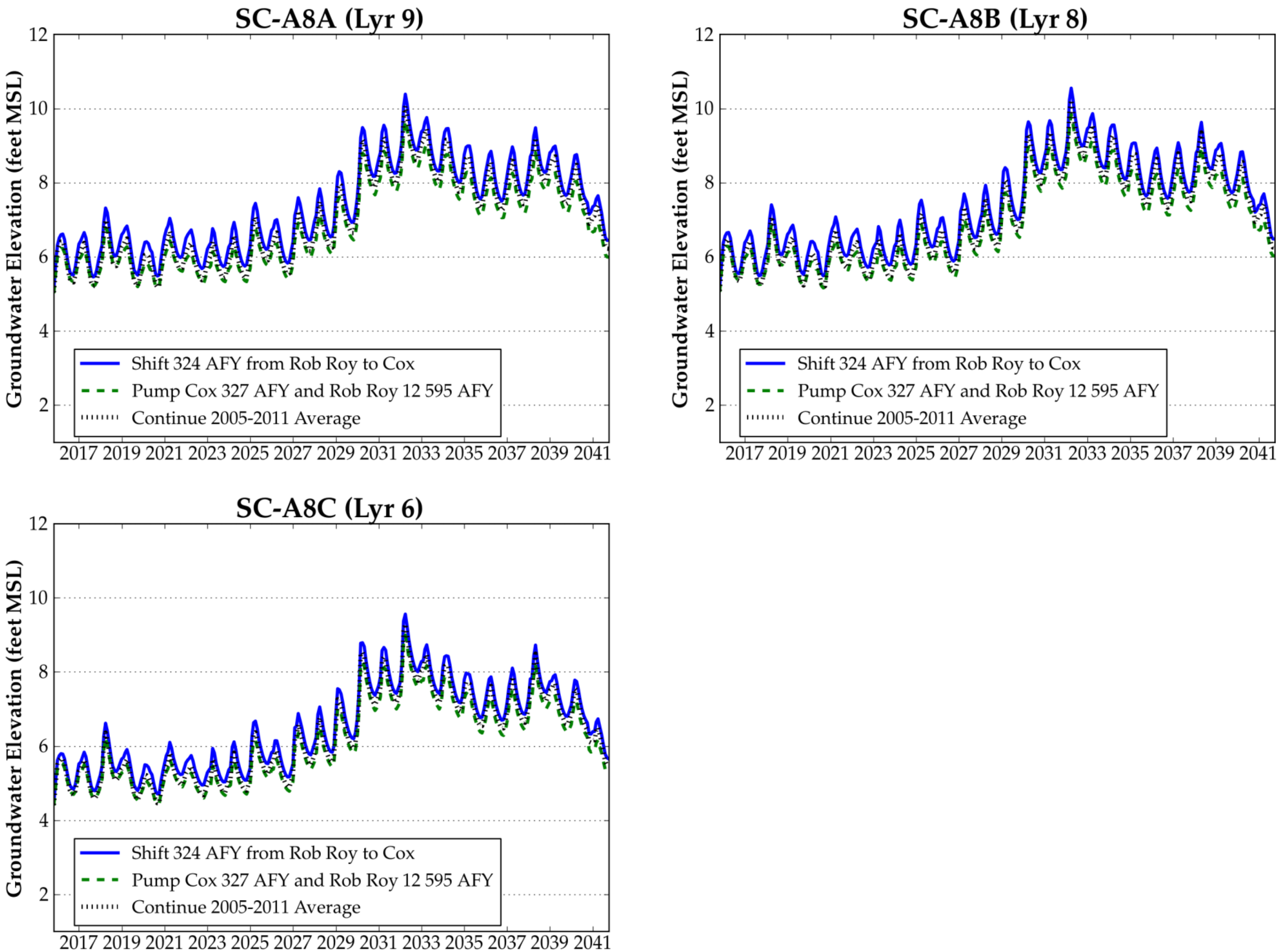

Figure 7-8. Scenarios 1 and 3 Hydrographs for SC-A8 (Dolphin and Sumner) Coastal Monitoring Wells

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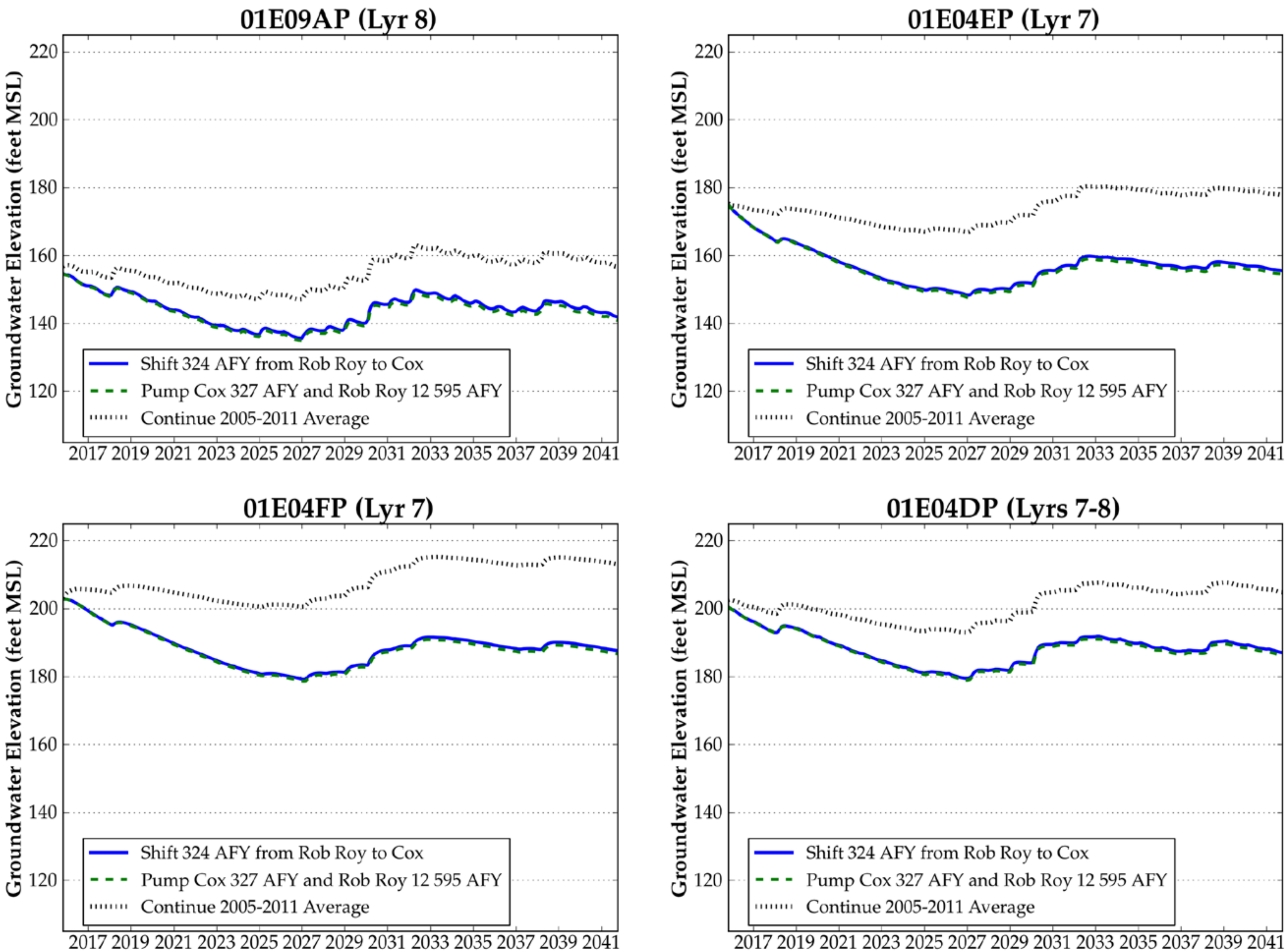

Figure 7-9. Scenarios 1 and 3 Hydrographs for Private Wells near Cox Well Field

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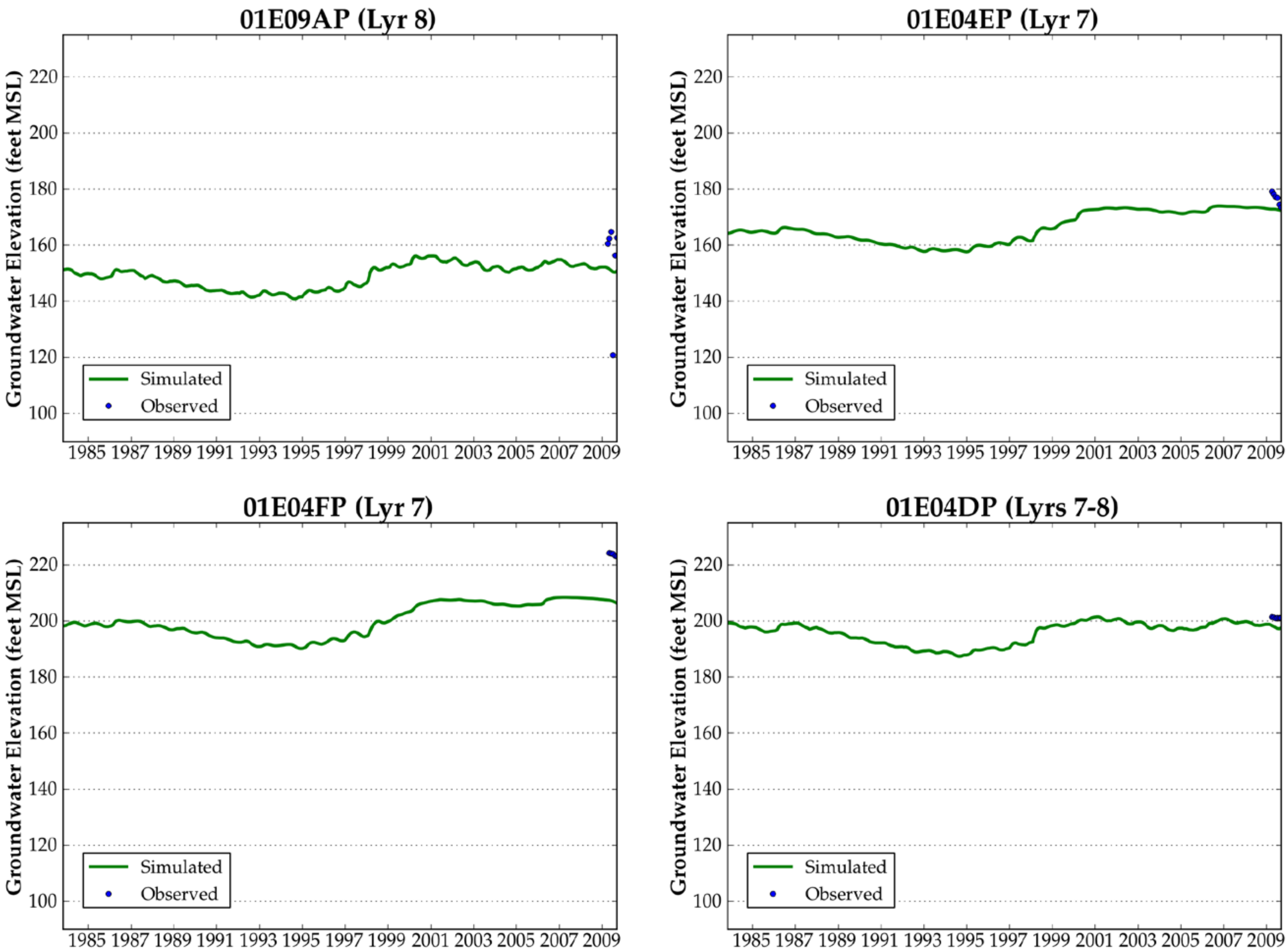

Figure 7-10. Hydrographs for Private Wells near Cox Well Field during Calibration Period

7.3.1 Scenario 1: Shift Pumping to New Cox Well

Figure 7-1 shows that shifting pumping to the new Cox well results in lower groundwater levels at the Cox well field than continuing current pumping but long-term groundwater level trends are stable. The stable trend indicates that the strategy to shift pumping to a new Cox well is within the sustainable yield of the Purisima Formation that supplies the Cox well field.

Figure 7-1 also shows that there is a multi-year decline in groundwater levels over the extended dry period simulated for Water Years 2016-2026 (hydrology from Water Years 1984-1994). Groundwater levels are simulated to drop approximately 20 feet over this period. This result should be considered when designing the new Cox well. The well design outlined in Section 2 placed the top of the well screen at a depth of 240 feet (50 feet msl) based on drawdown over a six-month period. In order to design the well to account for groundwater level declines over multiple years, the well should have a top of screen at least 20 feet deeper at a depth of 260 feet (30 feet msl). According to the hydrostratigraphic layering provided by Johnson, the Purisima F unit is estimated to have at least 350 feet of thickness below this depth that can be screened. The cost estimate for drilling the well provided in Section 8 assumes 660 feet of drilling with the goal of identifying the full depth of the F unit.

Figure 7-2 and Figure 7-3 show that the lower pumping at the Rob Roy well field in Scenario 1 results in higher groundwater levels at the Rob Roy well field than when continuing to pump current rates. However, Scenario 1 does not result in higher groundwater levels at the Black monitoring well upgradient and in a shallower layer than the Rob Roy well field.

SqCWD production well sites are closer to the Rob Roy well field than the Cox well field. Figure 7-4 through Figure 7-6 show that groundwater levels at SqCWD production well sites are marginally higher when pumping is shifted from Rob Roy well field to Cox well field than when continuing to pump current rates. The largest effect is at the Bonita well where groundwater levels for Scenario 1 are approximately 1 foot greater than the Baseline Simulation.

The effects at SqCWD coastal monitoring wells under Scenario 1 are even more marginal than at SqCWD production well sites (Figure 7-7 through Figure 7-8). The simulations show that reduction in SqCWD pumping is the main driver in reducing seawater intrusion risk as the Baseline Simulation shows a rise in groundwater levels at SC-A1 and SC-A8.

Figure 7-9 shows that groundwater levels at private wells near the Cox well field under Scenario 1 are up to 25 feet lower than the Baseline Simulation. These groundwater levels are also lower than minimum groundwater levels simulated at these wells (Figure 7-10) during the calibration period. The possible effect of these lower groundwater levels on supply at these private wells may need to be evaluated further.

7.3.2 Scenario 2: Improve Rob Roy #12 Water Quality

Table 7-3 shows that modifying Rob Roy #12 to improve water quality is not expected to improve system water quality, but the results of Scenario 2 are still examined to evaluate the effect of modifying Rob Roy #12 on groundwater levels. Comparing Scenario 1 and Scenario 2 results in Table 7-4 shows the effect of modifying Rob Roy #12 by isolating pumping in the deeper screen only has substantial effect at the Rob Roy well field.

Figure 7-11 shows that shifting pumping to the deepest screen at Rob Roy #12 results in lower groundwater levels at the production well even though pumping is reduced from the Rob Roy #12 well. Groundwater levels are lower at the Rob Roy #4 and Rob #10 wells in Scenario 2 than Scenario 1 because those wells make up for the reduced pumping at Rob Roy #12 well (Figure 7-12). Figure 7-12 also shows that the effect of shifting pumping deeper at Rob Roy #12 and farther away to Rob Roy #4 and Rob Roy #10 has almost no effect at the Black monitoring well.

7.3.3 Scenario 3: Maximize Rob Roy and Cox Pumping

Table 7-4, Figure 7-1, and Figure 7-9 show that groundwater level effects at the Cox well field and at private wells near the Cox well field from Scenario 3 are similar to Scenario 1 as Scenario 1 includes pumping that is close to the maximum pumping for the new Cox well.

Figure 7-2 and Figure 7-3 show lower groundwater levels for Scenario 3 than Scenario 1. At the Rob Roy well field, maximizing Rob Roy #12 pumping increases pumping from the Baseline Simulation and groundwater levels are lower for Scenario 3 than the Baseline Simulation. The Scenario 3 results show marginally lower groundwater levels at the Black monitoring well.

Figure 7-4 through Figure 7-8 show that maximizing Rob Roy pumping results in marginally lower groundwater elevations at the SqCWD production wells and

SqCWD monitoring wells. These marginally lower groundwater elevations closer to the coast may be outweighed by the benefit of providing water exceeding CWD's demand to water users outside CWD if the extra supply allows those water users to reduce pumping closer to the coast than the Rob Roy well field. If Scenario 3 represents an inland shift of pumping, it will enable coastal groundwater levels to rise and reduce seawater intrusion risk.

7.3.4 Stream Leakage Simulated for Scenarios

Streamflows for the Baseline Simulation and different scenarios are consistent with the modeling assumption that there is no leakage between the aquifer and streams. Maximum annual stream leakage simulated to and from Valencia Creek are less than 2% of total inflows to the Valencia Creek subbasin for the Basin Simulation and the three scenarios, in line with the results of the calibrated model.

7.4 Conclusions

The scenarios simulated by the groundwater model show that the groundwater management strategy considered by CWD to shift pumping from the Rob Roy well field to the Cox well field will be beneficial to CWD and improve basin management due to the following:

- Pumping a new well installed at the Cox well field at rates recommended in Section 2 is within the sustainable yield of the Purisima Formation.
- Revising design of the new well recommended at the Cox well field recommended in Section 2 with a lower screen will support pumping at recommended rates during multi-year dry periods.
- Pumping a new well at the Cox well field and constructing an iron and manganese treatment plant as discussed in Section 9 will improve system water quality by lowering chromium VI concentrations.
- Based on the draft MCL for chromium VI of 10 μg/L, the Rob Roy #12 well will be CWD's only Aromas well that meets drinking water standards. Installing a new well at the Cox well field and treating for iron and manganese will provide CWD with a second water source that meets drinking water standards and improve its water supply reliability by diversifying its supply.

- Developing the supply at the Cox well field potentially facilitates regional basin management by increasing inland supply in excess of CWD's demand that can be used to provide water to non-CWD users and reduce pumping closer to the coast. This strategy would reduce seawater intrusion risk.
- Effects on streamflow by the strategy are expected to be minimal.

The primary environmental effect of the strategy that may need further evaluation is the effect of predicted lower groundwater levels on the supply of private wells near the Cox well field.

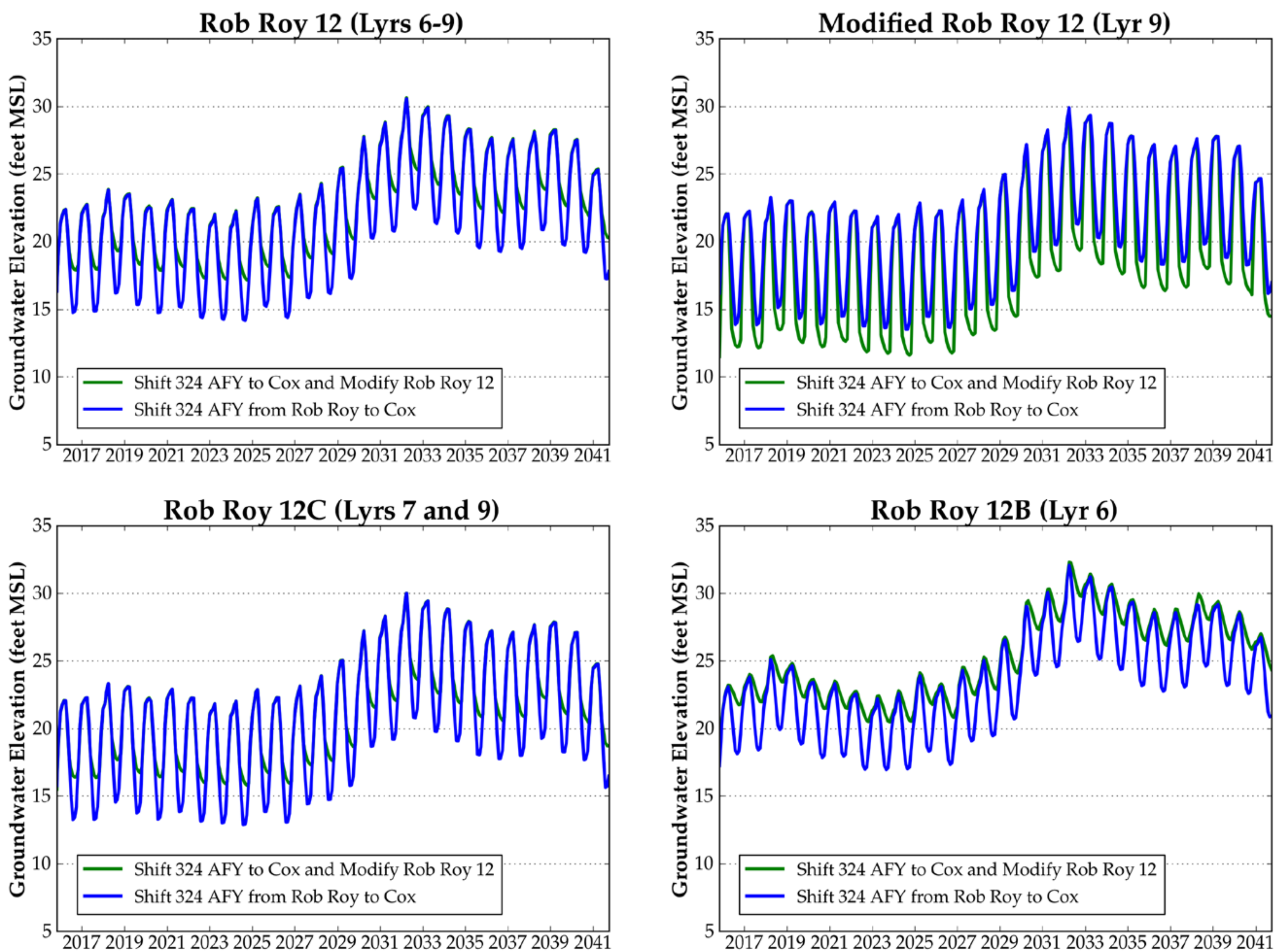

Figure 7-11. Scenarios 1 and 2 Hydrographs for Rob Roy 12 Wells Screened in Similar Layers

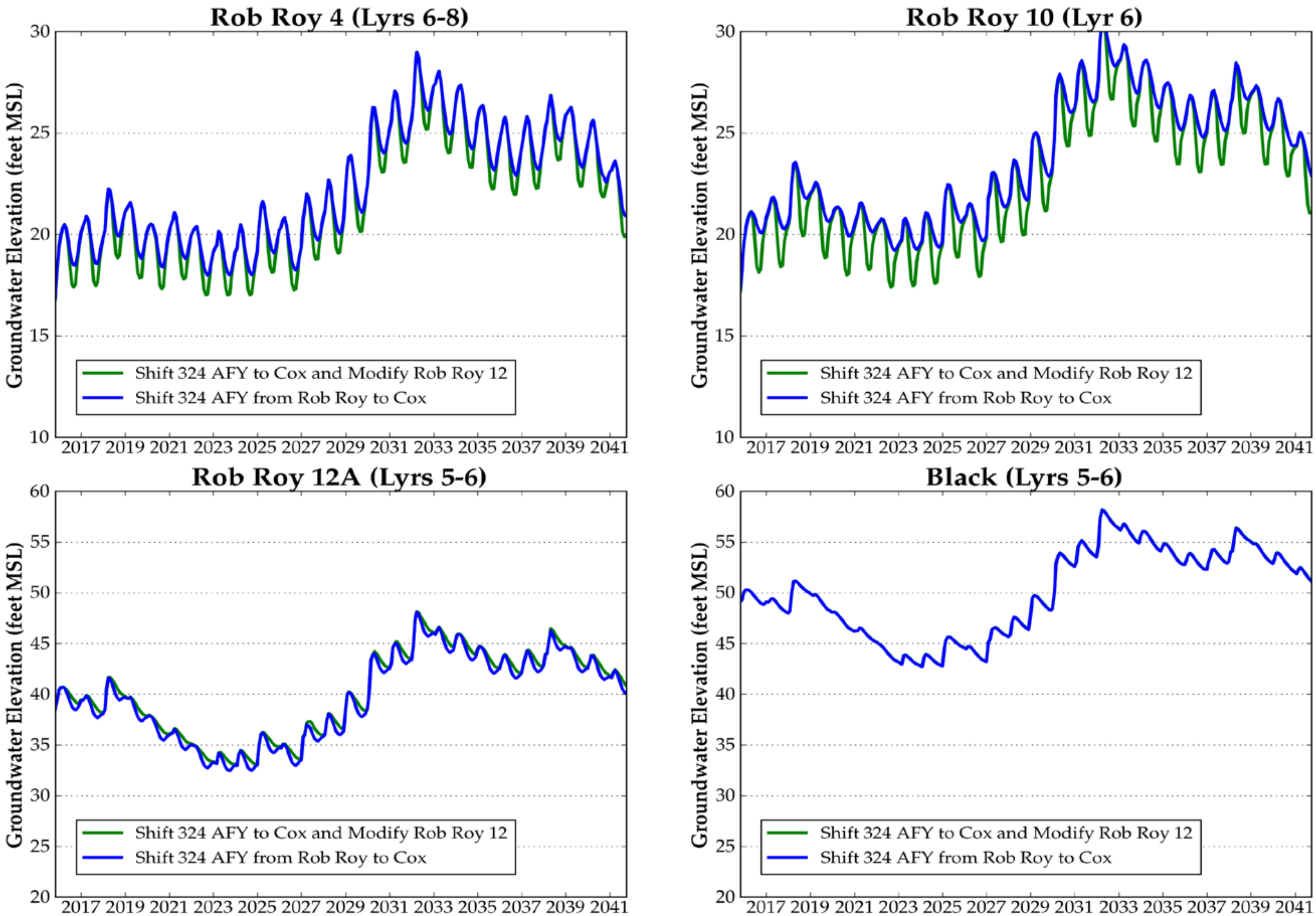

Figure 7-12. Scenarios 1 and 2 Hydrographs for Rob Roy 4, Rob Roy 10, Rob Roy 12A and Black Well

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SECTION 8
COST ESTIMATES FOR WELL RECOMMENDATIONS (TASK 4.3)

For grant Task 4.3, Section 3 recommends that existing wells at the Cox well field be replaced with a single primary production well designed and constructed using modern practices. For grant Task 4.4, Section 9 recommends that the existing well Cox #5 be rehabilitated as a backup well while Cox #2 and #3 wells be properly destroyed.

This report section summarizes the cost estimate for constructing and developing the new well and destroying two of the existing wells. The cost estimate for rehabilitation of the Cox #5 well to be a backup well is included in Section 9.

The cost estimates include preparation of a preliminary design report and technical specifications (~$25,000, Table 8-1), the drilling contractor (~$560,000, Table 8-2), and hydrogeologic oversight of the drilling contractor (~$108,000 Table 8-3). The total estimated cost is approximately $693,000. This cost estimate was provided as a draft technical memorandum to the Technical Advisory Committee (TAC) on June 5, 2013.

The approach for the cost estimate was to provide guidance for CWD to plan a budget that will not limit options in design of the well. If and when budgetary constraints are identified, the design can be adjusted accordingly. In any case, the preliminary design report will need to justify design choices that add to the cost. Specific assumptions in developing the cost estimate are discussed below.

In an addition to grant Task 4.2, Section 4 discusses the potential for modifying the Rob Roy #12 well to improve water quality at the well. This section also summarizes the cost of modifying Rob Roy #12 well. Three modification strategies are presented with costs ranging from $23,000 to $43,000 (Table 8-5). This cost estimate was provided as a draft technical memorandum to the Technical Advisory Committee (TAC) on July 2, 2013.

8.1.1 Assumptions for Preliminary Design Report and Technical Specifications (Table 8-1)

1. The estimate for preparing technical specifications includes specifications for well destruction, well construction, well development, and well testing.

   Assumptions for Drilling Contractor Services (Table 8-2)

2. The estimated cost for the well destruction is based on cost of destroying the San Lorenzo Valley Water District's Pastiempo #5 well by Welenco. Most of the cost items are lump sums. The cost of destroying the two wells at Cox will be less than 2 times the lump sum for several of the items. Grout pump rental will not increase for the second well. The mobilzation/demobilization and permitting item only needs to add a second permit. Well casing blank that needs to be perforated at the two Cox wells is approximately 400 feet versus 300 feet at the Pasatiempo #5 well.

3. There is a high mobilization cost because contractor may not be local, and this estimate would have to include travel time and per diem expenses. This number is derived from the average of the two mobilization costs from Zim Industries (Fresno, CA) for SqCWD's O'Neill Ranch well and Maggiora Brothers (Watsonville, CA) for San Lorenzo Valley Water District's Pasatiempo well, both constructed in 2012.

4. The estimate for pilot bore drilling and reaming the pilot bore was based on the higher cost of the O'Neill Ranch well as opposed to the lower cost of the Pasatiempo well. However, the pilot bore and final diameter of the well bore are both larger at O'Neill Ranch well than the new Cox well, so this is a high end cost estimate.

5. Stainless steel was chosen so the well would not rust and provide the longest life for this well. This is the most expensive well casing material. The other options are corten steel, which will last about 50 years based on existing Cox wells. Another option is corten steel above the water table, joined with a dielectric coupling to stainless steel for the portion of the well that will be underwater.

6. Louvers were chosen for the screens because they are the most durable and easiest to rehab. However, louvers are the most expensive perforation option. Wire wrapped screen is less expensive but has a higher risk for damage during rehabilitation and development. Louvered screen can also be repaired by swedging, while wire-wrapped screen cannot. The cost of louvered screen can be lowered by using blank sections in non-water bearing zones, but the estimate assumes that the entire interval of 300 feet is louvered because the blank sections will be determined during preliminary and final design.
7. Mechanical development is estimated as 50 hours. The total time spent for mechanical development for the O'Neill Ranch well was 61 hours. However, because the new Cox well will be constructed using 12 inch casing instead of 16 inch casing there is less surface area to develop and less volume of water should be produced during the development process, so this is a high end estimate.
8. The pump development time was also based on the time spent for the O'Neill Ranch well. Development time at the O'Neill Ranch well was controlled by treatment limitations so if treatment meets water quality standards faster at the Cox well, this procedure could take less than 40 hours and cost less.
9. The estimate for installing and removing the test pump is based on the higher cost at the O'Neill Ranch well as opposed to the Pasatiempo well. The pump used at the O'Neill Ranch was larger and that size pump may not be needed at the Cox #5 well, so this is a high end cost estimate.
10. A full 8 hour step test and full 24 hour constant rate test are included in the cost estimate. This is a high end cost assumption for aquifer testing, because both tests could be run for a shorter period of time. Since flow and water quality profiling has already been performed at Cox #5 (Section 2), profiling is not included in this cost estimate.
11. The estimate for the treatment plant to treat development water from the well is based on the lump sum value from the O'Neill Ranch well. The discharge rate at O'Neill Ranch well was likely higher than what will

occur at the new Cox well, so if a treatment system is required it may be able to be scaled down, and therefore not be as expensive. In addition, the site at Cox is much larger than at O'Neill Ranch, so it may be possible to bring more baker tanks on site and let the heavy drilling fluids settle out to reduce the turbidity. There is also the option of pumping discharge water to the holding pond. Another possibility is to drill the well in the summer and discharge to a dry creek bed, which may eliminate need to reduce turbidity with no receiving water. However, a treatment system may be required and is therefore included in the cost estimate.

12. Tasks such as noise control/sound wall and hauling/disposing of cuttings may not be necessary, but are included in the estimate total

8.1.2 Assumptions for Hydrogeologic Oversight of Field Activities (Table 8-3)

1. Discharge permitting is based on experience at O'Neill Ranch well where permitting of discharge into a Soquel Creek tributary was challenging. Permitting discharge at the Cox site may not be as challenging; for example, Valencia Creek, the potential receiving body may be dry during well development.

2. Well drilling and construction (Tasks 3B-3D) will be conducted on a 24 hour schedule and pilot hole drilling (Task 3B) and well construction (Task 3D) requires 100% oversight. The cost estimate reflects part-time oversight of borehole reaming (Task 3C).

3. Even as field activity is limited to day-time hours with development and step testing, 100% oversight is not required. The cost estimate reflects part-time oversight of development and testing (Tasks 3E and 3F), including the 24 hour constant rate test.

4. The cost estimate is based on assumed duration of two weeks for drilling and construction, two weeks for development, and two days for testing. Experience has shown that the time taken for these tasks is a function of contractor competence and diligence, factors of which the hydrogeologic

consultant has little control, especially under low-bid procurement procedures.

Tasks	HydroMetrics WRI Labor				Labor Total		Other Direct Costs	TOTALS
	Derrik Williams	Cameron Tana	Georgina King	Staff				
	President	Principal Engineer	Senior Hydrogeologist	Hydrogeologist				
Rates	$215	$195	$185	$115	Hours	($)	($)	($)
Task 1. Preliminary Design Report								
a. Review Background Data	2	2	8	0	12	$ 2,300	$ -	$ 2,300
b. Prepare Preliminary Design Report	6	2	30	2	40	$ 7,460	$ 60	$ 7,520
Subtotal Task 1	***8***	***4***	***38***	***2***	***52***	***$ 9,760***	***$ 60***	***$ 9,820***
Task 2. Technical Specifications								
a. Prepare Technical Specifications	8	8	48	28	92	$ 15,380	$ 100	$ 15,480
TOTAL	***16***	***12***	***86***	***30***	***144***	***$ 25,140***	***$ 160***	***$ 25,300***

Table 8-1. Cost Estimate to Prepare Preliminary Draft Report and Technical Specifications

Table 8-2. Engineer's Cost Estimate for Drilling Contractor Services

			Engineer's Estimate	
Item Description	**Units**	**Quant.**	**Unit Rate**	**Item Price**
Destruction of Cox #2 and #3 Wells				
Mob/Demob & Permit	Lump Sum	1.25	$2,000.00	$2,500.00
Bail Well, Install Tremie and Grout Well	Lump Sum	2	$2,400.00	$4,800.00
Blast Perforation of Well Casing	Lump Sum	1.5	$4,000.00	$6,000.00
10 Sack Sand Slurry Grout	Cubic Yards	28	$300.00	$8,400.00
Grout Pump Rental	Lump Sum	1	$650.00	$650.00
Excavation of Top Five Feet of Well	Lump Sum	2	$1,200.00	$2,400.00
Construction, Development, and Testing of Cox Replacement Well				
Mobilization and Demobilization	Lump Sum	1	$50,750.00	$50,750.00
Noise Control/Sound Barrier (optional)	Lump Sum	1	$12,000.00	$12,000.00
Conductor Casing	Linear Feet	55	$300.00	$16,500.00
Pilot Bore Drilling	Linear Feet	660	$70.00	$46,200.00
Geophysical Logging	Lump Sum	1	$2,500.00	$2,500.00
Reaming Pilot Bore	Linear Feet	660	$60.00	$39,600.00
Caliper Survey	Lump Sum	1	$2,000.00	$2,000.00
12-inch Diameter SS Blank Casing	Linear Feet	350	$215.00	$75,250.00
12-inch Diameter SS Well Screen	Linear Feet	300	$270.00	$81,000.00
12-inch Diameter SS Cellar with	Lump Sum	1	$4,200.00	$4,200.00
2-inch Diameter SS Gravel Feed Pipe	Linear Feet	180	$17.00	$3,060.00
Gravel Pack	Linear Feet	500	$40.00	$20,000.00
Cement Grout	Linear Feet	155	$40.00	$6,200.00
Bentonite Seal	Linear Feet	5	$80.00	$400.00
Install and Remove Test Pump	Lump Sum	1	$9,500.00	$9,500.00
Disposal Water Treatment System	Lump Sum	1	$98,800.00	$98,800.00
Mechanical Well Development	Hourly	50	$350.00	$17,500.00
Pumping Well Development	Hourly	40	$250.00	$10,000.00
Aquifer Testing of Well	Hourly	32	$250.00	$8,000.00
Alignment of Well	Lump Sum	1	$2,000.00	$2,000.00
Video Survey	Lump Sum	1	$1,000.00	$1,000.00
Disinfection	Lump Sum	1	$1,500.00	$1,500.00
Site Clean Up	Lump sum	1	$15,000.00	$15,000.00
Standby Time	Hourly	0	$300.00	$0.00
Haul and Dispose of Cuttings (optional)	Lump Sum	1	$12,000.00	$12,000.00
			Total with optional items	**$559,710.00**

Table 8-3. Cost Estimate to Provide Hydrogeologic Oversight of Field Activities

	HydroMetrics WRI Labor											
	Derrik Williams	**Cameron Tana**	**Field Lead**		**Georgina King**	**Staff**		**Admin**	**Labor Total**		**Other Direct Costs**	**TOTALS***
	President	Principal Engineer	Principal Hydrogeologist		Senior Hydrogeologist	Hydrogeologist		Office Support				
Tasks	Office	Office	Office	Field Work	Office	Office	Field Work	Office				
Rates	$215	$195	$195	$150	$185	$115	$115	$65	**Hours**	**($)**	**($)**	**($)**
Task 1. Bid Assistance												
1A. Pre-Construction Meeting	0	0	0	8	0	0	0	0	8	$ 1,200	$ 90	$ 1,290
1B. Contractor Questions/Issues	0	2	4	0	4	0	0	0	10	$ 1,910	$ -	$ 1,910
1C. Bid Selection	0	1	2	0	0	0	0	0	3	$ 585	$ -	$ 585
Subtotal Task 1									***18***	***$ 3,110***	***$ 90***	***$ 3,785***
Task 2. Permitting												
2A. Discharge Permitting	2	4	24	4	32	16	0	0	82	$ 14,250	$ 100	$ 14,350
Subtotal Task 2									***82***	***$ 14,250***	***$ 100***	***$ 14,350***
Task 3. Well Construction, Development and Testing												
3A. Mobilization	0	0	0	10	0	0	0	0	10	$ 1,500	$ 200	$ 1,700
3B. Conductor, Pilot Hole Drilling, and Geophysics	0	0	0	72	2	0	72	0	146	$ 19,450	$ 2,200	$ 21,650
3C. Borehole Reaming	0	0	0	4	0	0	16	0	20	$ 2,440	$ 850	$ 3,290
3D. Well Construction	0	0	0	48	1	0	48	0	97	$ 12,905	$ 1,550	$ 14,455
3E. Well Development	0	1	0	20	0	0	40	0	61	$ 7,795	$ 4,200	$ 11,995
3F. Aquifer Testing	0	2	0	0	2	0	24	0	28	$ 3,520	$ 530	$ 4,050
3G. Well Destruction	0	0	0	0	1	0	12	0	13	$ 1,565	$ 170	$ 1,735
3H. Demobilization	0	0	0	4	0	0	0	0	4	$ 600	$ 200	$ 800
Subtotal Task 3									***379***	***$ 49,775***	***$ 9,900***	***$ 59,675***
Task 4. Well Report												
4A. Draft Documentation	2	6	16	0	40	40	0	0	104	$ 16,720	$ -	$ 16,720
4B. Final Documentation	1	4	4	0	4	4	0	2	19	$ 3,105	$ 500	$ 3,605
Subtotal Task 4									***123***	***$ 19,825***	***$ 500***	***$ 20,325***
Total without Contingency										***$ 86,960***	***$ 10,590***	***$ 98,135***
Contingency 10%												***$ 9,814***
TOTAL									***602***	***$ 86,960***	***$ 10,590***	***$ 107,949***

8.2 Cost Estimate for Rob Roy #12 Modification

Section 4 evaluated whether modifying the Rob Roy #12 well could reduce chromium VI concentrations because the well head concentration of 3.7 µg/L may be above a future drinking water standard. It was estimated that 70% of the chromium VI, but only 26% of the flow is produced by the upper screen. Therefore, modifying the well so less water is produced from the upper screen may reduce chromium VI concentrations while maintaining the majority of flow.

Section 4 also evaluated effects of well modification on iron and manganese concentrations. The iron concentration at the well head is currently 86 µg/L, below the drinking water standard of 300 µg/L. By limiting flow from the upper 290 feet of the well, the overall iron concentrations may increase, but should stay below 300 µg/L. The concentration at 420 feet was measured at 422 µg/L, but concentrations in the cumulative flow between that depth and 290 feet were all below 30 µg/L. Manganese is non-detect at a detection limit of 10 µg/L throughout the well. The concentrations for chromium VI, iron and manganese collected by depth-discrete sampling while Rob Roy #12 was pumping are shown in Table 4-3.

Table 8-4. Chromium VI, Iron and Manganese Concentrations with Depth in Rob Roy #12

Sample Depth (ft)	Chromium VI (µg/L)	Iron (µg/L)	Manganese (µg/L)
Well Head	3.7	86	ND<10
245	3.7	26	ND<10
265	2.6	14	ND<10
290	1.6	27	ND<10
305	1.6	11	ND<10
315	0.68	22	ND<10
335	ND<1.0	16	ND<10
420	ND<1.0	422	ND<10
500	ND<1.0	71	ND<10

Since the flow and water quality profile is based on the current setup of the well, any well modification needs to be tested to make sure water supply and quality objectives are being met. It is also recommended that any modification be

reversible in case the modification does not meet objectives or modification is no longer necessary.
This section summarizes cost estimates for implementing and testing two strategies to extract less flow from the upper screen and reduce the amount of chromium VI pumped by the well. The first strategy is simply to lower the pump to 480 feet below ground surface (currently the pump is set at 220 feet below ground surface). The second strategy is to seal off the upper screen by placing an inflatable packer on the pump column down to the blank section below the upper screen, and lowering the pump to the same depth as Strategy 1. An inflatable packer is recommended because it is a modification that can be removed if needed at a later date with no damage to the well. Apart from providing costs for each of the strategies, our cost estimate also includes the cost to implement Strategy 2 following Strategy 1 in the event that Strategy 1 is unsuccessful.

The cost estimate includes time for HydroMetrics Water Resources Inc. to prepare technical specifications for the two strategies, provide oversight of the contractor, and to prepare a report documenting the work on the well and the outcome of the modifications. The estimate assumes three groundwater quality samples are collected for each of the two options (total of nine samples). Samples will be collected at the start of pumping, after pumping for 24 hours, and after pumping continuously for 72 hours. The first sample will be collected by the consultant, while the second and third samples will be collected by CWD staff.

Table 8-1 provides total costs (consultant and contractor) for the two strategies plus costs if Strategy 2 follows Strategy 1. Laboratory costs for water quality analyses are included under other direct costs. We have assumed that Title 22 and analysis for chromium VI will be analyzed for each of the six samples. Table 8-6 details the line items for work the contactor would carry out for the different strategies.

Table 8-5. Cost Estimate to Modify the Rob Roy #12 Well

	Principal Hydrogeologist	Senior Engineer	Senior Hydrogeologist	Hydrogeologist	Labor Total		Other Direct	TOTALS
Rates	$215	$195	$180	$150	**Hours**	**($)**	**($)**	**($)**
Strategy 1 Only								
Consultant to Prepare Technical Specifications for Strategy 1 - Lowering the Pump	1	2	2	6	11	$ 1,865	$ -	$ 1,865
Consultant Oversight of Field Testing Strategy 1: Lowering Pump, including Collecting and Analyzing Three Samples for Chrome VI and Title 22 Inorganics	1	2	2	8	13	$ 2,165	$ 1,400	$ 3,565
Consultant to Prepare Summary Report for Strategy 1 Only	1	3	6	12	22	$ 3,680	$ 50	$ 3,730
Contractor Costs for Strategy 1 only								$ 14,220
Strategy 2 Only								
Consultant to Prepare Technical Specifications for Strategy 2 - Installing Inflatable Packer below Upper Screen	1	2	2	10	15	$ 2,465	$ -	$ 2,465
Consultant Oversight of Field Testing Strategy 2: Install Inflatable Packer, including Collecting and Analyzing Three Samples for Chrome VI and Title 22 Inorganics	1	2	2	12	17	$ 2,765	$ 1,400	$ 4,165
Consultant to Prepare Summary Report for Strategy 2 Only	1	3	8	12	24	$ 4,040	$ 50	$ 4,090
Contractor Costs for Strategy 2 only								$ 19,720
Strategy 1 followed by Strategy 2								
Consultant to Prepare Technical Specifications for Strategy 1 and 2	2	4	4	16	26	$ 4,330	$ -	$ 4,330
Consultant Oversight of Field Testing Strategy 1 and 2	2	4	4	20	30	$ 4,930	$ 2,800	$ 7,730
Consultant to Prepare Summary Report for Strategy 1 followed by Strategy 2	2	6	14	24	46	$ 7,720	$ 50	$ 7,770
Contractor Costs for Strategy 1 followed by Strategy 2								$ 22,470
TOTAL for Strategy 1 only					*46*	*$ 7,710*	*$ 1,450*	*$ 23,380*
TOTAL for Strategy 2 only					*56*	*$ 9,270*	*$ 1,450*	*$ 30,440*
TOTAL for Strategy 1 followed by Strategy 2					*102*	*$ 16,980*	*$ 2,850*	*$ 42,300*

Assumptions: CWD staff will collect groundwater samples after 24 hours and 3 days and send to laboratory
Other direct costs include laboratory costs for water quality analyses, and travel expenses.. Table 2 provides the detailed contractor costs used in Table 1.

Table 8-6. Contractor Costs to Modify the Rob Roy #12 Well

Strategy 1 - Lower Pump			
	Qty	Rate	Cost
Mobilize to Site	1	$ 250.00	$ 250.00
8-inch black threaded and coupled pipe	260	$ 26.00	$ 6,760.00
8-inch ductile iron down well check valve	1	$ 1,800.00	$ 1,800.00
0/000 flat jacketed submersible cable	265	$ 14.00	$ 3,710.00
Splice kit and banding materials	1	$ 200.00	$ 200.00
Labor to lower pump 260 feet to 480 feet	1	$ 1,500.00	$ 1,500.00
Total			$ 14,220.00

Strategy 2 - Inflatable Packer and Lower Pump			
	Qty	Rate	Cost
Mobilize to Site	1	$ 250.00	$ 250.00
Pull pump, install packer, and reinstall pump at 480 feet	1	$ 2,000.00	$ 2,000.00
8-inch black threaded and coupled pipe	260	$ 26.00	$ 6,760.00
8-inch ductile iron down well check valve	1	$ 1,800.00	$ 1,800.00
0/000 flat jacketed submersible cable	265	$ 14.00	$ 3,710.00
Splice kit and banding materials	1	$ 200.00	$ 200.00
Cost of new packer	1	$ 5,000.00	$ 5,000.00
Total			$ 19,720.00

Strategy 1 followed by Strategy 2			
	Qty	Rate	Cost
Mobilize to Site	2	$ 250.00	$ 500.00
8-inch black threaded and coupled pipe	260	$ 26.00	$ 6,760.00
8-inch ductile iron down well check valve	1	$ 1,800.00	$ 1,800.00
0/000 flat jacketed submersible cable	265	$ 14.00	$ 3,710.00
Splice kit and banding materials	1	$ 200.00	$ 200.00
Labor to lower pump 260 feet to 480 feet	1	$ 1,500.00	$ 1,500.00
Pull pump, install packer, and reinstall pump back to 480 fee	1	$ 3,000.00	$ 3,000.00
Cost of new packer	1	$ 5,000.00	$ 5,000.00
Total			$ 22,470.00

SECTION 9
EVALUATE TYPE AND SITING OF A WATER TREATMENT PLANT (TASK 4.4)

This section provides the conceptual design and cost estimates of water treatment for the Cox well field developed by Kennedy/Jenks Consultants. As evaluated in Section 7 (Task 4.5), the groundwater management strategy to shift pumping from CWD's Rob Roy well field in the Aromas Red Sands to its Cox well field in the Purisima Formation will improve basin management.. However, the Cox wells produce water high in iron and manganese that will require treatment to meet drinking water standards. The section documents the evaluation of treatment alternatives and describes conceptual design drawings provided in Appendix F. This section was provided as an administrative draft technical memorandum to the Technical Advisory Committee (TAC) for review on January 31, 2013.

9.1 EVALUATION APPROACH

Kennedy/Jenks prepared the Basis of Design (BOD) Technical Memorandum (2012) that provided preliminary criteria for the layout and design of an iron and manganese groundwater treatment plant for the Cox well field to guide the development of the Conceptual Design. This Conceptual Design Technical Memorandum addresses the BOD criteria to identify the preferred treatment alternative, to provide project cost estimates, and to prepare conceptual design drawings package. For this analysis, three iron and manganese treatment alternatives were evaluated for the Cox well field.

This conceptual design analysis assumes one (1) new well and rehabilitation of one existing well at the Cox Well Field site will provide a reliable water supply with the new well as lead and the existing well as backup. The two wells will be connected to a water treatment plant to remove iron and manganese to provide a reliable water supply to CWD's customers. The water treatment plant will include a building to house the well equipment, electrical equipment, chemical systems and control room. The building will also include area for light equipment maintenance within the control room.

The Cox well field and the proposed new well are located on the same parcel as the CWD office. Two additional wells are located on site including Well 2 and Well 3. Well

2 and 3 will be abandoned and destroyed in accordance with Department of Water Resources Bulletin 74-81 and 74-90. The existing Cox Well #5 will be used as a backup well. Rehabilitation work is included in the conceptual design and cost estimate for the Cox Well #5. Rehabilitation is limited to downhole swabbing and cleaning, and installation of new wellhead equipment to meet the new hydraulic requirements.

9.2 WATER TREATMENT SYSTEM DESCRIPTION

The proposed water treatment system was identified in the BOD. The major elements included in this conceptual design addressed by the BOD include the following:

- A new groundwater production well in the vicinity of CWD office.
- A water treatment facility to remove iron and manganese in the groundwater in the vicinity of CWD office. (The treatment site by the CWD office is selected based on the existing topographic and provides efficient access for equipment and CWD's operation and maintenance staff)
- A well building to enclose and protect well equipment, electrical panels and controls, chemical feed systems and a treatment plant control room with lab sink.
- Backwash water recycling system including holding tank and pumping station.
- Site improvements including site grading, paving, fencing, infiltration system (for sink and floor drains) and pump to waste holding pond.
- Abandonment of two existing wells.
- Rehabilitation of Well 5.

The water treatment system will be designed to remove iron and manganese to concentrations below DPH drinking water standards. Oxidation/adsorption/ filtration is the most commonly used technology for iron and manganese removal from drinking water.

A general description of the treatment process for the proposed water treatment plant is for groundwater to be pumped from the well and through the pressure filters to remove iron and manganese. Potassium permanganate is injected into the well water pipeline to oxidize the iron and manganese prior to filtration. The treated water is discharged from the filters under pressure to a pipeline conveying to the existing 50,000 gallon tank. Chlorine is injected into the treated water pipeline for disinfection prior to leaving the water treatment plant site.

The filters are cleaned by backwashing and the treated water from other filters is used for backwash. Backwash is initiated by differential pressure across the filter or filter run time. Spent backwash water is discharged to the backwash tank. Backwash water is recycled through solids settling and decanting the clarified water back to the inlet of the treatment plant. Decanted water is metered at a maximum rate of ten percent of the plant influent flow rate. Solids are periodically withdrawn from the backwash tank by a vactor truck and disposed of off-site.

9.3 Water Treatment System Alternatives

Three water treatment system alternatives are developed based on the information collected from the BOD and a review of existing site conditions. The descriptions of the three treatment alternatives are presented below and the schematic flow diagrams are shown on Appendix E Drawing Sheet G2, G3, and G4. Each treatment process has been designed to accept 110% of the design well production to allow for backwash reclamation during peak production periods. Backwash reclamation is limited to 10% of the treatment raw water feed rate.

9.3.1 Alternative No. 1 – Iron and Manganese Treatment Plant with Horizontal Filter Vessel

This treatment system alternative will consist of a single horizontal pressure filter vessel with three cells. The vessel will be 7 feet diameter by 12 feet straight shell and made of welded steel construction. The pressure vessel will be filled with 24-inch depth of manganese greensand and 12-inch depth of anthracite for the removal of oxidized iron and manganese from the water. Each filter cell will operate in parallel with each cell processing one third of the supply volume under normal operation. Each cell will be sized for 200 gpm. Each cell will be cleaned by backwashing one at a time until all three cells are cleaned. Backwash supply water for a single cell will be drawn from other two cells. Backwash water will be captured, settled and reclaimed with periodic disposal of accumulated solids by pumper truck. The layout drawings for this alternative are provided in Appendix F Drawing Sheet C2 and Sheet C5.

9.3.2 Alternative No. 2 – Iron and Manganese Treatment Plant with Vertical Filter Vessels

This treatment system alternative will consist of three vertical pressure filter vessels. Each filter vessel will be sized for 200 gpm. The vessel will be 6 feet diameter by 6 feet straight shell and made of welded steel construction. Each pressure vessel will be filled with 24-inch depth of manganese greensand and 12-inch depth of anthracite for the removal of oxidized iron and manganese from the water. Backwash supply water for a single filter will be drawn from other two filters. Backwash water will be captured, settled and reclaimed with periodic disposal of accumulated solids by pumper truck. The layout drawings for this alternative are provided in Appendix F Drawing Sheet C3 and Sheet C6.

9.3.3 ALTERNATIVE NO. 3 – PACKAGE IRON AND MANGANESE TREATMENT PLANT

This alternative will consist of a package water treatment system in a skid mounted installation. The package plant will have three vertical pressure filter vessels with filled with manganese greensand and anthracite similar to Alternative No. 2. The filter piping and valves will be assembled with the filters. Each filter will be sized for 200 gpm. Backwash supply water for a single filter will be drawn from other two filters. Backwash water will be captured, settled and reclaimed with periodic disposal of accumulated solids by pumper truck. The layout drawings for this alternative are provided in Appendix F Drawing Sheet C4 and Sheet C7.

9.4 ALTERNATIVE COST COMPARISON

The alternatives are evaluated considering construction and operational and maintenance cost requirements. The conceptual level estimates of the probable construction costs for the three alternatives are presented in Table 9-1. The construction cost estimates are prepared based on the conceptual process description above, equipment manufacturer's budgetary prices, unit costs from standard estimating tools, and previous similar projects. The construction cost estimates include materials and installation costs, taxes, mobilization, bonding and insurance, contractor's overhead and profit and a 30% contingency, given the conceptual level of the design. The cost of constructing the new well is not included in this construction cost estimate. The new well construction cost estimate is provided in Section 8 (Task 4.3).

Table 9-1. Comparison of Conceptual Construction Costs

Item	Item Description	Horizontal Filter	Vertical Filters	Package Plant
1	Mobilization/Insurance/Bonds	$45,000	$45,000	$45,000
2	Site Work	$78,000	$78,000	$78,000
3	Concrete	$20,000	$20,000	$20,000
4	Yard Piping	$50,000	$50,000	$50,000
5	Chemical Feed System	$60,000	$60,000	$60,000
6	Facility Building	$195,000	$195,000	$195,000
7	Pressure Filters	$327,000	$350,000	$310,000
8	Backwash Storage Tank	$42,000	$42,000	$42,000
9	Backwash Recovery Pump Station	$10,000	$10,000	$10,000
10	Well Pump and Motor	$92,000	$92,000	$92,000
11	Rehab (E) Cox Well #5	$25,000	$25,000	$25,000
12	(N) Cox Well # 5 Pump & Pipeline	$104,000	$104,000	$104,000
13	Waste Pond	$15,000	$15,000	$15,000
14	Electrical	$153,000	$157,000	$151,000
15	SCADA	$77,000	$79,000	$76,000
16	Portable Generator	$10,000	$10,000	$10,000
17	Startup, Testing, and Training	$30,000	$30,000	$30,000
	Subtotal	$1,333,000	$1,362,000	$1,313,000
	Contingency 30%	$400,000	$409,000	$394,000
	Subtotal	$1,733,000	$1,771,000	$1,707,000
	Engineering, Administrative and Legal (15%)	$260,000	$265,650	$256,060
	Total Estimated Construction Cost	**$1,993,000**	**$2,036,650**	**$1,963,050**

9.4.1 OPERATION AND MAINTENANCE COSTS

The estimated operation and maintenance (O&M) costs for the three treatment alternatives are presented in *Table* 9-2. The O&M costs include chemical use, energy use, sludge disposal, filter media replacement, maintenance materials and labor to perform the O&M tasks. The cost for periodic well maintenance is not included in this O&M costs. Typical well maintenance costs recur every 15 to 20 years with inspection, swabbing and bailing of the well followed by super chlorination and pump service. This cost could range from $25,000 to $50,000 depending on the condition of the equipment and the well.

Table 9-2. Estimated Annual O & M Costs

Item	Item Description	Horizontal Filter	Vertical Filters	Package Plant
1	Chemical	$24,000	$24,000	$24,000
2	Energy	$49,000	$49,000	$49,000
3	Sludge Disposal	$8,000	$8,000	$8,000
4	Filter Media Replacement/Replenishment	$2,000	$2,000	$2,000
5	Maintenance Materials	$3,000	$3,000	$3,000
6	Labor	$54,000	$54,000	$54,000
	Total Estimated Annual O & M Cost	**$140,000**	**$140,000**	**$140,000**

The maintenance materials costs are estimated at 2% of the capital cost of electrical and mechanical equipment such as pumps, valves, and instrumentation that require preventive maintenance. The costs of replacing the filter media are covered separately.

The main energy usage for the alternatives comes from pumping and includes a well pump, a backwash pump, and chemical feed pumps. The energy cost is based on a rate of $0.12 per kilowatt-hour. Chemical usage for the alternatives includes addition of potassium permanganate, sodium hypochlorite, and polymer on as-needed basis. Filter media replacement is expected every 15 years and the filter media replenishment is included annually.

9.4.2 ALTERNATIVE EVALUATION

A list of evaluation criteria is developed to facilitate an alternative comparison. The evaluation criteria used to determine the recommended preferred alternative is summarize below:

- Constructability – determine the ease of construction and spacing requirements for the water treatment system.
- Ease of Operation – determine the ease of operating and maintaining the water treatment plant based on labor requirements for periodic maintenance, cleaning, and repairs.
- Reliability – determine the reliability of the water treatment system based on the operational information collected from other sites for iron and manganese treatment and support from the filter manufacturers.
- Equipment Access – determine the ease of access to the well pump, chemical feed system and filter equipment for routine servicing and maintenance. Ease of access to the interior of the filters for repair is also considered.
- Filter Vessels Fabrication and Availability – determine the availability of types of filter vessels and time requirements for fabrication and delivery.
- Construction Costs – provide a score for ranking the three treatment alternatives based on the construction costs summary table presented above.
- O&M Costs – provide a score for ranking the three treatment alternatives based on the annual O&M costs summary table presented above.

Each criterion is assigned a score from 1 to 3, with 1 being the best. The ranking for the three treatment alternatives is then developed. An evaluation of the alternatives is summarized in Table 9-3 below. The total score is the sum of each scored evaluation criteria.

Table 9-3. Comparison of Alternatives and Overall Ranking

Factors	Horizontal Filter	Vertical Filters	Package Plant
Constructability	2	2	1
Ease of Operation	1	1	1
Reliability	2	1	1
Equipment Access	2	1	1
Filter Vessels Fabrication and Availability	2	1	1
Construction Cost	2	3	1
O&M Costs	1	1	1
Total Score	12	10	7
Overall Ranking	**3**	**2**	**1**

Note: The energy use and waste production for three alternatives are not shown on the table, but the energy cost and waste handling cost are included in the O&M costs. The energy use and waste production are approximately the same for all three alternatives.

9.5 Recommended Alternative

Based on the overall ranking of the alternatives, the package iron and manganese treatment plant is recommended. This alternative is a prepackaged system and relatively easy to install and operate. The filter vessels and piping / valving are assembled in the shop. Construction with package plant normally has lower installation cost. The package plants are common used for water treatment plants with low flow capacity. There are currently many existing package plants for iron and manganese treatment.

Horizontal filter vessels are common used for treatment plants with higher flow capacity. Most of the filter manufacturers have used vertical filter vessels or package plant with similar flow capacity as the proposed CWD water treatment plant.

The conceptual level estimate of the probable construction cost for the package iron and manganese plant is presented in Table 9-4. The schematic flow and layout drawings for the recommended alternative are provided in Appendix F Drawing Sheets G4, C4 and C7.

Table 9-4. Construction Cost

Item	Item Description	Cost
1	Mobilization/Bonds/Insurance	$45,000
2	Site Work	$78,000
3	Concrete	$20,000
4	Yard Piping	$50,000
5	Chemical Feed System	$60,000
6	Facility Building	$195,000
7	Pressure Filters	$310,000
8	Backwash Storage Tank	$42,000
9	Backwash Recovery Pump Station	$10,000
10	Well Pump and Motor	$92,000
11	Rehab (E) Cox Well #5	$25,000
12	(N) Cox Well #5 Pump & Pipeline	$104,000
13	Waste Pond	$15,000
14	Electrical	$151,000
15	Instrumentation & Controls	$76,000
16	Portable Generator	$10,000
17	Startup, Testing, and Training	$30,000
	Subtotal	$1,313,000
	Contingency 30%	$394,000
	Subtotal	$1,707,000
	Engineering, Administrative and Legal	$256,050
	Total Conceptual Construction Cost	**$1,963,050**

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SECTION 10
SUMMARY OF CHROMIUM VI TREATMENT TECHNOLOGIES (TASK 4.4*)

Kennedy/Jenks Consultants conducted a review of chromium VI treatment technologies as part of additional scope to grant Task 4.4. This report section was provided to the Technical Advisory Committee (TAC) for review as a memorandum on June 17, 2013.

10.1 EXISTING AND UPCOMING REGULATIONS

Chromium VI is a heavy metal that is naturally occurring in groundwater, but may also occur due to industrial contamination. In water, it exists either in its more reduced form, trivalent chromium (Chromium (III)) or its more oxidized form, hexavalent chromium (chromium VI)). Chromium (III) is an essential nutrient; however, Chromium VI may pose a potential public health risk, even when present at low levels.

Potential carcinogenic risks resulting from inhalation of chromium VI have long been recognized, but a drinking water regulation for chromium VI has not yet been promulgated. Instead, the present drinking water regulations are based on total chromium, with the California MCL set at 0.05 mg/L) or 50 μg/L and the United States Environmental Protection Agency (USEPA) MCL set at 0.10 mg/L (100 μg/L). In 2011, the California Office of Environmental Health Hazard Assessment (OEHHA) established a public health goal (PHG) for chromium VI at 0.02 μg/L, which triggered a requirement for the California Department of Public Health (DPH) to set a MCL for chromium VI. DPH proposed the draft MCL of 10 μg/L in August 2013 with promulgation of a final MCL anticipated in 2014. Additionally, the USEPA is reviewing toxicity data to determine potential carcinogenicity of chromium VI in drinking water, and depending on the results of that review, may propose establishing a drinking water standard sometime in the future.

10.2 EXISTING TREATMENT TECHNOLOGIES

Title 22, Chapter 15 of the California Code of Regulations designates four (4) technologies as Best Available Technologies (BAT) for the removal of total chromium to below 0.05 μg/L; namely, coagulation/filtration, ion exchange, lime softening (chromium III only), and reverse osmosis. However, these technologies have not been approved for removal of chromium VI to the low levels expected to be mandated under a new MCL (expected to be between 1 to 25 μg/L).

There have only been a few pilot and demonstration level treatment plants for the removal of hexavalent chromium at these low levels. The technologies used at pilot and demonstration plants include:

- Four mature treatment technologies that are likely to be designated as BAT for chromium VI:
- Strong Base Anion Exchange (SBA),
- High-Pressure Membrane,
- Weak Base Anion Exchange (WBA),
- Reduction, Coagulation, Filtration (RCF)
- Two emerging technologies that are currently undergoing research and testing:
- Biological reduction, filtration
- Chemical Reductive Media (CRM)

Each of these treatment technologies is briefly discussed below with inclusion of key considerations for each treatment technology such as footprint and residuals disposal as well as some cost comparison information where available. These following summaries are based on the review of reports from the Water Research Foundation (McNeill et. al. 2012), Soquel Creek Water District (Jacobs Engineering, 2011) and presentations from the Water Research Foundation Hexavalent Chromium Workshop 2013 (Blute and Wu, 2013; Drago, 2013, and Najm, 2013).

10.3 Mature Technologies

The following four technologies are considered mature technologies with possible BAT designation.

10.3.1 STRONG BASE ANION EXCHANGE

The SBA exchange system requires a relatively simple treatment train (see Figure 10-1) which includes:

1. Bag Filters – Removes large particles prior to the resin vessels.
2. Resin Vessels – Remove chromium VI by replacement of chloride with chromium VI on the resin bed.
3. Brine (sodium chloride) – Used to regenerate the resin vessels with chloride and remove the chromium VI in the spent brine.

The major consideration with this system is the ability to dispose of the spent brine inexpensively as well as the water quality of the source water. The frequency of regeneration required can largely vary due to the presence of other competing ions such as sulfate and nitrate.

The SBA exchange system was pilot tested in the spring of 2013 at SqCWD's San Andreas well as part of a Water Research Foundation project (Jacobs Engineering, 2011, 2013). The preliminary results of the SBA pilot testing were promising with breakthrough occurring after 15,000 to 30,000 bed volumes (BVs). The performance of the SBA exchange system was better than anticipated with higher water efficiency resulting in lower volumes of brine waste disposal thus reducing the overall operational cost (Jacobs Engineering, 2013).

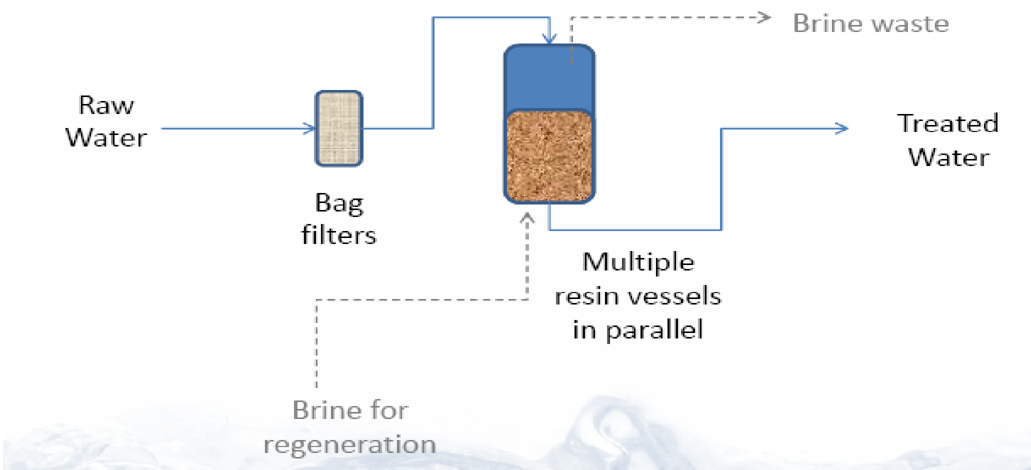

Figure 10-1: Strong Base Anion Exchange

Source: Blute, Nicole and Wu, Xueying. "Chromium Treatment Studies at Glendale, Residuals, and Treatment Testing Guidelines Presentation" Water Research Foundation Chromium (VI) Workshop, February 4, 2013.

10.3.2 High-Pressure Membrane System

The use of a high-pressure membrane (reverse osmosis or nanofiltration) system has been shown to be effective in the removal of chromium VI from systems. Key considerations to this treatment include high capital cost, high energy cost, brine (reject) disposal and large water loss. A waste stream of 15-20% of your influent water can be expected with a high-pressure membrane system. Because of the high water loss, high-pressure membrane systems are unlikely to be selected for chromium VI removal unless desalination is the primary objective. High-pressure membranes are not considered further in this section.

10.3.3 Weak Base Anion Exchange

The weak base anion exchange system requires a treatment train as shown in Figure 10-2. This includes:

1. pH Adjustment – Addition of either carbon dioxide (CO_2) or acid to pH 6.0 for optimal removal of chromium VI by the resin. Large amounts of CO_2 or acid may be required if the alkalinity of the source water is high. In addition, pH adjustment to 6.0 may be difficult due to the lack of natural buffering in this pH range.

2. Bag Filters – Removes large particles prior to the resin vessels.
3. Weak Base Anion Exchange Resin Vessels – The weak base anion exchange resin exchanges chloride for chromium VI and then converts chromium VI to chromium III, which binds to the column.
4. pH Adjustment – Aeration or caustic addition to increase the pH prior to distribution to the system.

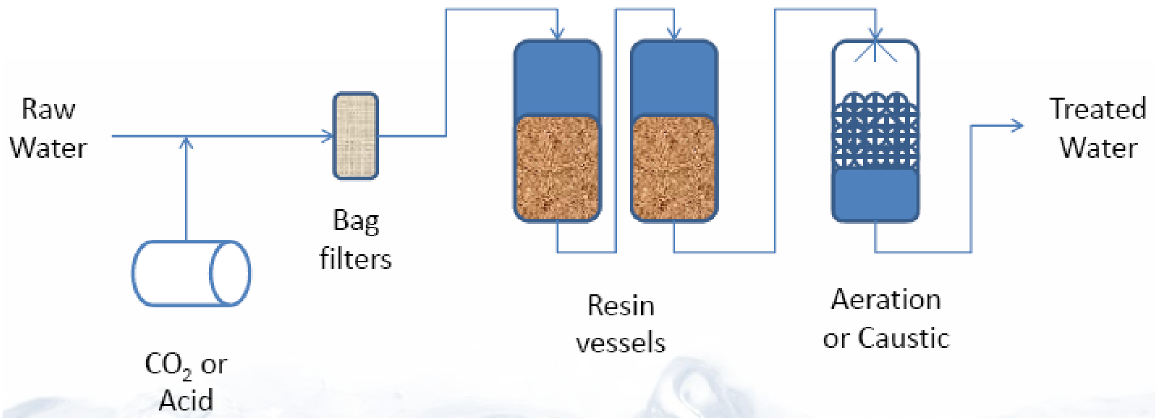

Figure 10-2: Weak Base Anion Exchange

Source: Blute, Nicole and Wu, Xueying. "Chromium Treatment Studies at Glendale, Residuals, and Treatment Testing Guidelines Presentation" Water Research Foundation Chromium (VI) Workshop, February 4, 2013.

Key considerations for use of this treatment technology include pH adjustment, residual disposal and pre-conditioning of the resin bed. Chromium, copper, vanadium and uranium have a high affinity for the column and based on the Glendale work are likely to require the resin to be disposed of as a California Hazardous waste due to high concentrations of chromium, copper and vanadium as well as be considered a Technologically-Enhanced, Naturally-Occurring Radioactive Material (TENORM) waste or source material if sufficient uranium is present in the source water. As well, it was found that pre-conditioning of the resin may be required due to formaldehyde release during the early stages of column use.

10.3.4 Reduction, Coagulation, Filtration

The reduction, coagulation, filtration system requires a more complicated treatment train as shown in Figure 10-3. This includes:

1. Ferrous (Fe[II]) Sulfate Addition – Ferrous sulfate is used to reduce chromium VI to chromium III through an oxidation-reduction reaction of Fe(II) to Fe(III). Required concentrations of iron are expected to be between 50:1 and 75:1 Fe:Cr(VI).
2. Reduction Vessel – The reduction of chromium VI to chromium III by Fe(II) requires time. Testing conducted at Glendale indicate that a residence time between 15-45 minutes is required.
3. Oxidation of Ferrous – Addition of chlorine is recommended to oxidize any remaining Fe(II). Initial studies at Glendale did not indicate any re-oxidation of chromium III during this process.
4. Filtration – This step uses either a gravity filter or microfiltration to remove the chromium III from the system. Addition of a polymer may be necessary for total removal of chromium III. During the Glendale studies, gravity filtration was shown to remove total chromium to levels of 1 to 5 µg/L whereas microfiltration removed down to non-detect levels of total chromium.

Key considerations with this treatment system include a large footprint, complicated system, and the ability to dispose of residuals through an existing sewer system. The RCF system requires residuals treatment through solids thickening and dewatering if disposal through an existing sewer system is not possible so that the spent backwash water can be recycled. The ACWA Residuals Study showed that the residuals may be considered a California Hazardous Waste if dried prior to disposal, rather than discharged to the sewer as a liquid waste.

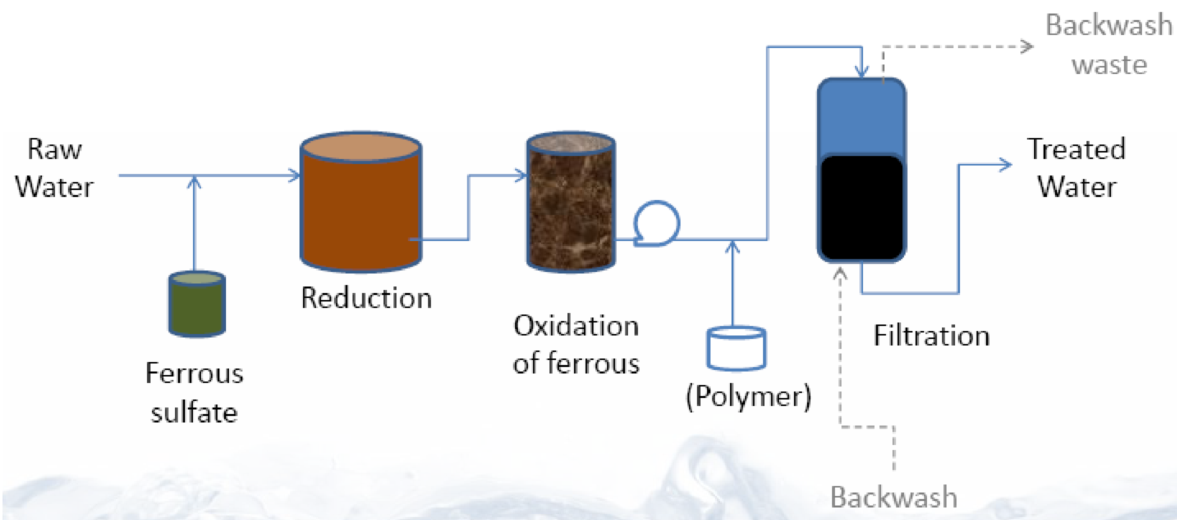

Figure 10-3: Reduction, Coagulation, Filtration

Source: Blute, Nicole and Wu, Xueying. "Chromium Treatment Studies at Glendale, Residuals, and Treatment Testing Guidelines Presentation" Water Research Foundation Chromium (VI) Workshop, February 4, 2013.

10.4 EMERGING TECHNOLOGIES

The following technologies are currently undergoing research and testing.

10.4.1 BIOLOGICAL REDUCTION, FILTRATION

Biological reduction filtration requires a somewhat complicated treatment train, but has the added capacity for co-removal (or reduction) of other contaminants (namely, nitrate, selenium, and perchlorate. This system requires:

1. Acetic Acid & Phosphoric Acid Addition – Acetic acid is added as an electron donor and phosphoric acid is added as a "nutrient" to promote biological growth within the system.
2. Fluidized Bed Reactor – The reactor promotes growth of microbes within the system that reduce nitrate, chromium, selenium and perchlorate if present. Fixed bed reactors are also being studied.
3. Aeration – Required in California due to the anoxic conditions created in the fluidized bed reactor.
4. Filtration/Disinfection – Required in California due to drinking water regulations. The filtration will remove total chromium, but may require addition of a polymer for removal down to low levels of total chromium. Chlorine would be used for disinfection.

Key considerations with this method include a larger footprint and complicated system. The cost and footprint requirements for a FBR system are currently being researched by Kennedy/Jenks and Envirogen at a pilot study being conducted at the City of Davis.

10.4.2 CHEMICAL REDUCTIVE MEDIA

Iron based reductive media have been shown to remove hexavalent chromium in drinking water at bench and - pilot-scale applications. While the exact removal mechanism is uncertain, it is thought to be a combination of reduction, adsorption and precipitation/filtration of the hexavalent chromium (Jacobs Engineering, 2011). Figure 4 is a process schematic of the SMI-III Chemical Reduction Media technology from Jacobs Engineering (2011).

The CRM process would include pre-filtration, reductive media in pressure vessels, and, most likely, post-filtration as the reactive media is consumed in the process and residual iron must be removed. The process may also require pH adjustment and residuals processing.

Iron-based chemical reductive media was to be pilot tested in the Spring of 2013 at SqCWD's San Andreas well as part of a Water Research Foundation project (Jacobs Engineering, 2011, 2013). However, the strong showing of the SBA exchange system pilot study has made the CRM approach no longer cost-competitive. Therefore, the recommendation was to eliminate further CRM pilot-scale testing in order to support additional testing of the SBA exchange system (Jacobs Engineering, 2013).

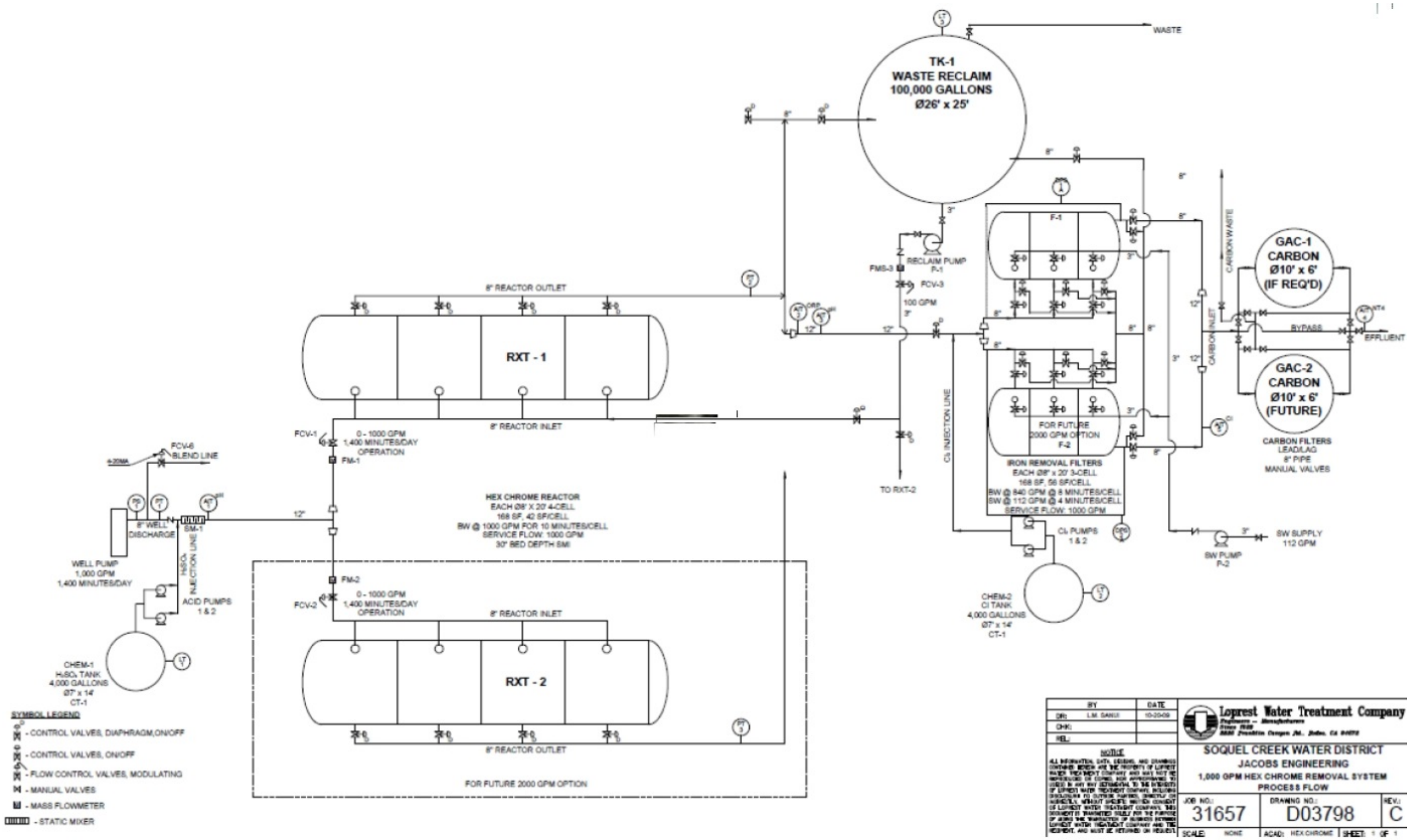

Figure 4: Chemical Reductive Media,

Source: Jacobs Engineering, 2011.

10.5 COST OF TREATMENT

Cost information has been complied for three of the four mature chromium VI treatment technologies under a WITAF/Water RF Project that compares RCF, WBA and SBA. Figure 10-4 shows the range of values that may apply for each treatment system. This range was created using two scenarios.

Scenario 1 - Discharge of residual streams to onsite sanitary sewer for all but the WBA, for which the spent resin was disposed of at a municipal landfill.

Scenario 2 - Assumed that the residuals needed more expensive forms of disposal.

These ranges show that the specific treatment option that is optimal for a system will greatly depend on a number of factors. Costs for Chemical Reductive Media and Biological Reduction, Filtration are still being developed.

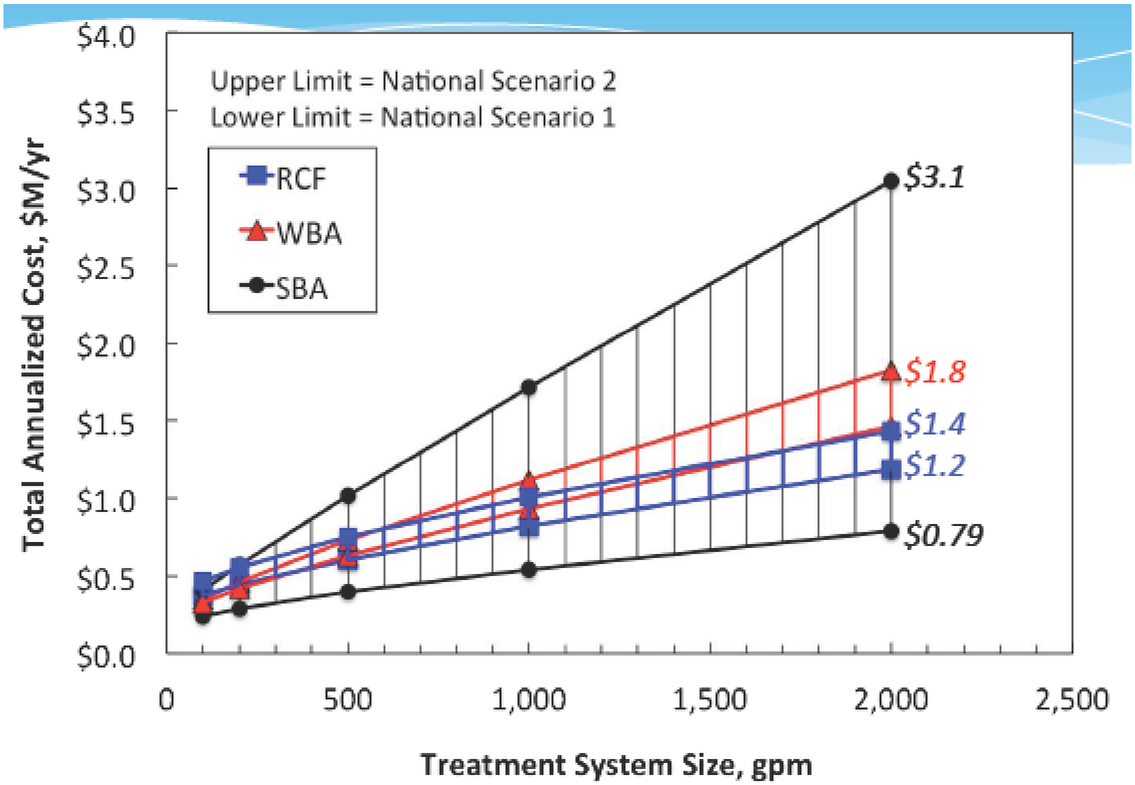

Figure 10-4: Total Annualized Cost of Treatment

Source: Najm, Issam. "Practical & Economic Feasibility of Implementing Chromium VI Treatment." Water Research Foundation Chromium (VI) Workshop, February 4, 2013.

10.6 CONCLUSIONS

Several treatment options for the removal of chromium VI to low levels are potentially available. Selection of a treatment process is complicated and depends on a number of site-specific factors including water quality, well capacity, footprint limitations, and cost. All the treatment options discussed above are still being researched to develop increased knowledge regarding their efficacy as well as operational limitations of each treatment type. Currently, there are few demonstration level treatment systems for the removal of chromium VI and information at this level is needed to further develop information that is necessary for water systems to be able to determine which treatment option may be best for their water sources.

In Spring 2013, a pilot-scale testing was conducted at SqCWD's San Andreas well as part of a Water Research Foundation project (Jacobs Engineering, 2011, 2013). The SBA and CRM technologies were selected for pilot-scale testing due to the availability of a sewer at the well site to dispose of brine from the SBA process and potential cost effectiveness of the CRM process.

The preliminary results of the SBA pilot-scale test were considered highly promising (Jacobs Engineering, 2013) so that the CRM technology was no longer considered cost-competitive with SBA. Therefore, it was recommended to eliminate the CRM pilot-scale testing in order to support further testing of the SBA exchange system (Jacobs Engineering, 2013).

SECTION 11
CONCLUSION

The recommendations from this technical study are summarized as follows:

- Implement strategy of redistributing pumping from the Rob Roy well field to the Cox well field. Simulation results showed that the strategy to redistribute pumping to a new Cox well is within the sustainable yield of the Purisima Formation that supplies the Cox well field. Shifting pumping from the Aromas area to the Purisima area will also reduce system chromium VI concentrations while increasing CWD's reliability by diversifying its supply.
- Replace existing production wells at Cox well field with a single modern-designed production well. A new well would likely have a discharge capacity of 300 to 400 gpm and it was estimated that dry season production of approximately 160 acre-feet can be sustained. The top of the screen should be at least 260 feet deep and the well should be drilled 660 feet deep to screen the full depth of the Purisima F unit. The total estimated cost for constructing and developing the new well and destroying two of the existing wells are provided is approximately $700,000.
- Install a water treatment plant at the Cox well field to treat for iron and manganese. The recommended package system in a skid mounted installation is relatively easy to install and operate and is commonly used for treatment plants with low flow capacity such as provided by a new Cox well. The construction cost is estimated at approximately $2 million and the annual operations and maintenance costs are estimated at $140,000.
- Identify and pursue regional partnerships to use CWD's increased inland pumping capacity to facilitate regional basin management. Capacity in excess of CWD's demand can be used to help non-CWD pumpers reduce pumping closer to the coast to address seawater intrusion risk.
- As part of testing the newly constructed Cox well, evaluate the effect of predicted lower groundwater levels on the supply of private wells near the Cox well field.

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SECTION 12
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APPENDIX A: TASK 4.2 FIELD DATA ON COMPACT DISK

This Appendix includes all data discussed in Section 2 (Task 4.2). In addition,, other data that were collected, but not analyzed, are also included on the compact disk. These data include:

- Groundwater level data from Cox #5, Cox #2, and Cox #3 collected during the Cox #5 profile that were not analyzed to evaluate well performance or aquifer properties because the pump was stopped and re-started to accommodate installation of BESST, Inc. profiling tools;
- Laboratory reports for all water quality analyses performed for Rob Roy #12 and Cox #5 well profiles; and
- Recovery data collected during Cox #3 aquifer test.

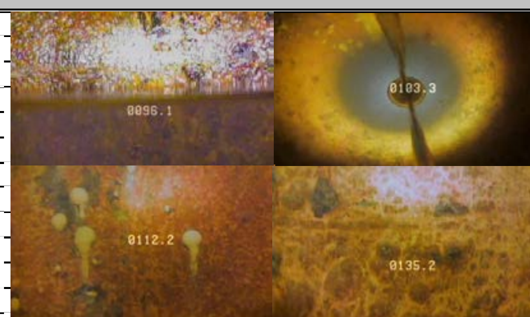
APPENDIX B: SUMMARY SHEETS OF COX #3 AND #5 VIDEOS

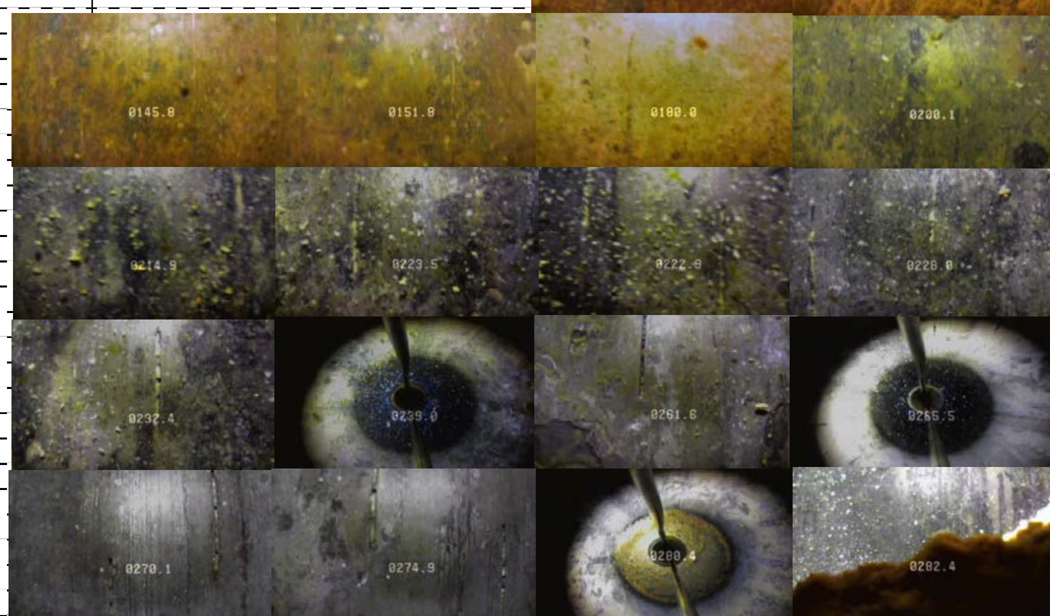
Newman Well Surveys

Video Survey Report

Company:	Central Water Dist.	**Date:**	29-May-12
Well:	Well #3	**Run No.**	One
Field:	Aptos	**Job Ticket:**	72028A
State:	California	**Total Depth:**	282.4 ft
		Fluid Level:	96.1 ft
Location:	400 Cox Rd.	**Elevation:**	290.0 ft
Zero Datum:	Top of pump pad	**Tool Zero:**	Side view lens (Add 1.5 ft. to downward view)
Reason for Survey	General inspection		

Depth	Remarks
0.0 ft	12" I.D. steel casing
96.1 ft	Water level, moderate scale and growth on casing.
135.0 ft	*Vertal milled slot perf's begin, continue to bottom*
200.0 ft	Less growth on casing to 230 ft.
230.0 ft	Light scale & growth on casing to bottom
282.4 ft	Bottom of well





Notes: No casing problems were seen. Vertical milled slot perforations were moderately plugged from 135.0 ft. to 230.0 ft. Perforations were mostly open from 230 ft. to bottom.

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Newman Well Surveys

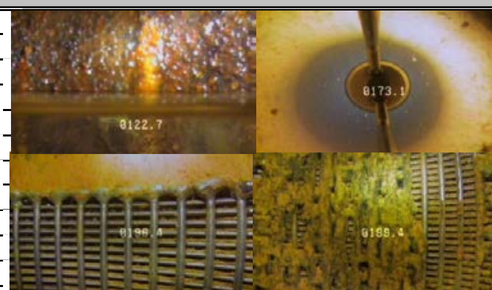
Video Survey Report

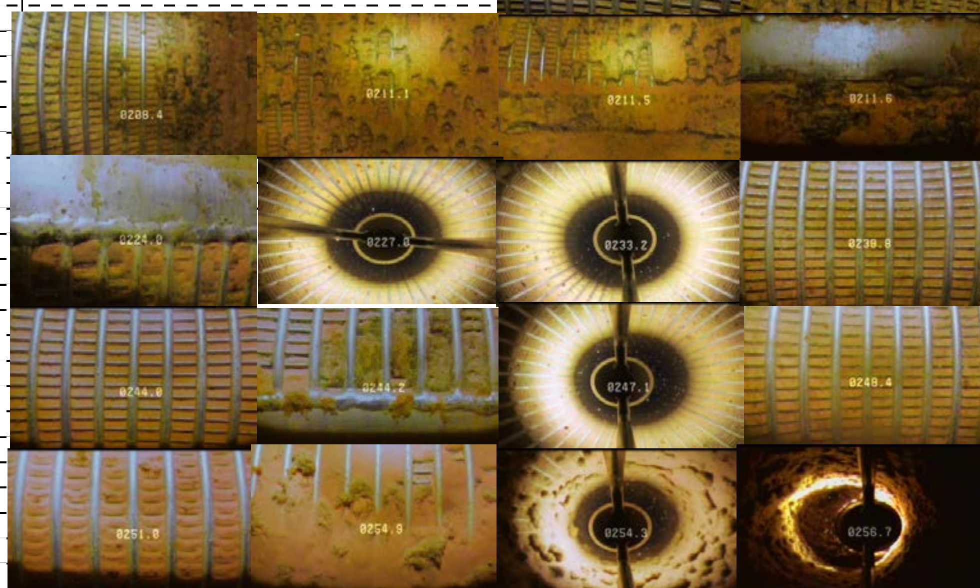
Company:	Central Water Dist.	**Date:**	29-May-12
Well:	Well #5	**Run No.**	One
Field:	Aptos	**Job Ticket:**	72028B
State:	California	**Total Depth:**	258.5 ft
		Fluid Level:	122.7 ft
Location:	400 Cox Rd.	**Elevation:**	322.0 ft

Zero Datum: Top of pump pad **Tool Zero:** Side view lens (Add 1 ft. to downward view)

Reason for Survey General inspection

Depth	Remarks
0.0 ft	12" I.D. steel casing
122.7 ft	Water level
196.4 ft	Stainless steel screen to 211.4 ft.
211.4 ft	Blank casing to 224 ft.
220.0 ft	Transition to 8" I.D. casing
224.0 ft	Stainless steel screen to bottom
258.5 ft	Bottom of well





Upper screen was mostly plugged on one side and slightly plugged on the other.
Lower screens were plugged for the first couple of ft. and the bottom couple of ft.
Otherwise, they were open.

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APPENDIX C: COX WELL DRAWINGS AND RECORDS

WELL DATA

(1) Place and Owner: CENTRAL SANTA CRUZ COUNTY WATER DISTRICT

(2) Source of Information: Carl Burow

Collected by: A. G. Hood Date: ~~March 1970~~

(3) Number or Name	#2	#3	#5
Date drilled	7-28-53	11-20-1960	1967
(4) Location: Neighborhood	Cox Road	Cox Road	Cox-Road
Size of lot	15 acre	15 acre	15 acre
Distance to: Sewer	None nearby	None nearby	None nearby
Sewage disposal	Long	Long	Long
Abandoned well	None	None	None
Nearest property line	300'	300'	300'
(5) Housing: Type	Galv. iron	None	None
Condition	Good	Good	Good
Pit depth (if any)	None	None	None
Floor (material)	Concrete	Concrete	Concrete
Drainage	Good	Good	Good
(6) Well Depth	261'	300'	352'
(7) Casing: Depth	261'	300'	352'
Diameter	12"	12"	12"
Kind	DHR 12 ga	Steel 1/4"	Steel 1/4"
Height above floor	8"	8"	8"
Distance to highest perforations	105'	172'	192'
Surface sealed (yes or no)	Yes	Yes	Yes
Gravel pack (yes or no)	No	Yes	Yes
Second casing depth	None	50'	90'
Second casing diameter	None	32"	20"
Annular seal (depth)	85'	50'	90'
(8) Impervious Strata: Penetrated — Thickness	26'	10'	42'
Depth to	235'	290'	310'
(9) Water Levels: Depth to — Surface			
Static	126'	113'	142'
When pumping	160'	215'	195
(10) Pump: Make	Peerless	Peerless	Western
Type	Centrif	Centrif	Centrif
Capacity, g.p.m.	300	300	300
Lubrication	Oil	Oil	Oil
Power	40 HP Elec.	50 HP Elec.	40 HP
Auxiliary power	Gasoline	No	No
Control	Auto	Auto	Auto
Discharge location	Above Ground	Above Ground	Above Ground
Discharge to	Tank	Tank	Tank
(11) Frequency of Use	DAILY — peaking only - ~~daily~~ - Summer	peaking only; ~~Daily~~ Summer	Peaking Only, Summer + [illegible]
(12) Flood Hazard	[illegible] capacity [illegible] none	None	None
(13) Remarks and Defects (Use other side if necessary)	None	None	None

(14) Show well log on other side.

STATE OF CALIFORNIA
DEPARTMENT OF PUBLIC HEALTH

13744-450 8-67 10M OSP

(REV. 11-51)
FORM 229.1432

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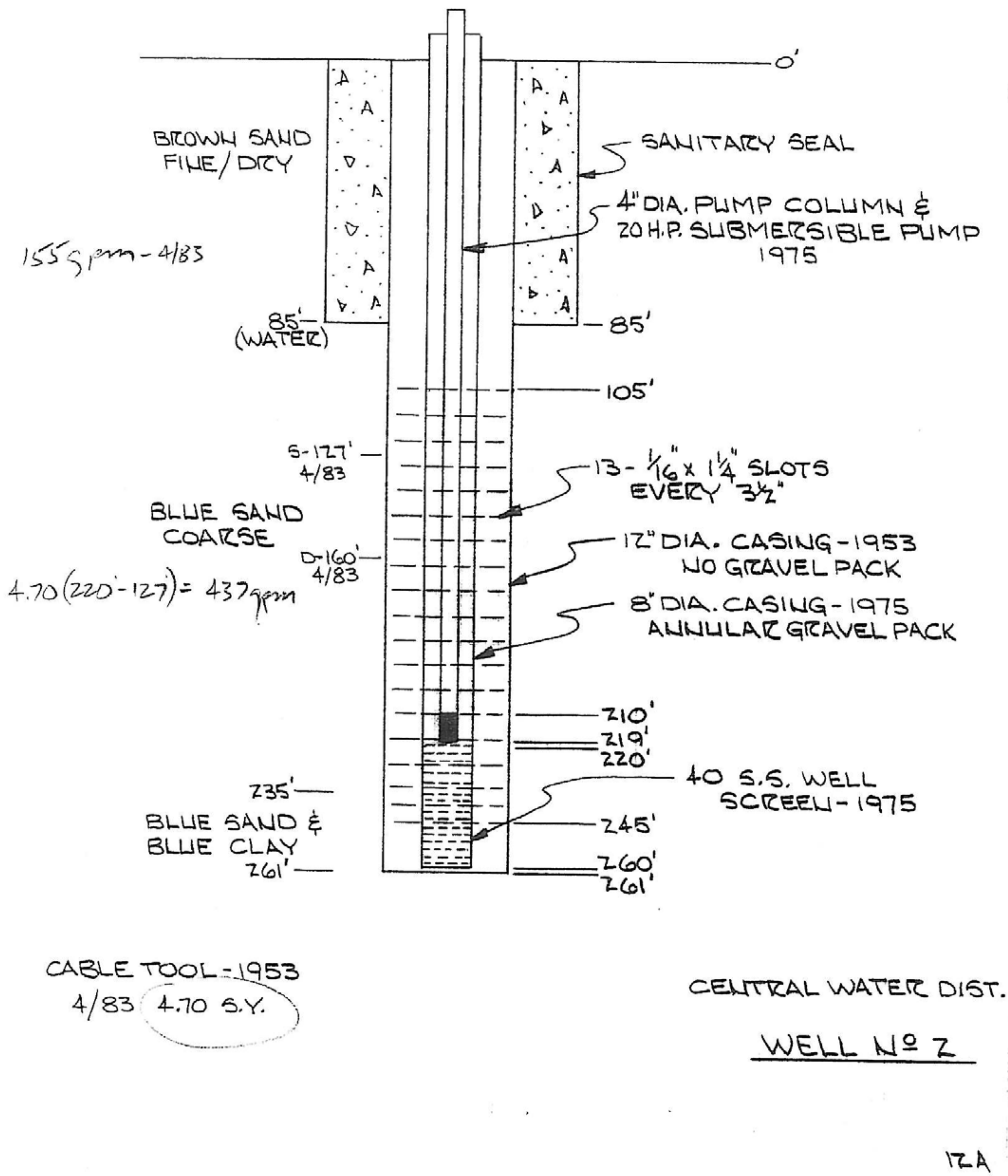

0'
BROWN SAND
FINE/DRY
SANITARY SEAL
4" DIA. PUMP COLUMN &
20 H.P. SUBMERSIBLE PUMP
1975
155 gpm - 4/83
85'
(WATER)
85'
105'
S-127'
4/83
13 - 1/16" x 1 1/4" SLOTS
EVERY 3 1/2"
BLUE SAND
COARSE
D-160'
4/83
12" DIA. CASING - 1953
NO GRAVEL PACK
4.70(220'-127') = 437 gpm
8" DIA. CASING - 1975
ANNULAR GRAVEL PACK
210'
219'
220'
235'
40 S.S. WELL
SCREEN - 1975
BLUE SAND &
BLUE CLAY
245'
261'
260'
261'
CABLE TOOL - 1953
4/83 4.70 S.Y.
CENTRAL WATER DIST.
WELL No 2
12A

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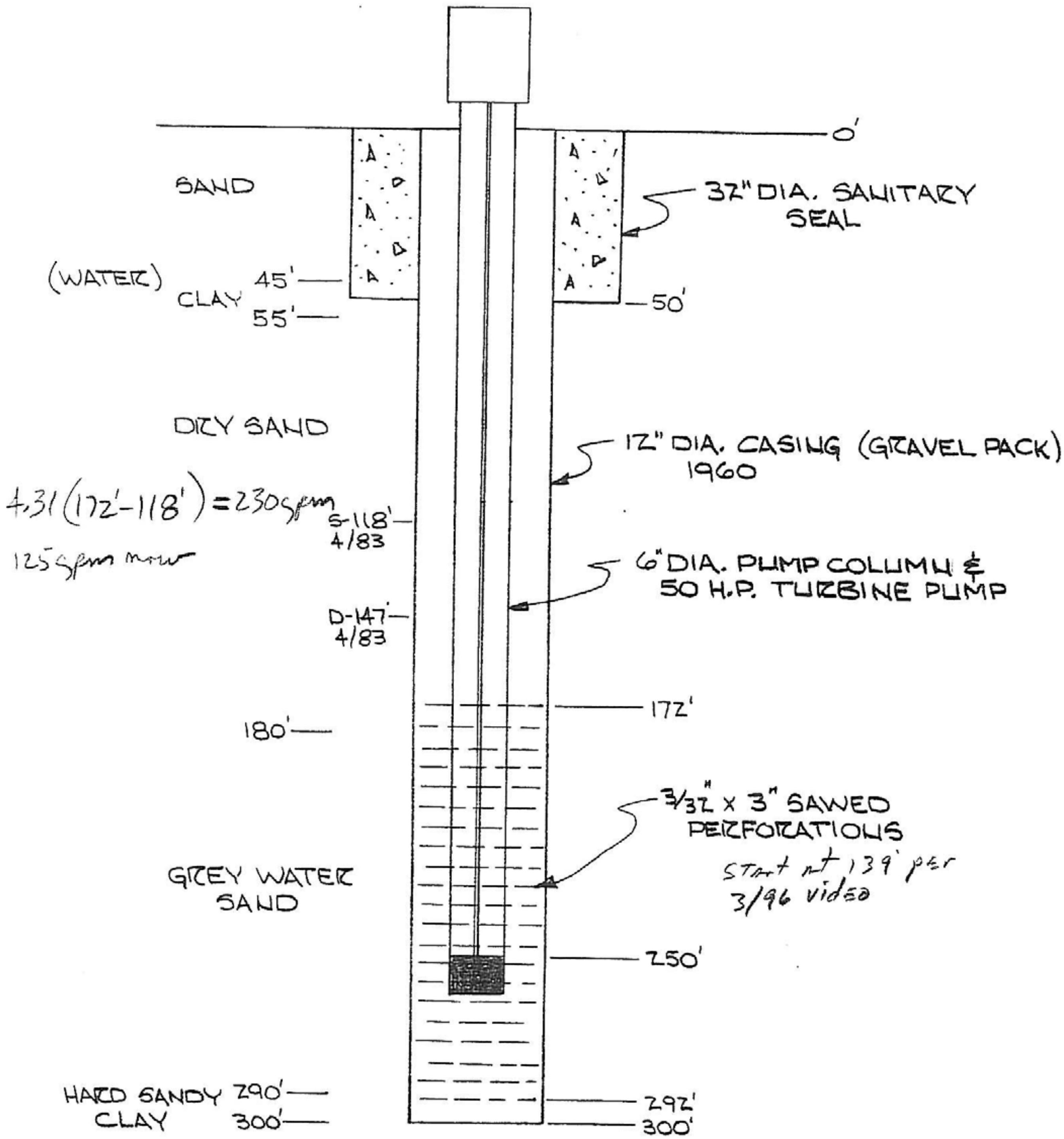

CENTRAL WATER DIST.

WELL No 3

ROTARY - 1960
4/83 (4.31 S.Y.)

13A

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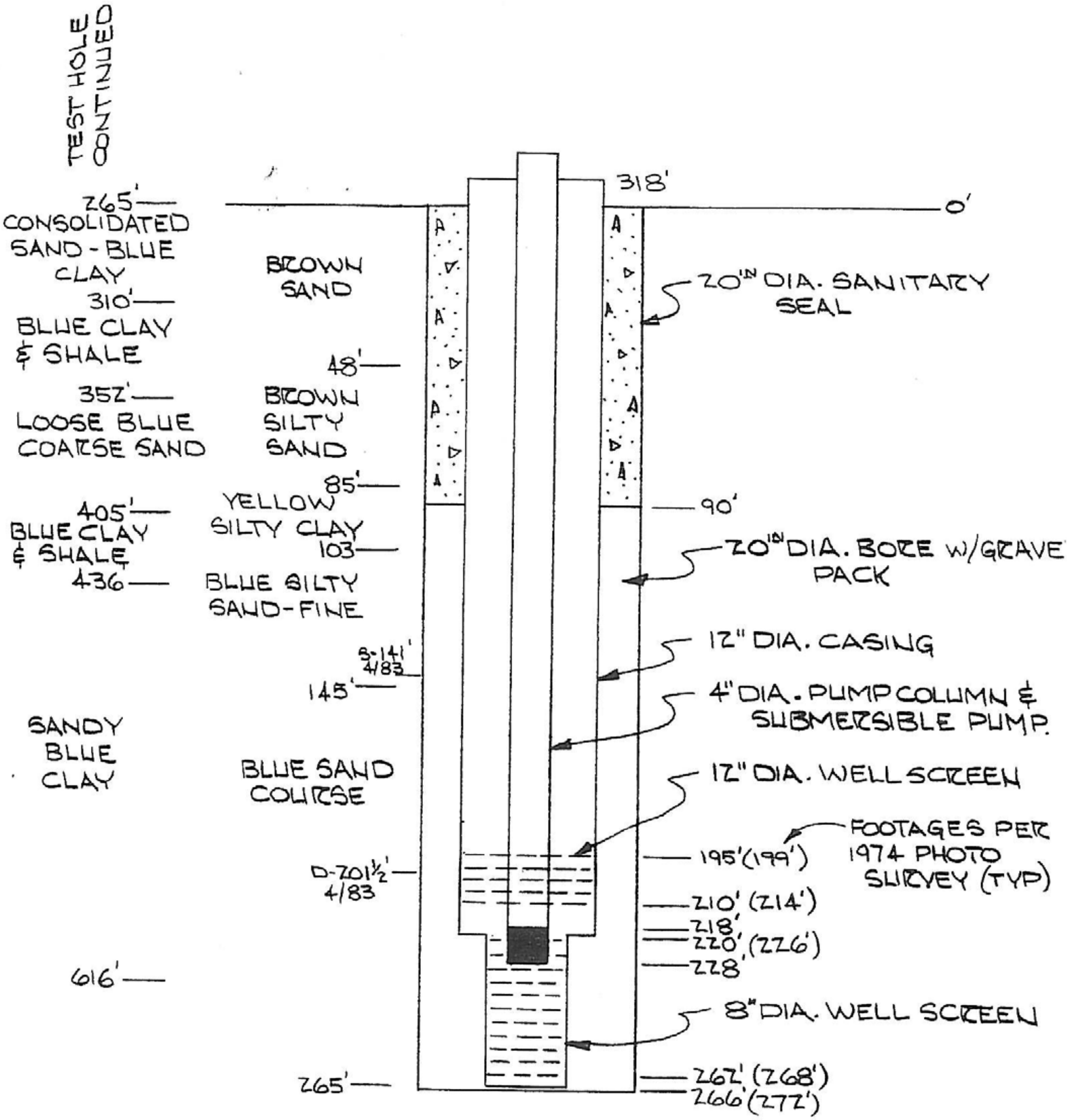

CENTRAL WATER DIST

WELL No 5

ROTARY - 1967

4/83 - 2.94 S.Y.

14A

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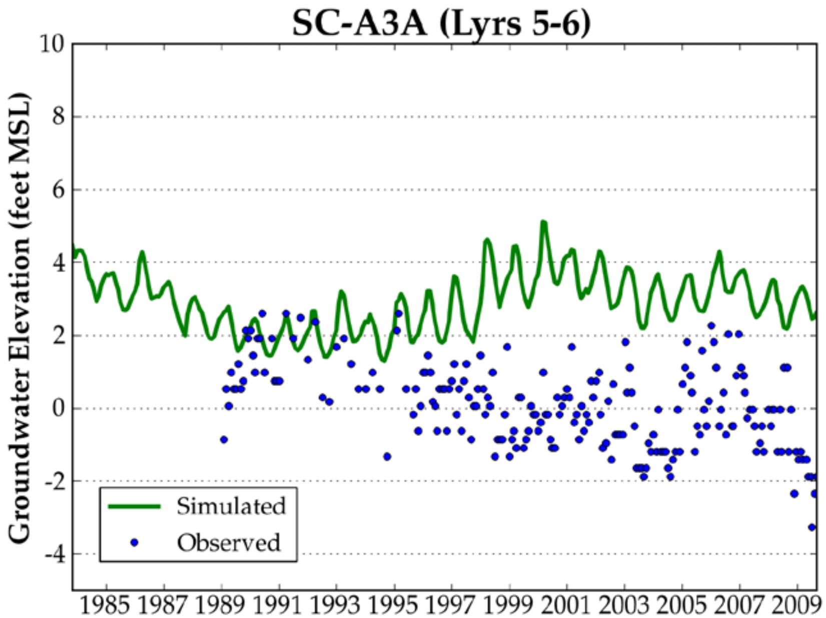
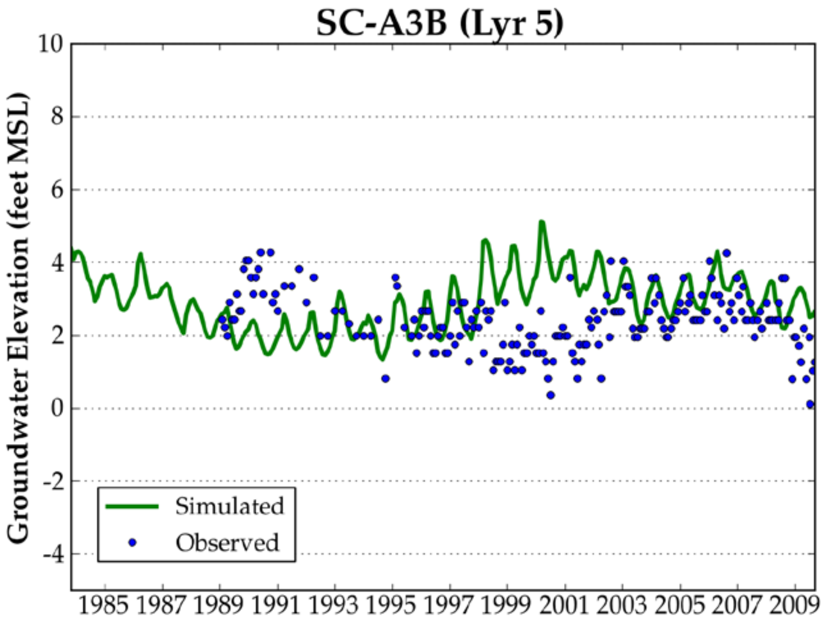
APPENDIX D: HYDROGRAPHS



SC-A3A (Lyrs 5-6)
Groundwater Elevation (feet MSL)
10
8
6
4
2
0
-2
-4
Simulated
Observed
1985 1987 1989 1991 1993 1995 1997 1999 2001 2003 2005 2007 2009



SC-A3B (Lyr 5)
Groundwater Elevation (feet MSL)
10
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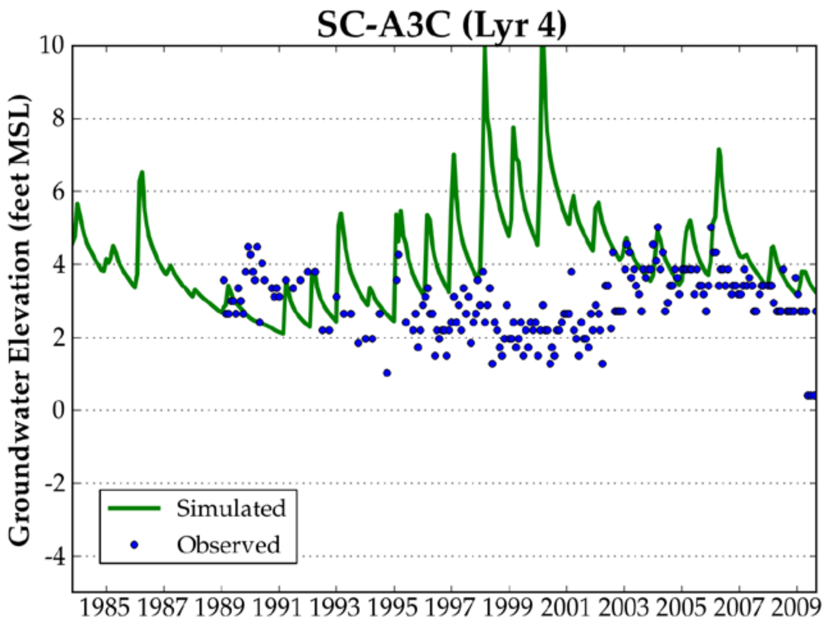
Simulated
Observed
1985 1987 1989 1991 1993 1995 1997 1999 2001 2003 2005 2007 2009



SC-A3C (Lyr 4)
Groundwater Elevation (feet MSL)
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Simulated
Observed
1985 1987 1989 1991 1993 1995 1997 1999 2001 2003 2005 2007 2009

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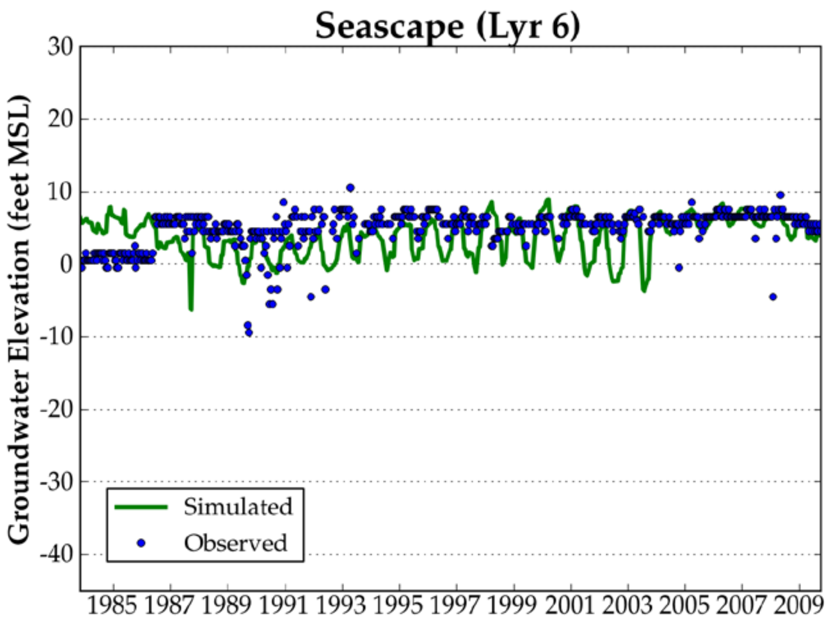
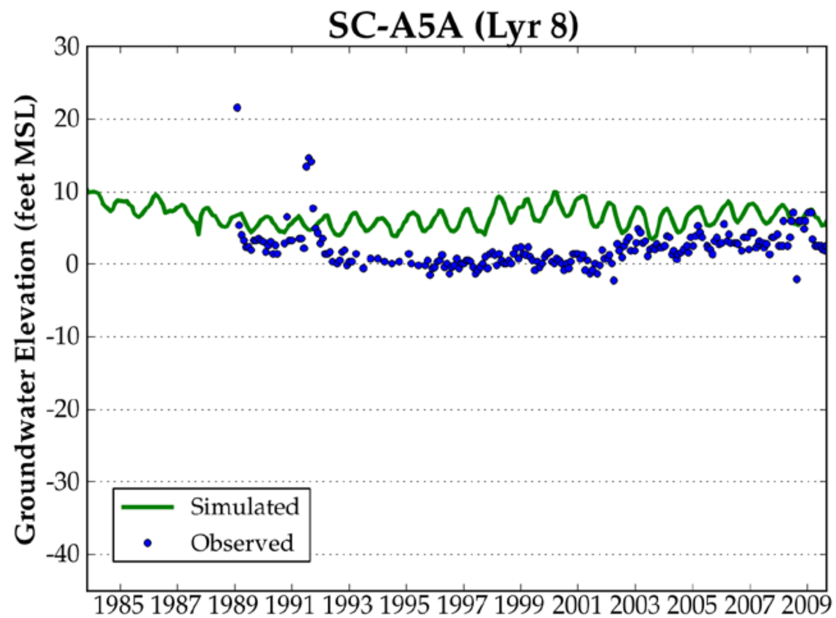

Seascape (Lyr 6)
Groundwater Elevation (feet MSL)
30
20
10
0
-10
-20
-30
-40
1985 1987 1989 1991 1993 1995 1997 1999 2001 2003 2005 2007 2009
Simulated
Observed


SC-A5A (Lyr 8)
Groundwater Elevation (feet MSL)
30
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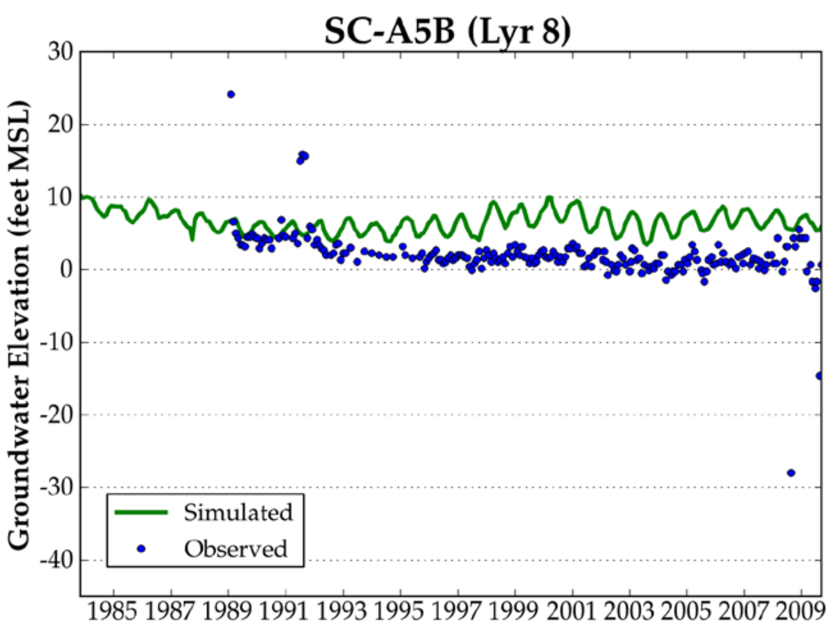
-30
-40
1985 1987 1989 1991 1993 1995 1997 1999 2001 2003 2005 2007 2009
Simulated
Observed


SC-A5B (Lyr 8)
Groundwater Elevation (feet MSL)
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Simulated
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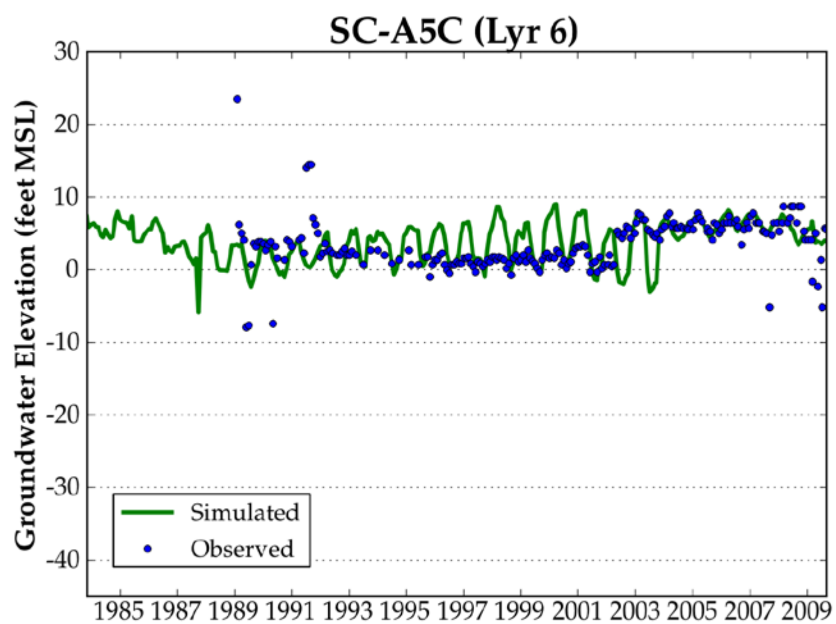
SC-A5C (Lyr 6)
Groundwater Elevation (feet MSL)
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Simulated
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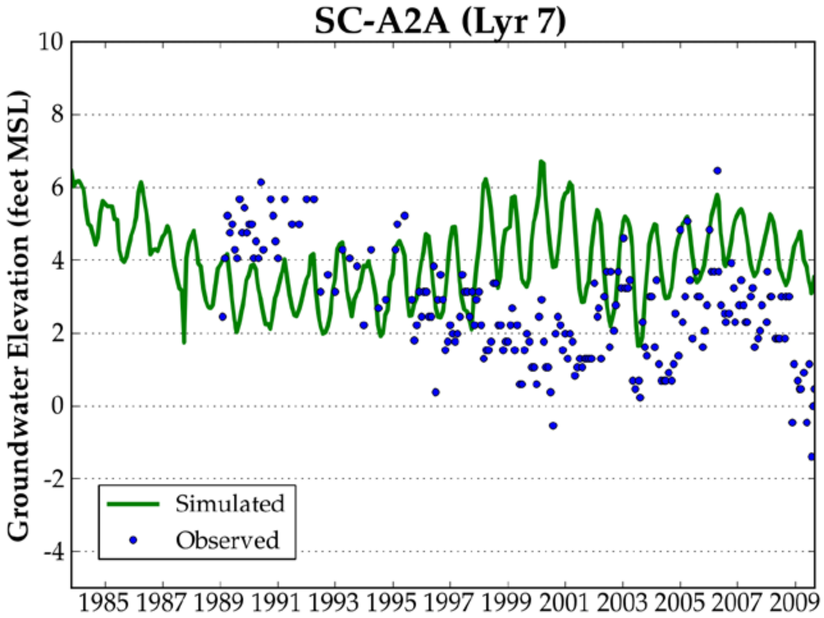
SC-A2A (Lyr 7)
Groundwater Elevation (feet MSL)
Simulated
Observed
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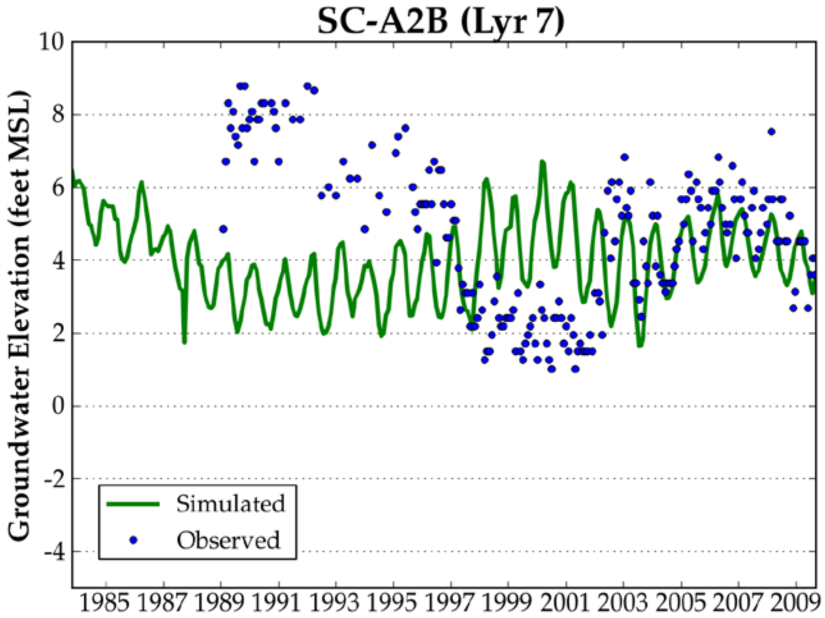
1985 1987 1989 1991 1993 1995 1997 1999 2001 2003 2005 2007 2009


SC-A2B (Lyr 7)
Groundwater Elevation (feet MSL)
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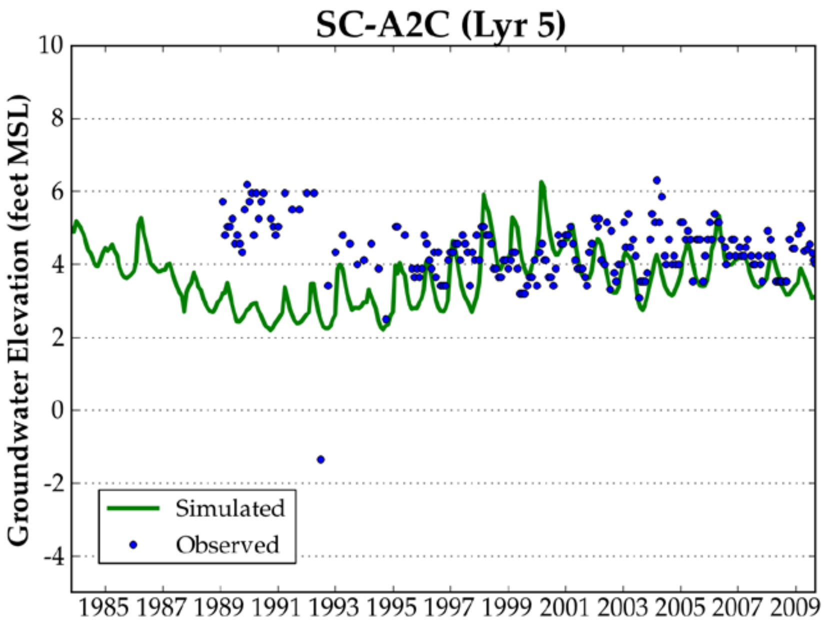
SC-A2C (Lyr 5)
Groundwater Elevation (feet MSL)
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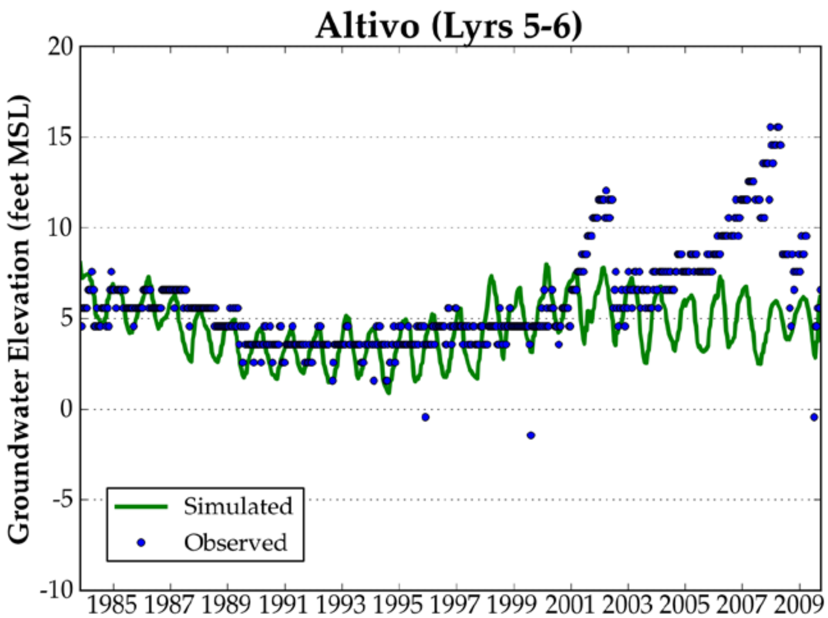
Altivo (Lyrs 5-6)
Groundwater Elevation (feet MSL)
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15
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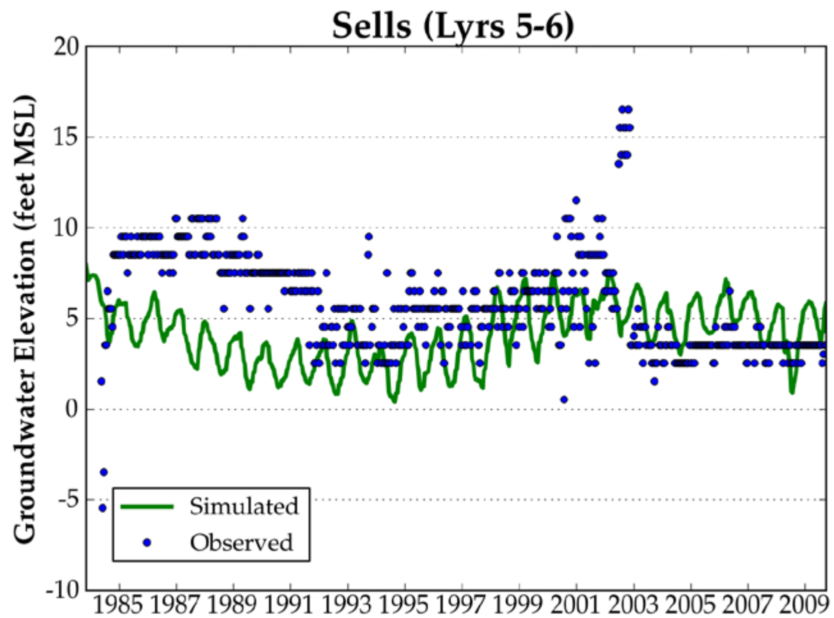
-5
-10
Simulated
Observed
1985 1987 1989 1991 1993 1995 1997 1999 2001 2003 2005 2007 2009


Sells (Lyrs 5-6)
Groundwater Elevation (feet MSL)
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Simulated
Observed
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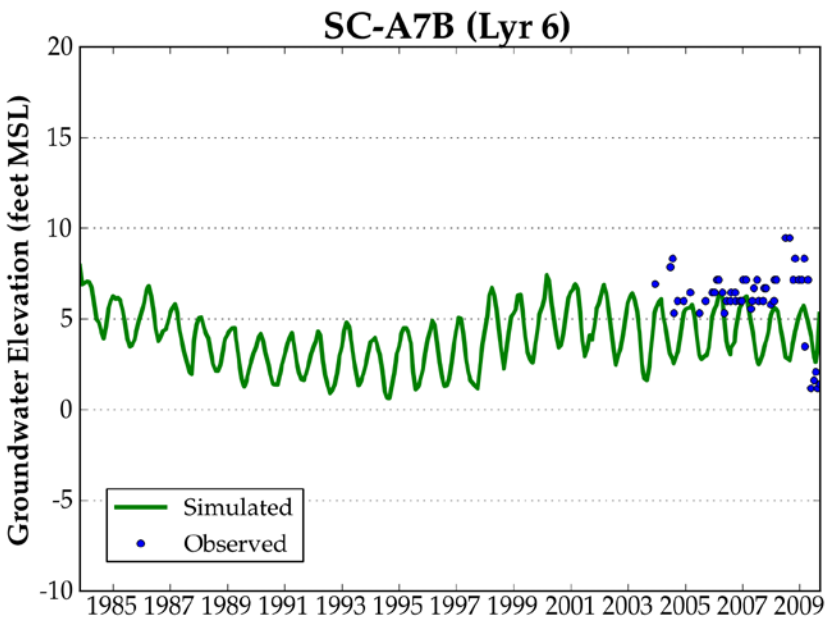
SC-A7B (Lyr 6)
Groundwater Elevation (feet MSL)
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Simulated
Observed
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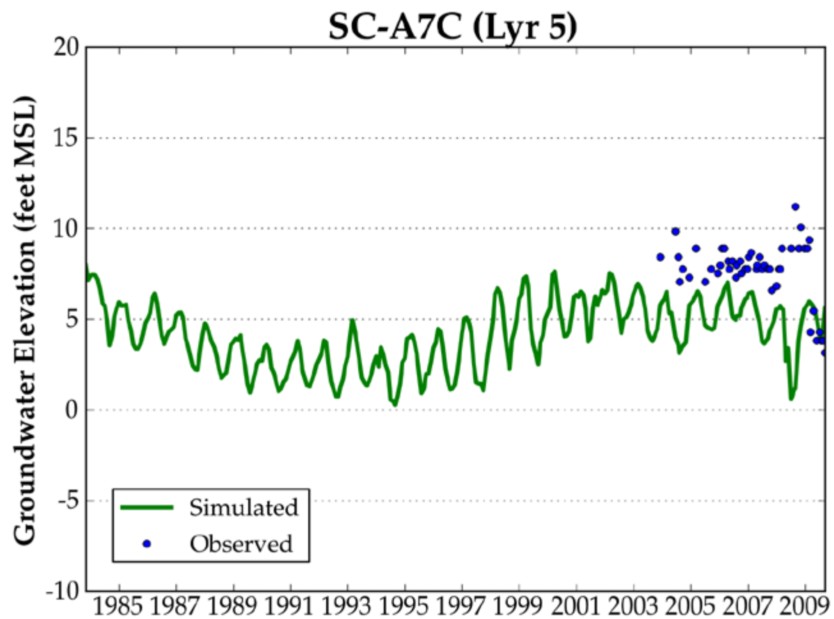
SC-A7C (Lyr 5)
Groundwater Elevation (feet MSL)
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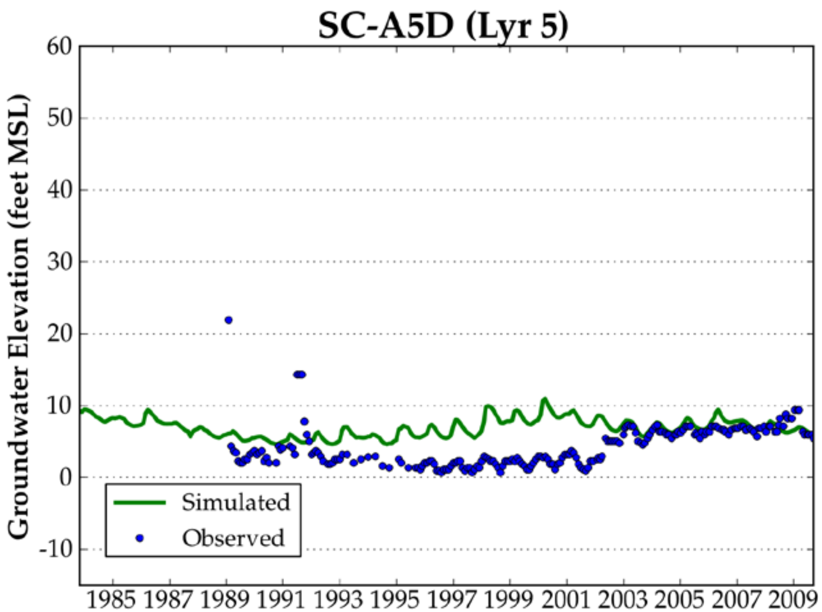
SC-A5D (Lyr 5)
Groundwater Elevation (feet MSL)
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Simulated
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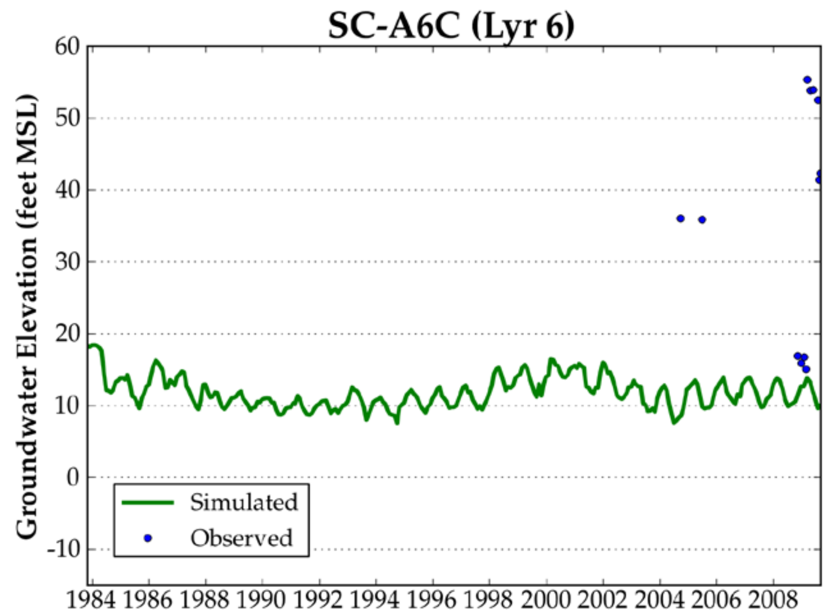
SC-A6C (Lyr 6)
Groundwater Elevation (feet MSL)
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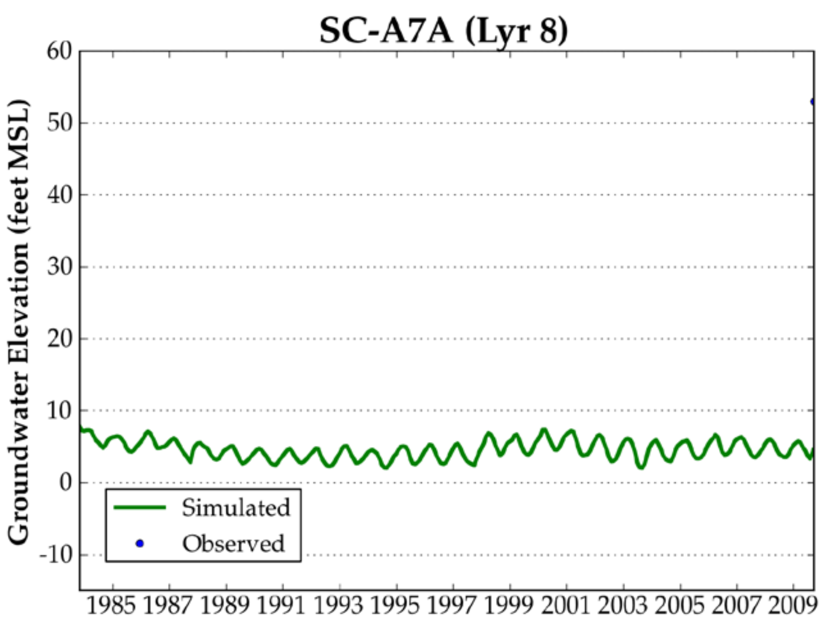
SC-A7A (Lyr 8)
Groundwater Elevation (feet MSL)
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Simulated
Observed
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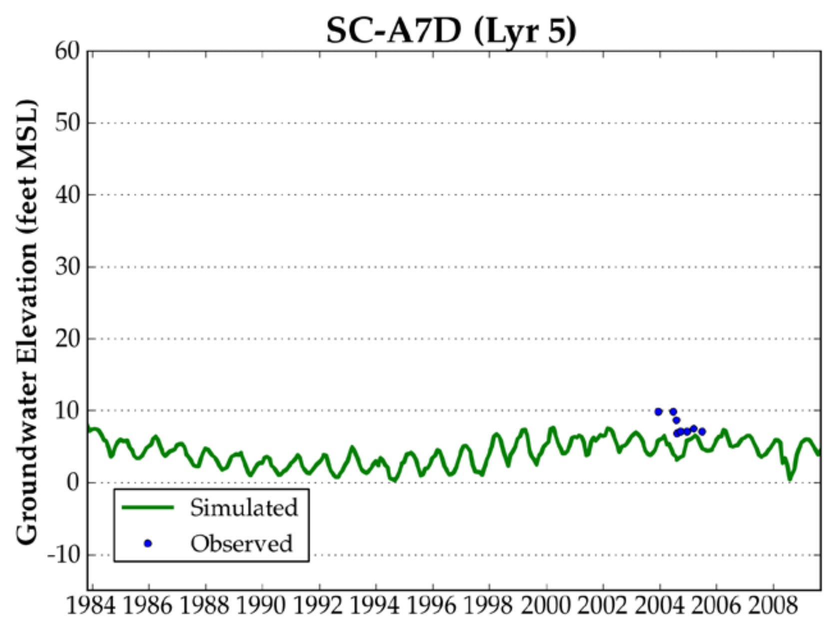
SC-A7D (Lyr 5)
Groundwater Elevation (feet MSL)
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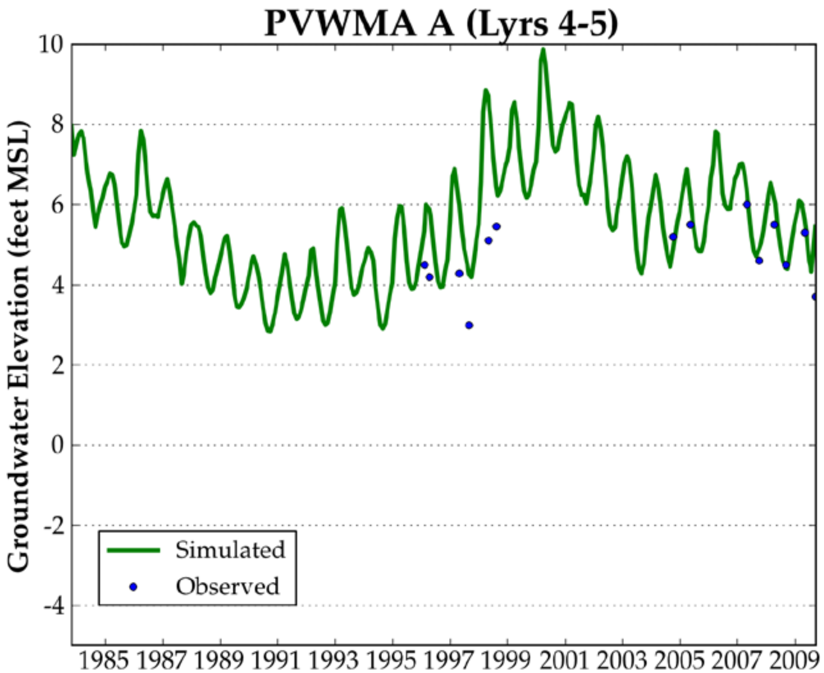
PVWMA A (Lyrs 4-5)
Groundwater Elevation (feet MSL)
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Simulated
Observed
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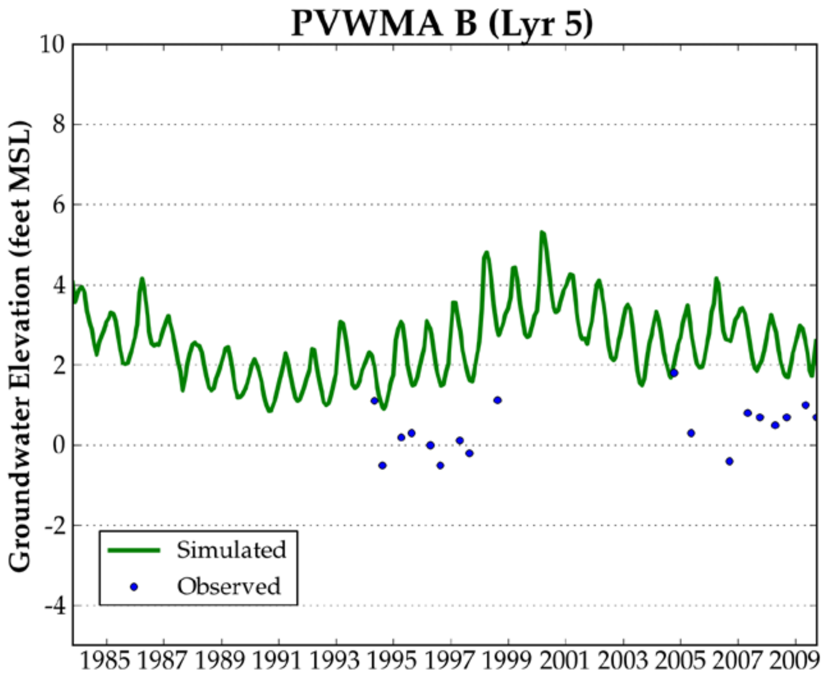
PVWMA B (Lyr 5)
Groundwater Elevation (feet MSL)
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APPENDIX E: CALIBRATED AND SIMULATION MODEL FILES ON CD

APPENDIX F: CONCEPTUAL DESIGN FOR IRON AND MANGANESE TREATMENT SYSTEM

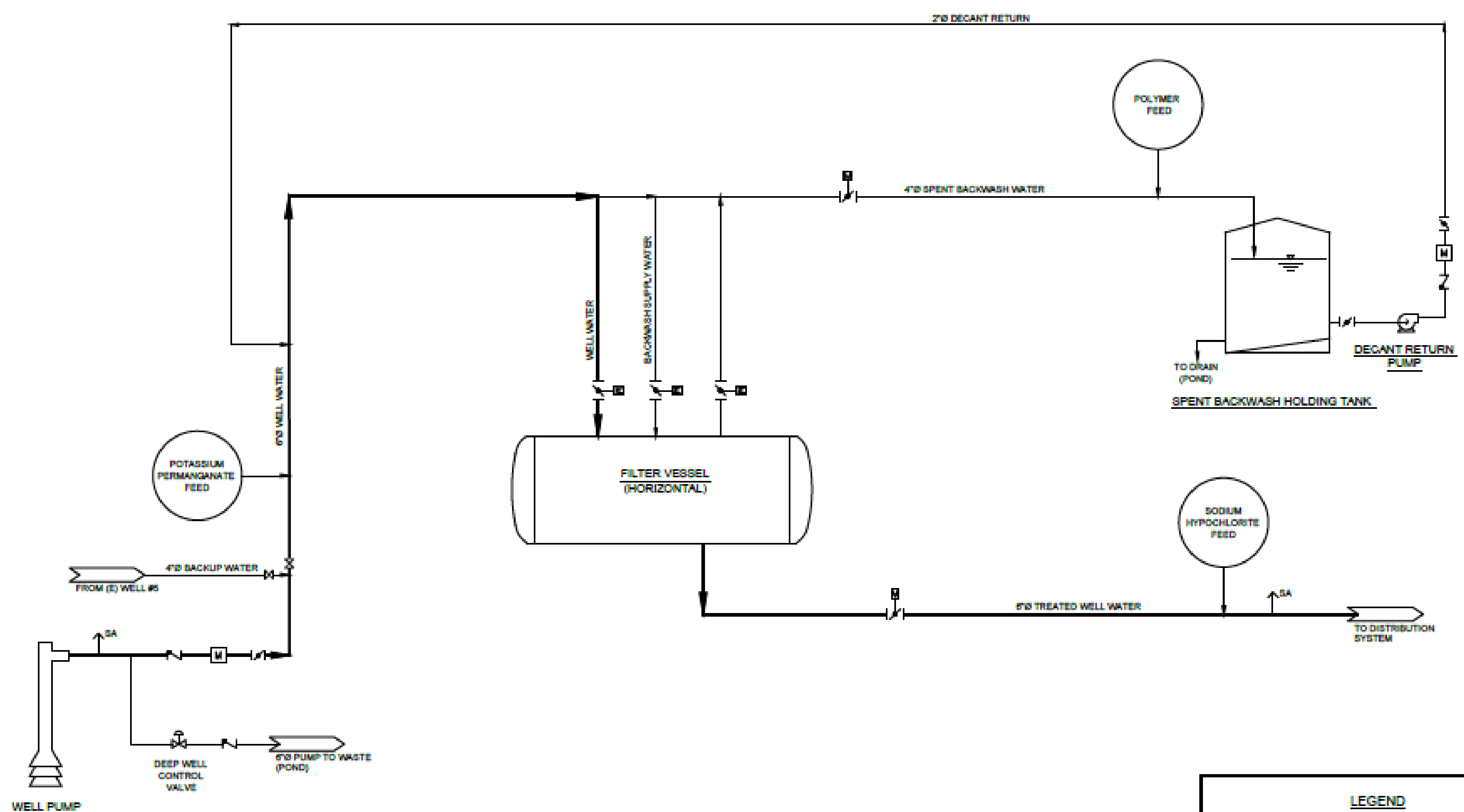
DRAWING SHEETS

DESIGN CRITERIA
(ALTERNATIVE 1 - HORIZONTAL FILTER)

1. DESIGN FLOW RATE	400 GPM
2. WATER QUALITY	
INFLUENT MANGANESE	0.518 MG/L
INFLUENT IRON	1.202 MG/L
MCL MANGANESE	0.050 MG/L
MCL IRON	0.3 MG/L
3. WELL	
NUMBER OF PUMPS	1
TYPE	VERTICAL TURBINE
CAPACITY	400 GPM
MOTOR	60 HP
4. TREATMENT FILTERS	
NUMBER OF FILTERS	1
FILTER TYPE	HORIZONTAL WELDED STEEL
DESIGN PRESSURE	100 PSI
FILTER MEDIA	GREENSAND W/ ANTHRACITE CAP
DIAMETER	7 FT
SHELL LENGTH	12 FT
OVERALL LENGTH	15 FT
NO. OF CELLS	3
CELL LENGTH	4 FT
CELL CAPACITY	200 GPM
CELL AREA	28 SF / CELL
BACKWASH RATE	12 GPM/SF
BACKWASH FLOWRATE	336 GPM / CELL
BACKWASH DURATION	10 MINUTES
BACKWASH & FILTER TO WASTE VOLUME	11,000 GALONS
BACKWASH FREQUENCY	24 HOURS
5. SPENT BACKWASH HOLDING TANK	
TYPE	WELDED STEEL
DIAMETER	14 FT
SIDE SHELL HEIGHT	12 FT
STORAGE CAPACITY	13,000 GALLONS
6. DECANT RETURN PUMPS	
NUMBER OF PUMPS	1
TYPE	CENTRIFUAL
CAPACITY	40 GPM
MOTOR	3 HP
7. CHEMICAL FEED SYSTEM	
POTASSIUM PERMANGANATE	
CONCENTRATION	20%
DOSAGE	2.2 MG/L
FEED RATE	0.1 GPH
TANK VOLUME	150 GALLONS
SODIUM HYPOCHLORITE	
CONCENTRATION	12.5%
DOSAGE	0.5 MG/L
FEED RATE	0.1 GPH
TANK VOLUME	150 GALLONS
POLYMER (AS NEEDED BASIS)	
CONCENTRATION	1.0%
DOSAGE	2.0 MG/L
FEED RATE	0.1 GPH
TANK VOLUME	55 GALLONS

NOTES:

1. NOT ALL VALVES, METERS OR EQUIPMENT ARE SHOWN FOR CLARITY.



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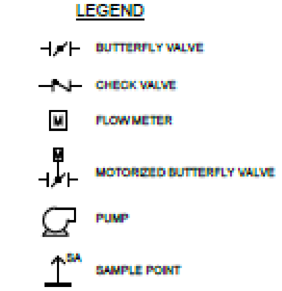

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SCALES
1"
25mm
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DESIGNED KMW
DRAWN CCE
CHECKED ARP

CENTRAL WATER DISTRICT
APTOS, CA
GROUNDWATER TREATMENT PROJECT

Kennedy/Jenks Consultants
10850 GOLD CENTER, DRIVE SUITE 350, RANCHO CORDOVA, CA

SCHEMATIC FLOW DIAGRAM
ALTERNATIVE 1 - HORIZONTAL FILTER

FILE NAME 1264008.00g02
JOB NO. 1264008.00
DATE JANUARY 2013
SHEET G2 OF --

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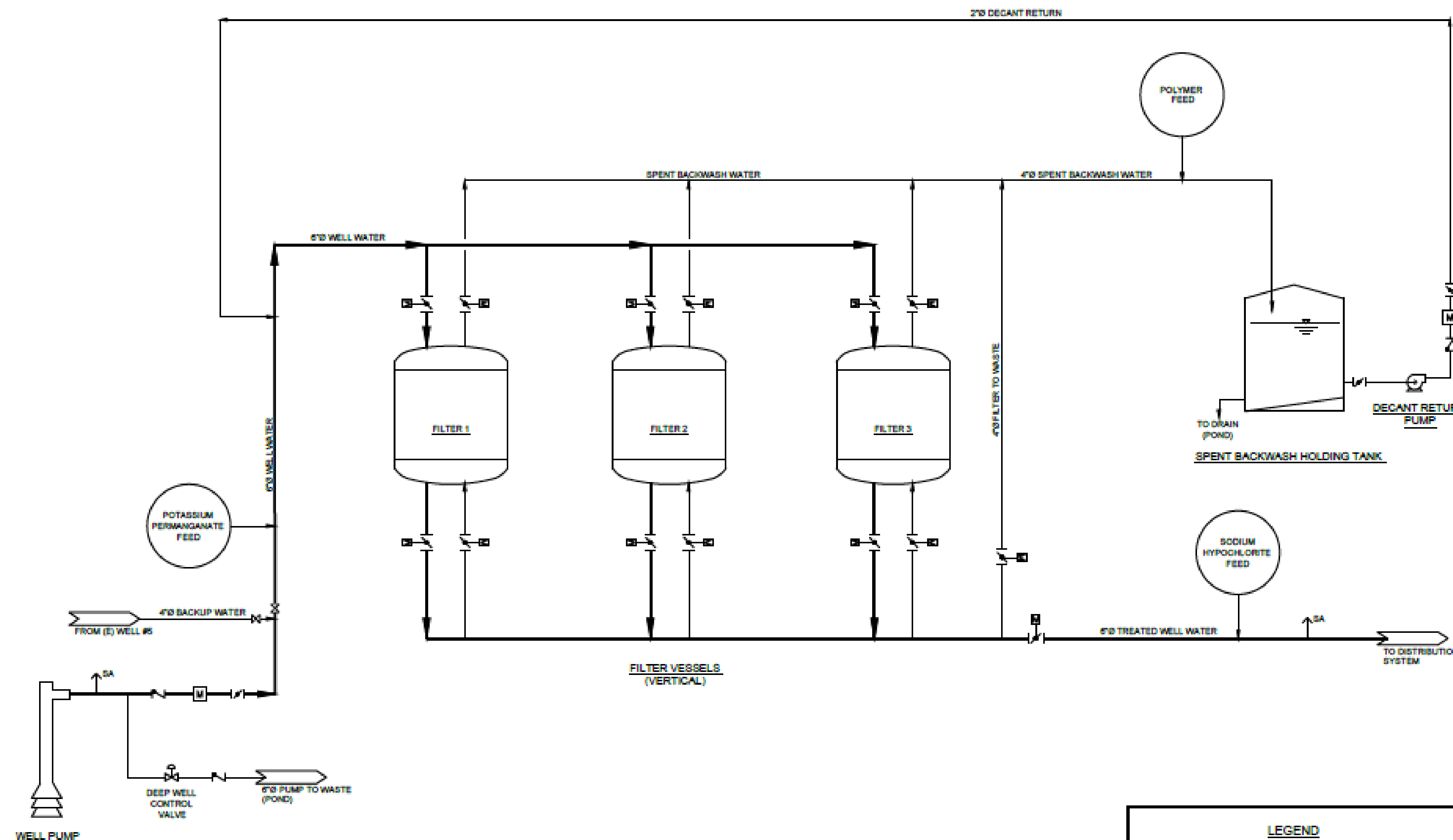
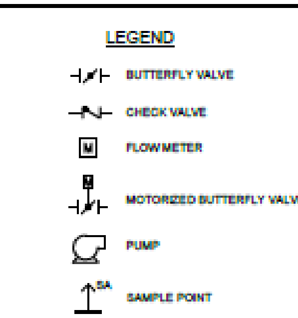
DESIGN CRITERIA
(ALTERNATIVE 2 - VERTICAL FILTERS)

1. DESIGN FLOW RATE	400 GPM
2. WATER QUALITY	
INFLUENT MANGANESE	0.518 MG/L
INFLUENT IRON	1.202 MG/L
MCL MANGANESE	0.050 MG/L
MCL IRON	0.3 MG/L
3. WELL	
NUMBER OF PUMPS	1
TYPE	VERTICAL TURBINE
CAPACITY	400 GPM
MOTOR	60 HP
4. TREATMENT FILTERS	
NUMBER OF FILTERS	3
FILTER TYPE	VERTICAL WELDED STEEL
DESIGN PRESSURE	100 PSI
FILTER MEDIA	GREENSAND W/ ANTHRACITE CAP
DIAMETER	6 FT
SIDE SHELL HEIGHT	6 FT
OVERALL HEIGHT	10 FT
FILTER CAPACITY	200 GPM / FILTER
FILTER AREA	28 SF / FILTER
BACKWASH RATE	12 GPM/SF
BACKWASH FLOWRATE	336 GPM / FILTER
BACKWASH DURATION	10 MINUTES
BACKWASH & FILTER TO WASTE VOLUME	11,000 GALONS
BACKWASH FREQUENCY	24 HOURS
5. SPENT BACKWASH HOLDING TANK	
TYPE	WELDED STEEL
DIAMETER	14 FT
SIDE SHELL HEIGHT	12 FT
STORAGE CAPACITY	13,000 GALLONS
6. DECANT RETURN PUMPS	
NUMBER OF PUMPS	1
TYPE	CENTRIFUAL
CAPACITY	40 GPM
MOTOR	3 HP
7. CHEMICAL FEED SYSTEM	
POTASSIUM PERMANGANATE	
CONCENTRATION	20%
DOSAGE	2.2 MG/L
FEED RATE	0.1 GPH
TANK VOLUME	160 GALLONS
SODIUM HYPOCHLORITE	
CONCENTRATION	12.5%
DOSAGE	0.5 MG/L
FEED RATE	0.1 GPH
TANK VOLUME	160 GALLONS
POLYMER (AS NEEDED BASIS)	
CONCENTRATION	1.0%
DOSAGE	2.0 MG/L
FEED RATE	0.1 GPH
TANK VOLUME	55 GALLONS

NOTES:

1. NOT ALL VALVES, METERS OR EQUIPMENT ARE SHOWN FOR CLARITY.





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1"

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APTOS, CA
GROUNDWATER TREATMENT PROJECT

Kennedy/Jenks Consultants
10850 GOLD CENTER, DRIVE SUITE 350, RANCHO CORDOVA, CA

SCHEMATIC FLOW DIAGRAM
ALTERNATIVE 2 - VERTICAL FILTERS

FILE NAME 1264006.00g03
JOB NO. 1264006.00
DATE JANUARY 2013
SHEET G3 OF --

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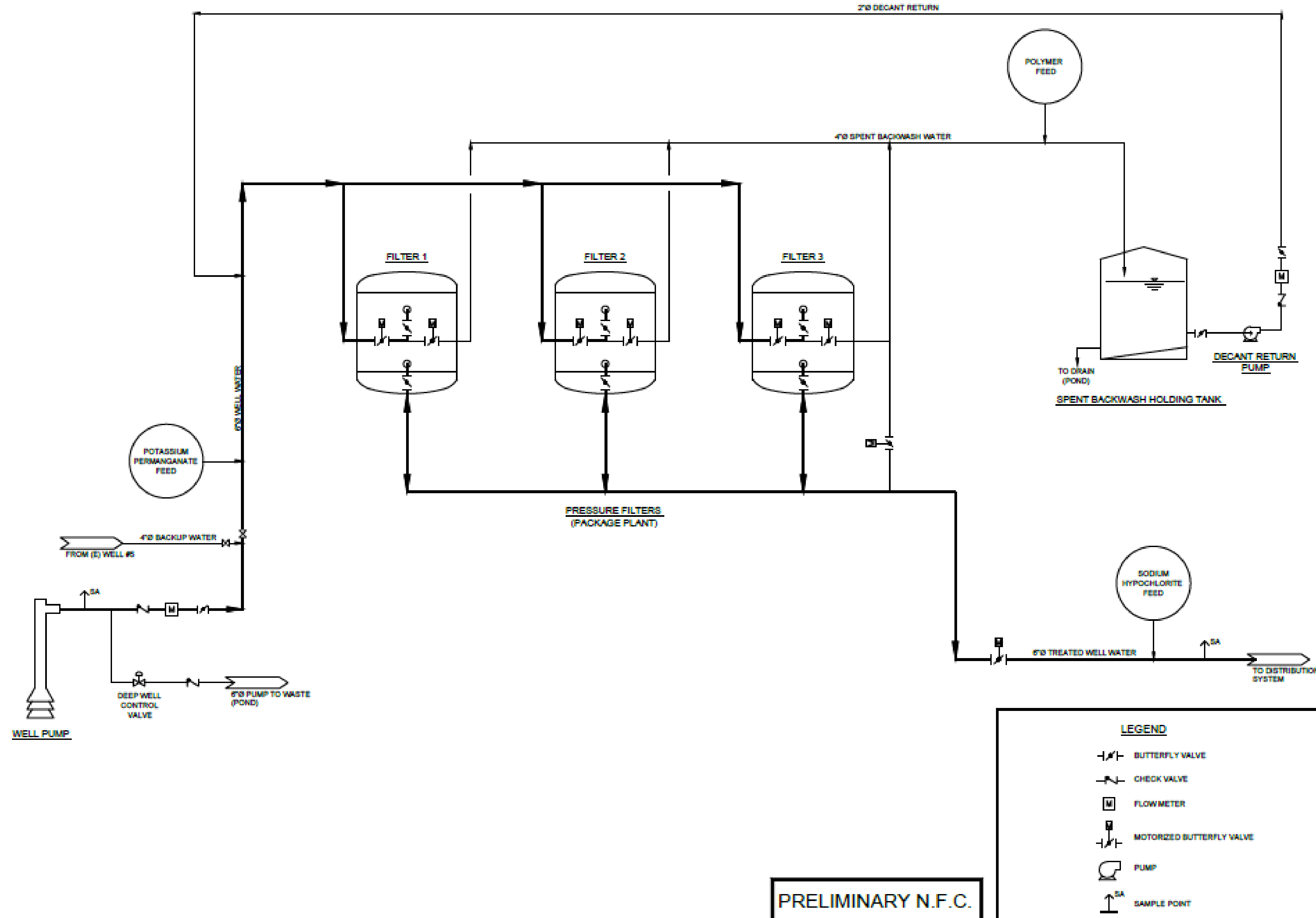
DESIGN CRITERIA
(ALTERNATIVE 3 - PACKAGE PLANT)

1. DESIGN FLOW RATE	400 GPM
2. WATER QUALITY	
INFLUENT MANGANESE	0.518 MG/L
INFLUENT IRON	1.202 MG/L
MCL MANGANESE	0.050 MG/L
MCL IRON	0.3 MG/L
3. WELL	
NUMBER OF PUMPS	1
TYPE	VERTICAL TURBINE
CAPACITY	400 GPM
MOTOR	60 HP
4. TREATMENT FILTERS	
NUMBER OF FILTERS	3
FILTER TYPE	VERTICAL WELDED STEEL
DESIGN PRESSURE	100 PSI
FILTER MEDIA	GREENSAND W/ ANTHRACITE CAP
DIAMETER	6 FT
SIDE SHELL HEIGHT	6 FT
OVERALL HEIGHT	10 FT
FILTER CAPACITY	200 GPM / FILTER
FILTER AREA	28 SF / FILTER
BACKWASH RATE	12 GPM/SF
BACKWASH FLOWRATE	336 GPM / FILTER
BACKWASH DURATION	10 MINUTES
BACKWASH & FILTER TO WASTE VOLUME	11,000 GALONS
BACKWASH FREQUENCY	24 HOURS
5. SPENT BACKWASH HOLDING TANK	
TYPE	WELDED STEEL
DIAMETER	14 FT
SIDE SHELL HEIGHT	12 FT
STORAGE CAPACITY	13,000 GALLONS
6. DECANT RETURN PUMPS	
NUMBER OF PUMPS	1
TYPE	CENTRIFUAL
CAPACITY	40 GPM
MOTOR	3 HP
7. CHEMICAL FEED SYSTEM	
POTASSIUM PERMANGANATE	
CONCENTRATION	20%
DOSAGE	2.2 MG/L
FEED RATE	0.1 GPH
TANK VOLUME	160 GALLONS
SODIUM HYPOCHLORITE	
CONCENTRATION	12.5%
DOSAGE	0.5 MG/L
FEED RATE	0.1 GPH
TANK VOLUME	160 GALLONS
POLYMER (AS NEEDED BASIS)	
CONCENTRATION	1.0%
DOSAGE	2.0 MG/L
FEED RATE	0.1 GPH
TANK VOLUME	55 GALLONS

NOTES:

1. NOT ALL VALVES, METERS OR EQUIPMENT ARE SHOWN FOR CLARITY.



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STATE OF CALIFORNIA

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CENTRAL WATER DISTRICT
APTOS, CA
GROUNDWATER TREATMENT PROJECT

Kennedy/Jenks Consultants
10850 GOLD CENTER, DRIVE SUITE 350, RANCHO CORDOVA, CA

SCHEMATIC FLOW DIAGRAM
ALTERNATIVE 3 - PACKAGE PLANT

FILE NAME 1264008.00g04
JOB NO. 1264008.00
DATE JANUARY 2013
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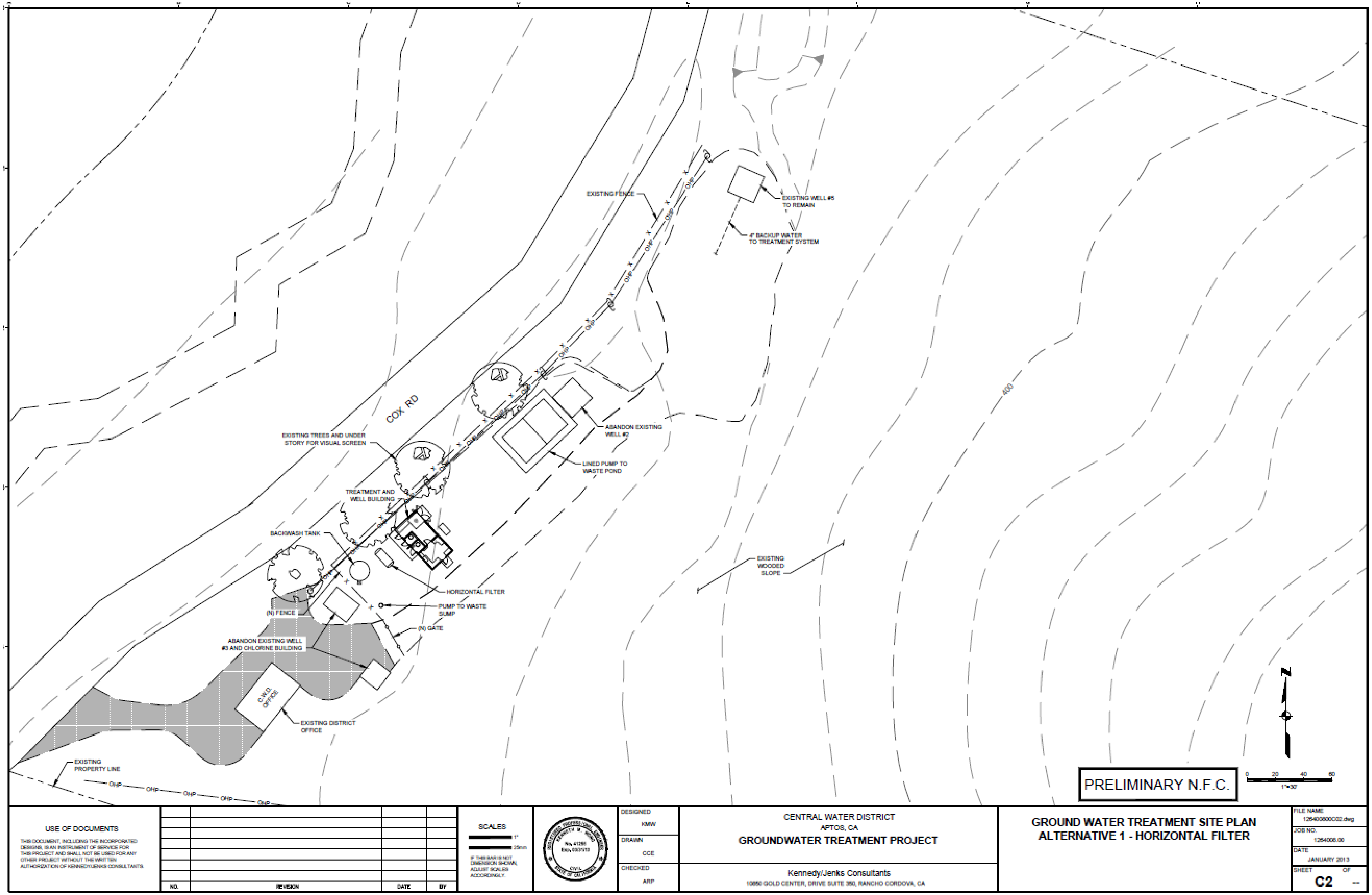
COX RD
EXISTING FENCE
EXISTING WELL #5 TO REMAIN
4" BACKUP WATER TO TREATMENT SYSTEM
EXISTING TREES AND UNDER STORY FOR VISUAL SCREEN
ABANDON EXISTING WELL #2
LINED PUMP TO WASTE POND
TREATMENT AND WELL BUILDING
BACKWASH TANK
HORIZONTAL FILTER
PUMP TO WASTE SUMP
(N) FENCE
(N) GATE
ABANDON EXISTING WELL #3 AND CHLORINE BUILDING
C.W.D. OFFICE
EXISTING DISTRICT OFFICE
EXISTING WOODED SLOPE
EXISTING PROPERTY LINE
OHP
400
N
PRELIMINARY N.F.C.
0 20 40 60
1"=30'
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DRAWN CCE
CHECKED ARP
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APTOS, CA
GROUNDWATER TREATMENT PROJECT
Kennedy/Jenks Consultants
10850 GOLD CENTER, DRIVE SUITE 350, RANCHO CORDOVA, CA
GROUND WATER TREATMENT SITE PLAN
ALTERNATIVE 1 - HORIZONTAL FILTER
FILE NAME
JOB NO.
1264008.00
DATE
JANUARY 2013
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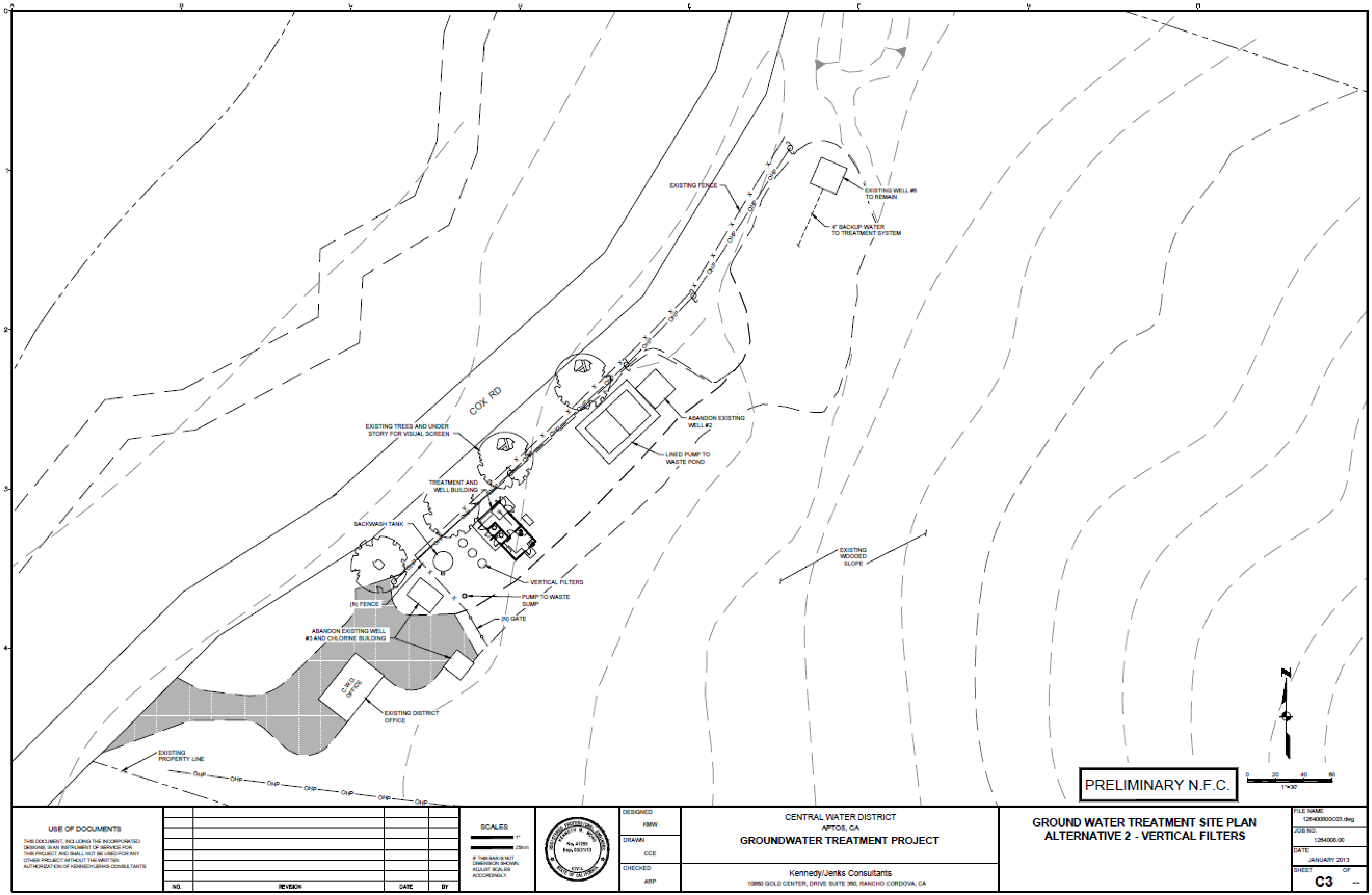

COX RD
EXISTING FENCE
EXISTING WELL #5 TO REMAIN
4" BACKUP WATER TO TREATMENT SYSTEM
ABANDON EXISTING WELL #2
LINED PUMP TO WASTE POND
EXISTING TREES AND UNDER STORY FOR VISUAL SCREEN
TREATMENT AND WELL BUILDING
BACKWASH TANK
VERTICAL FILTERS
PUMP TO WASTE SUMP
(N) FENCE
(N) GATE
ABANDON EXISTING WELL #3 AND CHLORINE BUILDING
C.W.D. OFFICE
EXISTING DISTRICT OFFICE
EXISTING WOODED SLOPE
EXISTING PROPERTY LINE
PRELIMINARY N.F.C.
0 20 40 60
1"=30'
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DESIGNED
KMW
DRAWN
CCE
CHECKED
ARP
CENTRAL WATER DISTRICT
APTOS, CA
GROUNDWATER TREATMENT PROJECT
Kennedy/Jenks Consultants
10850 GOLD CENTER, DRIVE SUITE 350, RANCHO CORDOVA, CA
GROUND WATER TREATMENT SITE PLAN
ALTERNATIVE 2 - VERTICAL FILTERS
FILE NAME
JOB NO.
DATE
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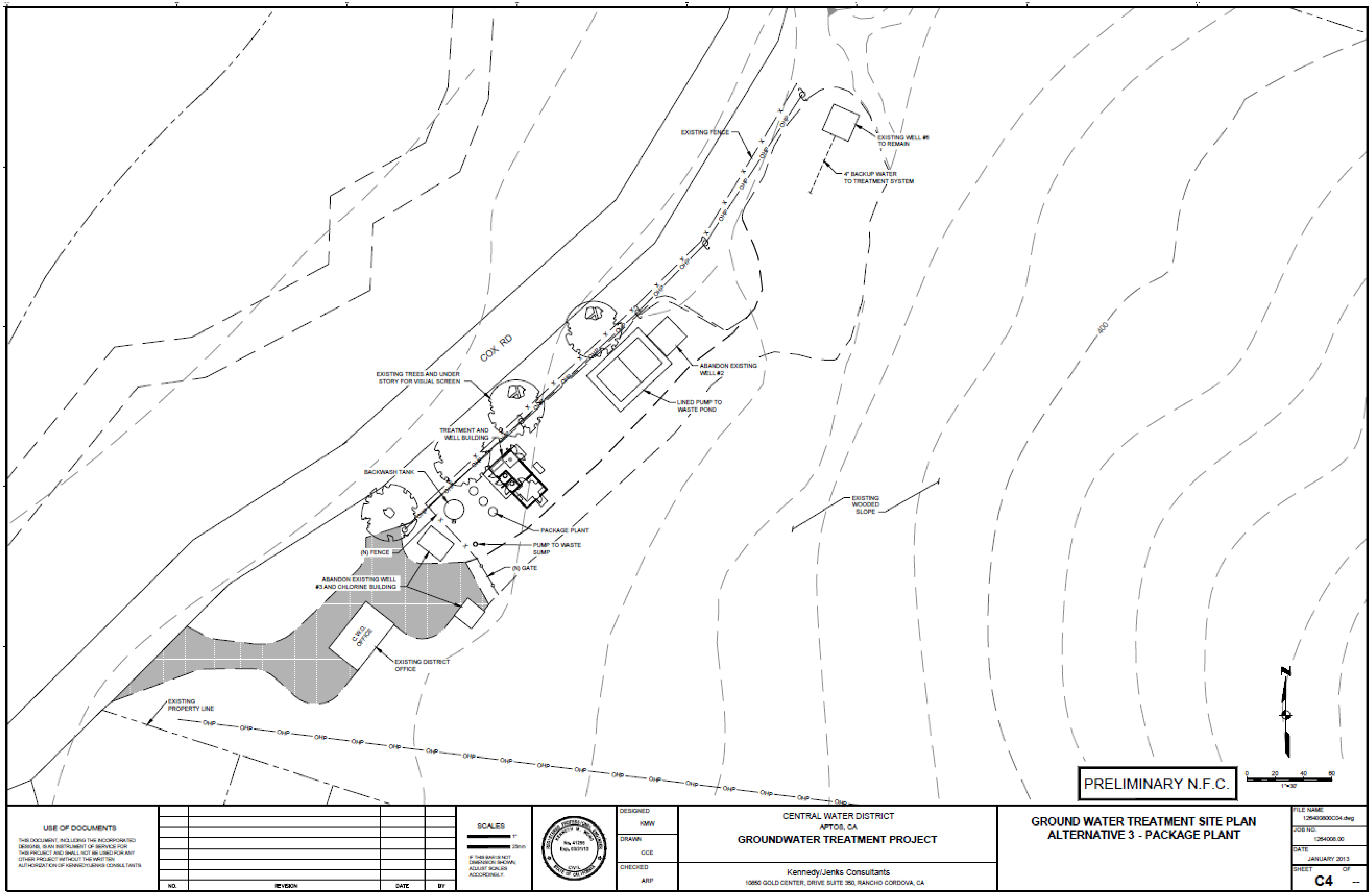
COX RD
EXISTING FENCE
EXISTING WELL #5 TO REMAIN
4" BACKUP WATER TO TREATMENT SYSTEM
ABANDON EXISTING WELL #2
LINED PUMP TO WASTE POND
EXISTING TREES AND UNDER STORY FOR VISUAL SCREEN
TREATMENT AND WELL BUILDING
BACKWASH TANK
PACKAGE PLANT
PUMP TO WASTE SUMP
(N) FENCE
(N) GATE
ABANDON EXISTING WELL #3 AND CHLORINE BUILDING
C.W.D. OFFICE
EXISTING DISTRICT OFFICE
EXISTING PROPERTY LINE
EXISTING WOODED SLOPE
400
PRELIMINARY N.F.C.
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APTOS, CA
GROUNDWATER TREATMENT PROJECT
Kennedy/Jenks Consultants
10850 GOLD CENTER, DRIVE SUITE 350, RANCHO CORDOVA, CA
GROUND WATER TREATMENT SITE PLAN
ALTERNATIVE 3 - PACKAGE PLANT
FILE NAME
JOB NO.
1264006.00
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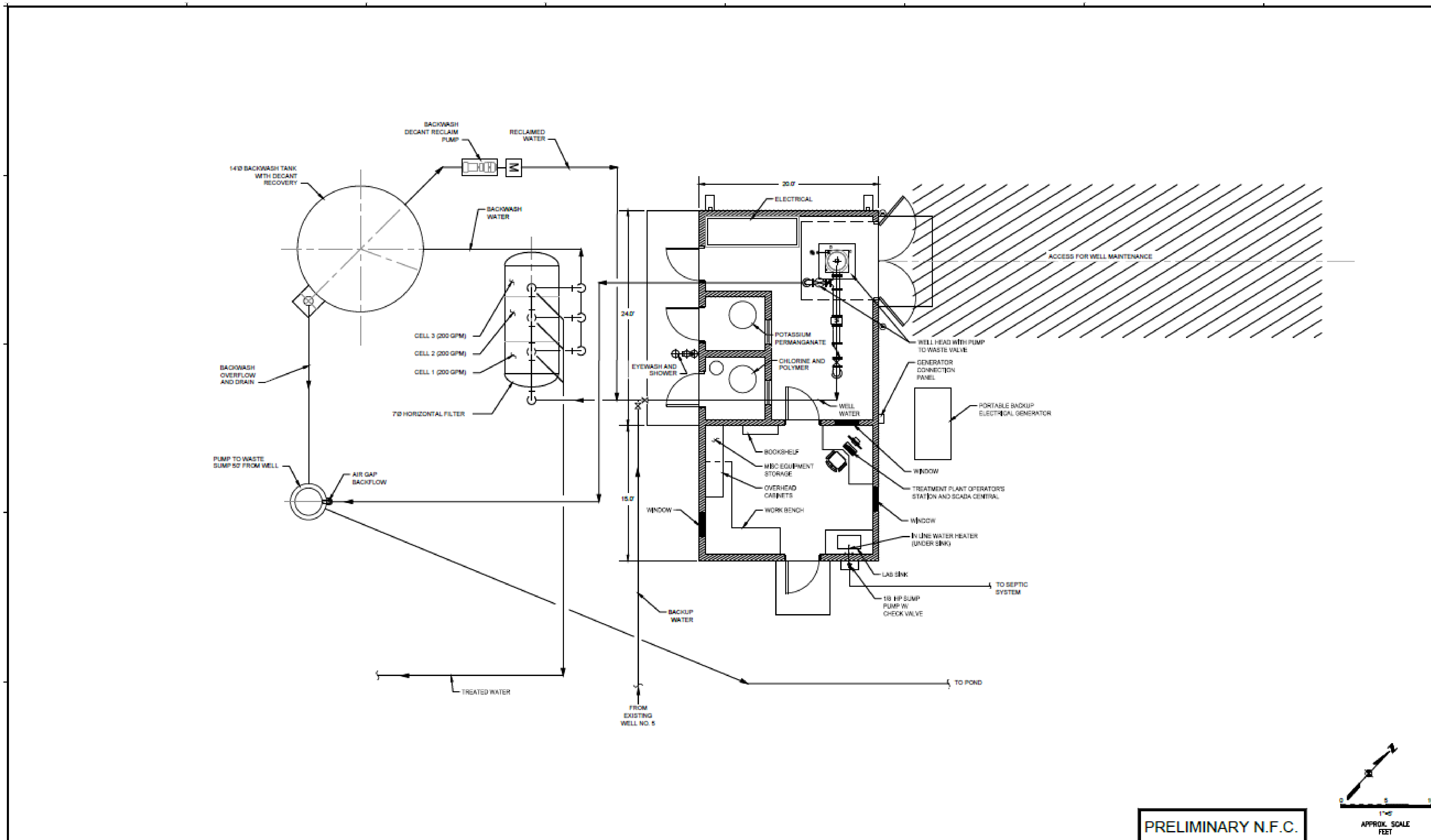

USE OF DOCUMENTS	NO.	REVISION	DATE	BY	SCALES	DESIGNED KMW / DRAWN CCE / CHECKED ARP	CENTRAL WATER DISTRICT APTOS, CA GROUNDWATER TREATMENT PROJECT	CONCEPTUAL SITE PLAN ALTERNATIVE 1 - HORIZONTAL FILTER	FILE NAME 126400800C05.dwg
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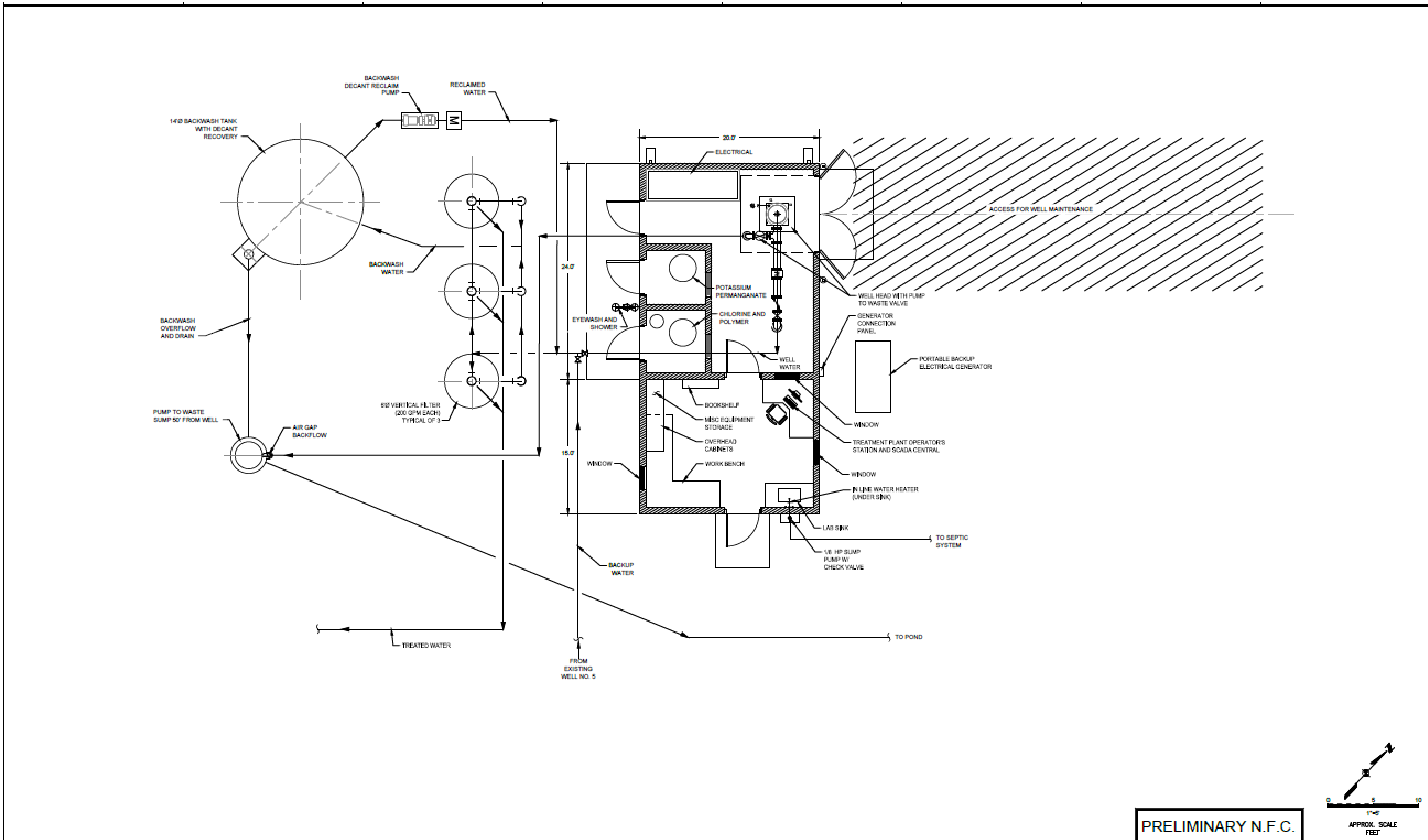

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CENTRAL WATER DISTRICT
APTOS, CA
GROUNDWATER TREATMENT PROJECT

Kennedy/Jenks Consultants
10850 GOLD CENTER, DRIVE SUITE 350, RANCHO CORDOVA, CA

CONCEPTUAL SITE PLAN
ALTERNATIVE 2 - VERTICAL FILTERS

FILE NAME 126400800C06.dwg
JOB NO. 1264008.00
DATE JANUARY 2013
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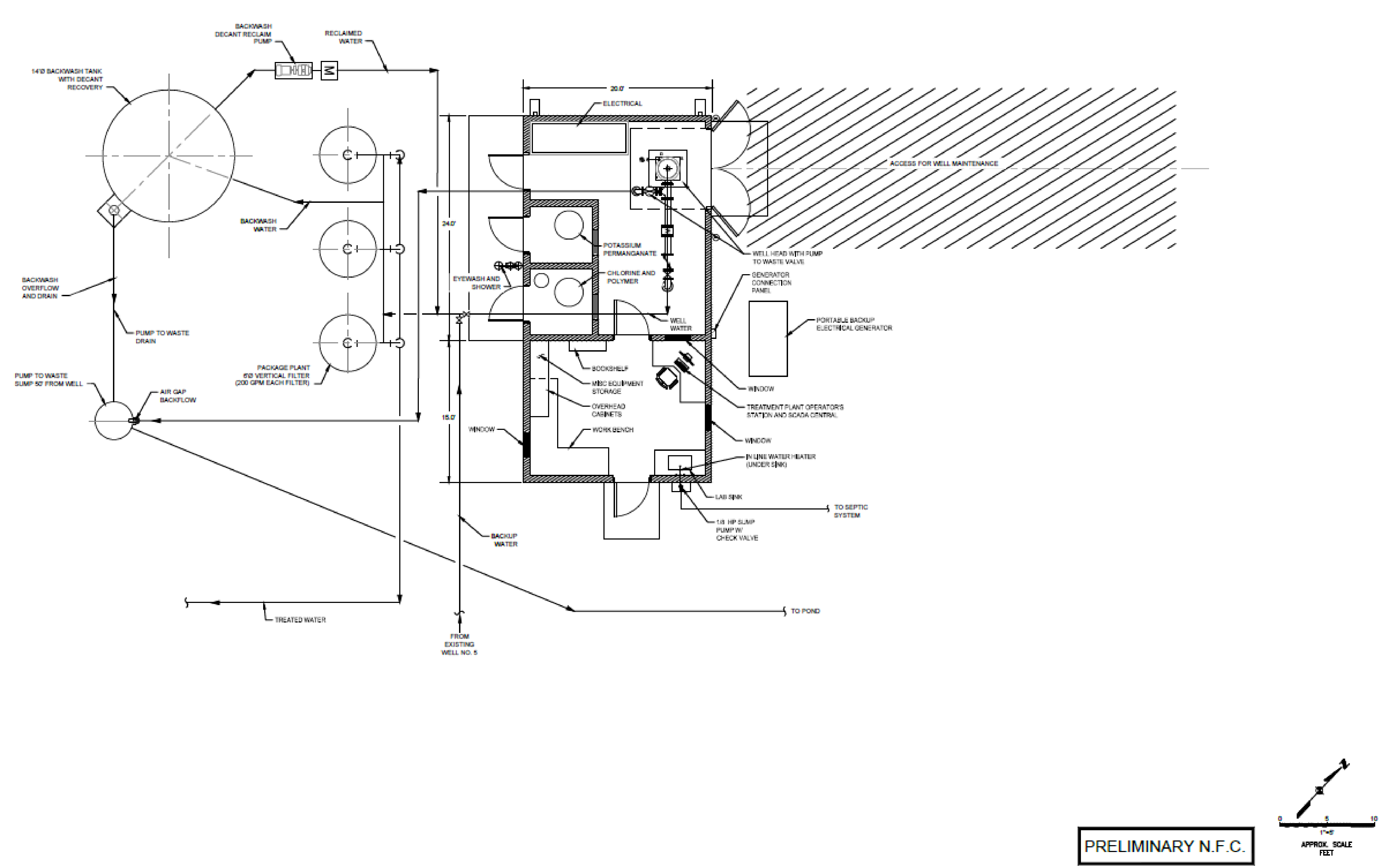

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						DRAWN CCE	GROUNDWATER TREATMENT PROJECT		JOB NO. 1254008.00
						CHECKED ARP	Kennedy/Jenks Consultants 10850 GOLD CENTER, DRIVE SUITE 350, RANCHO CORDOVA, CA		DATE JANUARY 2013
									SHEET C7 OF –

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